Lake Ray Hubbard Bridge
An Opportunity to Innovate

Abstract

Three miles of bridge across Lake Ray Hubbard on State Highway 66 East of Dallas offered plenty of opportunities for testing new products and methods for bridge construction. These opportunities exercised by the Texas Department of Transportation and Traylor Bros. Inc., the general contractor for the construction of this project.

The $40,000,000 construction project consisted of two bridges measuring approximately 2 miles and one mile in length, each two lanes wide, with slurry displaced drilled shafts, precast concrete beams, deck panels, a 4" cast in place concrete deck and an extruded concrete parapet wall.

The Lake Ray Hubbard Bridge implemented precast bent caps to overcome a potential safety hazard that existed on the project. Although the majority of the caps were cast in place, 43 precast caps were fabricated on the land, shipped by barge and erected by cranes working off a temporary construction bridge. The implementation of this technology prevented the contractor from being delayed approximately one week per bent cap over the duration of the construction of the bridge, thus saving the State of Texas at least 43 weeks of potential delay damages.

Ground Granulated Blast Furnace Slag (GGBFS) was incorporated into several of the concrete mixes to increase durability of the structure. The Federal Highway Administration offered Innovative Bridge Research Funding to pay for the implementation of this change.

To overcome the slightly slower cure times of the GGBFS mixes, maturity testing was implemented to help determine key concrete strength goals. Maturity testing is a relatively new test method that allows for in situ strength determinations for concrete, based on the heat of hydration and the cure time. Maturity testing develops a numeric depiction of the strength gain for any particular concrete batch design which can be replicated in the field using thermocouples and maturity meters which measure internal concrete temperature and time.

Lastly, TxDOT decided to try an alternative to tining the shorter of the two bridges. Since the first new bridge, showed some signs of possible drying shrinkage cracking, the second bridge was not tined during concrete deck placement. After completion of the structure, deck texturing was saw cut into the surface, for a small added price, but a potential for great improvement to the life of the bridge.

In summary, this bridge offered many chances to implement new technology. These chances were taken by both TxDOT and Traylor Bros. Inc, to the benefit of the taxpayers of Texas.

Lake Ray Hubbard separates the small suburb of Rockwall, Texas from the City of Dallas. Interstate 30 and State Highway 66 are the only East to West crossings of Lake Ray Hubbard, and are the major means of commuters to travel to the city. Although the Interstate bridge was improved 6 years ago, the State Highway 66 bridge, which carried two way traffic for 40 years, consisted of a 26 foot wide structure with no shoulders or median divider.

The State Highway 66 Lake Ray Hubbard Bridge construction consists of two structures totaling about $35,000,000 of the overall $40,000,000 project. The Westbound bridge is 2 miles in length, 48 foot wide with 104 bents spaced at 100foot intervals, 3 drill shafts per bent, cast in place caps, Type IV prestressed concrete beams, 4" precast concrete deck panels and a 4 inch concrete cast in place deck. This bridge includes two traffic lanes, and a separated bicycle lane. The Eastbound Bridge is slightly less than one mile long with 45- 100 foot bents, a width of 40 feet, similar drill shaft and deck construction and configuration, and precast bent caps.

Most of the bridge decks of both the East and Westbound bridges contain 35% Ground Granulated Blast Furnace Slag replacement of Portland Cement. The Westbound bridge deck was tined during the deck placement operation while the Eastbound bridge deck was grooved after curing. Maturity testing was implemented for verification of strength in all structural elements as well as the bridge deck.

The following is a more in depth discussion of the technology implemented during the construction of this bridge.

Ground Granulated Blast Furnace Slag.

In an effort to test concrete durability, it was decided that the superstructure concrete for the Lake Ray Hubbard Bridge would be modified with a cement replacement of 35% Ground Granulated Blast Furnace Slag (GGBFS). GGBFS is a cementitious by-product of iron production.

Our intentions were to use GGBFS for the first structure as a 35% or 50% replacement for cement, and to use 100% Portland Cement for the second structure. However, the finishing properties, ultimate strength, the low heat of hydration, and corresponding lower early strengths changed our plans somewhat. Since this project was time sensitive, some portions of the project were not conducive to the use of GGBFS. Specifically, drill shafts had a very quick turnaround time, and the contractor had a limited number of shaft forms. Thus, GGBFS would have slowed the production by about one day per bent. It was decided that Portland cement alone would be a more practical ingredient in the shafts.

After much testing, three mixes utilizing 35% replacement GGBFS was used for bridge caps, bridge decks and barrier rail. Originally the deck of the second bridge was planned to utilize 100% Portland Cement. However, when the ease of finish was discovered, it was decided that both decks would be placed with the GGBFS substitution.

Some interesting properties of the GGBFS were noted in the field application. One significant observation was the slump retention during pumping operations. Deck concrete was pumped up to 1150 feet. Under normal operations, it would be reasonable to lose 1 to 2 inches of slump from the concrete pump to the end of the pump line. However, the GGBFS concrete lost little to no slump.

Finishing of the GGBFS concrete proved to be somewhat easier as well. The concrete had a somewhat less "sticky" consistency, and did not stick to trowels which made the finishing quicker. Also, the initial set was slower due to the lower heat of hydration.

Although early strengths are lower, test results show that the concrete with GGBFS generally gained required 28 day compressive strengths of 4000 psi in 7 days. The following is a graph representing the strength development of the GGBFS mixes as opposed to the straight Portland Cement mixes on this project.

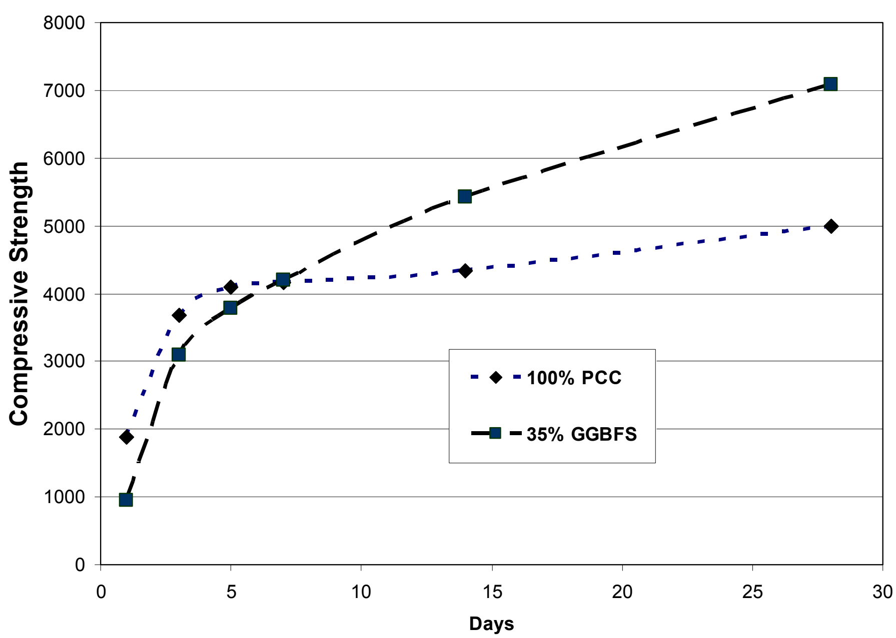

Figure 1: Strength Gain Comparison

Early permeability testing has been performed on the 35% GGBFS mix used for the bridge deck. ASTM Test Method C1202, "Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration," [1] more commonly known as the rapid chloride permeability test. This test passes a 60V current through a section of a 4" cylinder. The total current passing through the section over time is measured in Coulombs. A lower value indicates less current is able to pass through, thus less moisture would be able to pass through as well. [2]

The results of the permeability testing of the 35% GGBFS mix for bridge decks at 56 days ranged from 1450 Coulombs to 1700 Coulombs. All these numbers are considered in the "Low" permeability range. Further tests are pending on the 100% Portland Cement mix, and a control mix used for approach slabs, which contained 50% GGBFS substitution.

It should be noted that the contractor was hesitant to use the 50% substitution due to the low early strengths. The low strengths appeared to be limited to the first three or so days. Seven day compressive strengths for the 50% mix exceeded 5000 psi. Although GGBFS does provide an excellent end product, on time critical work, the added delay for strength gain should be considered.

An added advantage to the use of the GGBFS is the attractive finish the concrete gains. Although some GGBFS concrete is extremely white, with the aggregates available in the Dallas area, the finished produce has a slightly tan color. About $60,000 was saved when the painting item was removed from the plans.

Maturity testing

In an effort to counteract some of the slow strength development times of the GGBFS mixes, TxDOT implemented maturity testing on this project.

Maturity testing is a non-destructive test method which utilizes standard concrete strength testing methods as a basis, but integrates technology to streamline and improve the process. [3]

The Maturity method of strength testing is founded on the premise that strength gain in concrete is a factor of time and heat of hydration, and that for any given concrete design, there exists a profile, or "fingerprint" of that design that defines the strength gain over time. The fingerprint of that specific design can be graphically depicted and will be uniform for that design, regardless of the external

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[1] ASTM C1202-97 *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, American Society for Testing Materials, West Conshohocken, PA, 2001.
[2] Slag Cement Association, *Reducing Permeability*, WWW.slagcement.org.
[3], Proceedings, Texas Section American Society of Civil Engineers Fall Meeting, 1999, *Development of a Maturity Specification for Concrete Testing in Texas* ,Tracey Friggle, P.E., Pages 231-237.

weather conditions. To clarify, any particular concrete design will eventually gain a particular strength. As the heat of hydration for the mix is increased, the time needed to reach that strength is decreased, and vice-versa. So, if concrete temperature is plotted against time, the area under the curve at a given strength is equal regardless of the curing conditions. This is shown in the following pictorial example:

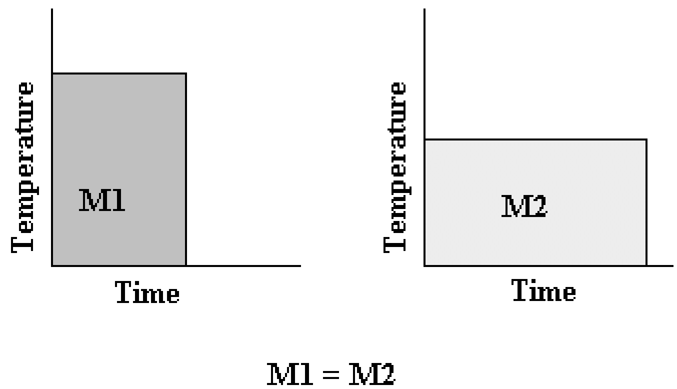

Figure 2

It follows then, that when a function of heat and time, or maturity, of any particular concrete is plotted against the strength of that concrete, a curve is formed that depicts this fingerprint. From determining the maturity of concrete at any given time, we are able to determine the corresponding strength of the concrete at that time.

Standard TxDOT specifications require strength and cure time for operations to proceed. For example, direct traffic is not allowed on bridge decks for 21 days, although cure time is just four days. Formwork and falsework for structural elements must remain in place for a minimum of 4 days. Regular traffic cannot be allowed on concrete pavement until it is 7 days old. These requirements are in addition to minimum strength requirements. Most cylinder and beam breaks are done on the 7th day to avoid the risk of low early strengths, since lack of sufficient test specimen breaks could result in a delay of continuing operations for 14 days[4]

Maturity testing allows field personnel to monitor strengths continuously or as needed for continuing operations. This means that when cure times are met, and strength is known, operations can proceed.

For example, under standard specifications, bridge caps would have to have forms in place for 4 days. Forms could be removed on the 4th day if test results were favorable. On the seventh day, more cylinders would be broken to verify design strength. If strength was obtained in test specimens, girders could be set. If the

[4] *Texas Department of Transportation 1993 Standard Specifications for Construction of Highways, Streets and Bridges*. Item 420, Concrete Structures and Item 421, Portland Cement Concrete, Pages 570 to 622.

test specimens indicated lower than required strength, the operations would cease until the 14th day. Coring of the element might be required.

For the same example, when maturity indicates strength, forms could be removed and curing blankets added, to maximize form use, even in less than 4 days. Beams could be set on the 5th day if maturity indicated appropriate strength. Of course, in the winter, the maturity number would be lower, due to cold weather lowering the heat of hydration. Ultimately, most operations are shorter in duration, but those that are longer are so for the justifiable reason of certain low early strengths.

This process is inherently better for determining strengths in the winter. Using the old cylinder breaks, strengths were correlated to a test specimen that was cured in a warm bath. This is not representative of the field member, which probably suffered some loss of heat of hydration in cold weather. Using maturity testing, we were ensured that the measurement in the field was representative of the actual structural member.

Maturity testing proved useful on the Lake Ray Hubbard Bridge, to counteract the lower early strengths of concrete with GGBFS, and to speed the overall bridge construction. GGBFS in many instances required forms to be left in place for structural elements an extra day. This slowed down the forming operations. To counteract this result, maturity usually indicated seven day strength long before the seventh day. This accelerated the following operation, thus allowing the overall construction to stay on or ahead of schedule.

In the end, although the GGBFS had lower early strengths than would be expected from straight cement mixes, the bridge construction proceeded much more quickly than standard testing would have allowed. The entire structure, with 328 drilled shafts, 111 columns, 102 caps and 53,000 square yards of deck was completed in 15 months.

The test procedure for developing a strength maturity relationship is outlined in ASTM C 1074 (1)[5]. Maturity testing is implemented as follows. The first step requires the development of a strength-maturity relationship. Once the mix design is developed, test beams or cylinders are produced. Maturity meters, which record the time and temperature through the use of a thermocouple, are incorporated into a few of the specimens. Then the strengths and maturity readings are recorded at specific increments. The Texas specifications require testing at 1, 3,7,14 and 28 day increments.[6] Generally, a 5 day break is added for a more detailed curve.

[5] American Society for Testing and Materials (1995). 1995 *Annual Book of ASTM Standards.* Volume 04.02 Concrete and Aggregates. Philadelphia, PA. Designation C1074-93. “Standard Practice for Estimating Concrete Strength by Maturity Method.” Pp 537-543.

[6] Texas Department of Transportation Standard Specifications for Construction of Highways, Streets and Bridges, 1993. Special Provision 420-014 Concrete Structures, and Special Provision 421-032 Portland Cement Concrete.

The data points are recorded, and a curve plotted. The corresponding function of that curve defines the maturity relationship for that particular design[7]. The following is an example of that data.

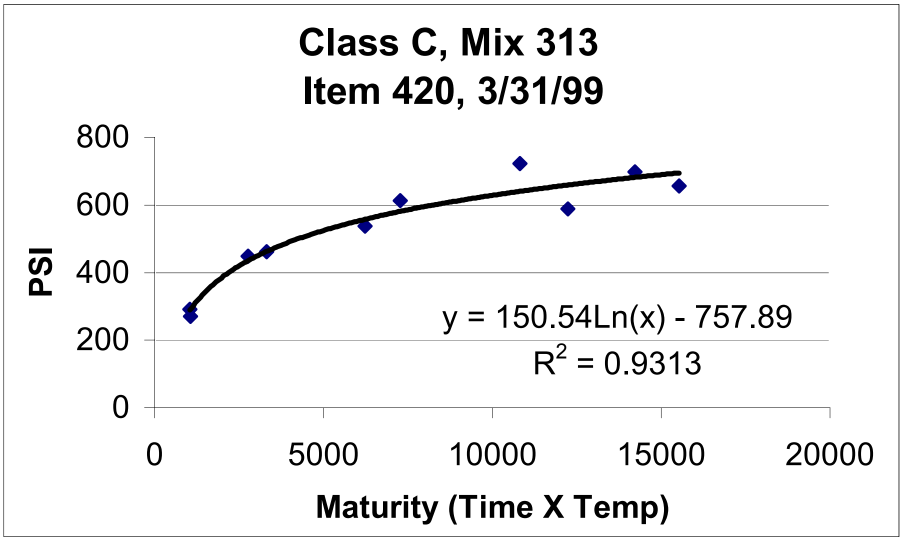

Figure 3

A maturity curve must be developed for every concrete design used on any given project. Once the curves are complete, data points representing the various strengths required for continuing construction operations can be determined. To field implement maturity testing, field personnel insert maturity thermocouples and meters, into the concrete element. Once the maturity reading indicates that the strength for continuing operations has been achieved, work may proceed. The thermocouple wire can be clipped, and the meter reused.

A more conservative use of maturity testing was implemented for structural elements at Lake Ray Hubbard. While structural elements were cast and maturity applied, two cylinders were cast as well, representing every structural concrete pour. Maturity meters were incorporated into both the structure and the specimen. When the maturity meter in the structural element indicated that maturity had been achieved, and the strength was sufficient to proceed, the cylinders were broken and the strengths compared to the corresponding maturity reading for that strength. If the breaks indicated that the maturity curve was accurate, this served as verification that the field maturity reading was also accurate, thus the strength obtained.

[7] TxDOT Manual of Testing Procedures. Section 207, Test Method Tex 426-A. Estimating Concrete Strength by the Maturity Method.

There are many advantages to using maturity testing for concrete strength. Specifically, on this project, maturity testing counteracted the lower early strengths obtained from concrete with GGBFS replacement by giving instantaneous strength data albeit slower strength gain than straight cement mixes. These instantaneous results neutralized any lost time due to low early strengths.

Maturity testing provides in place non-destructive results. Under non-maturity specifications, concrete strengths as based on either lab cured specimens or cores of the actual concrete element. Both of these methods contain some shortcomings. Concrete cylinders representing strength are usually cured under ideal temperature and moisture conditions, while the element they represent is exposed to natural weather patterns. Thus, in cold weather, test specimens could indicate a higher strength than what actually exists in the field, possibly compromising the structure. In summer, cylinders may gain strength more slowly than the actual structural element, thus slowing the construction process unnecessarily.

Although cores provide the same in-situ results as maturity testing, they are obviously less than desirable due to the expense, difficulty, and aesthetics of the process.

Maturity testing also has the indirect benefit of producing higher quality concrete. Generally, it is in the contractor's best interest to produce concrete as close to the minimum specification requirements as possible, in order to minimize cost. With maturity testing, high early strengths allow acceleration of construction, thus reducing project overhead. This is beneficial to the contractor as well as the owner.

Maturity testing allows higher confidence in strength information. Once the initial curve is established, only meter readings and verification specimens are needed. This reduces the incidence of improper cylinder breaks, and improper interpretation of the results. Further, the innate error of cylinder and beam breaks is confined to the production of the curve, and is not proliferated throughout the strength testing.

Deck Grooving

Surface texturing for the Westbound Bridge followed TxDOT standards for steel tine grooving behind the paver. Tining gives the surface a more textured finish, theoretically improving skid resistance and drainage.

Two bridge deck units on the Westbound Bridge developed multiple short longitudinal hairline cracks that were visible as soon as the curing blankets were removed. In both cases, records indicated the curing compound was applied in a timely manner, the weather conditions were ideal, and the properties of the concrete were more than sufficient. The cracks did not fall precisely over beams, or deck panels, and seemed to be more prevalent at the end of the unit that was placed later in the day, rather than at the beginning of the pour.

There was thought that the placement equipment shifting quickly and placing a load on concrete with only an initial set might have caused the cracks. But it was determined that these cracks were most likely drying shrinkage cracks. Drying shrinkage cracks are sometimes caused when the concrete suffers a loss of moisture through evaporation, thus causing uneven setting conditions, and unusual stresses to the structure. Although the concrete had been handled properly, it seemed possible that the curing compound had not covered the entire surface, and that even the minimal exposure to the air had caused some areas to dry more quickly than desirable. We believed this could be attributed to either tining done after a minimal amount of initial set, or tined grooves not receiving curing compound.

Tining of the concrete deck increases the surface area by adding the vertical tined faces, thus decreasing the overall coverage of curing compound. Further, the vertical tined faces are harder to coat with curing compound. Also, tining is the last step in the placement operations, and sometimes lags further behind than might be desirable. If the concrete had some degree of initial set, the tining could scratch through that surface set. It is reasonable to assume that uneven cure, inadequate cure or damaged initial surface set might contribute to the cracks seen on this deck.

To test whether improvements to surface finish and curing compound application could be a factor in the deck cracking, a decision was made to forego tining on the second bridge. The surface was floated to a relatively smooth finish. Curing compound was applied at the same rate as was applied to the Westbound Bridge. After the entire deck was complete and cured, the grooves were saw cut into the surface.

There are no dry shrinkage cracks on the second bridge, although statistically, this is probably not an indication that the saw cutting grooves eliminates the problem of surface cracking. The cracks on the first bridge were limited to two out of 37 deck pours. The second bridge had zero cracked decks out of 15. But, regardless of the statistics, the second bridge did not show drying shrinkage cracks.

There was concern that a smooth surface finish and saw cutting might have caused some spalling, if too much paste rose to the surface in finishing. No spalling has been observed.

The saw cut deck has a very smooth, uniform look. The sawing can only reach to within 18” of the barrier rail, thus leaving a few inches of bridge deck with a smooth finish. The width of this particular deck was sufficient to allow for the smooth edges

To groove the 18,000 square yard deck took about 33 hours with a two man crew. The added cost for this work was about $2.00 per square yard. TxDOT received $0.06 credit for tining. Clearly, the saw cutting is a more expensive method to surface texture the bridge. However, the increased durability of the deck is certainly worth the comparatively small investment.

With the implementation of the 2003 TxDOT Specifications book, TxDOT plans to specify saw cutting all bridge decks rather than tining.

Precast Caps

Early in the project, Traylor Bros and TxDOT personnel noted that 180,000 Volt transmission lines were precariously close to the existing bridge, which was approximately the footprint for the construction of the second bridge. Plan dimensions show the power lines to be as much as 150 feet clear of the construction, however, the actual location varied from 40 to 60 feet. Power lines were about 60 feet to 100 feet high. The mast arms on most of the Manitowoc cranes were 150 feet in length. Obviously, cranes on barges on rough waters were at a high risk of hitting the lines placing the safety of crews and equipment in great hazard. Many options were discussed for alternative means of construction. However, moving the power lines would have been very costly with no guarantee that the contractor would have not been delayed in spite of the relocation. Eventually, the contractor proposed an option to the State for constructing the second bridge from a temporary trestle bridge. In effect, the contractor built two bridges simultaneously – a temporary construction bridge and the permanent structure.

The trestle bridge spanned 600 feet of lake at a time, resting on piling driven to bearing. As the construction progressed, the trestle “leapfrogged” over itself, in the direction of the construction. As each operation progressed, the piling and trestle were pulled from the back and moved to the front of the bridge.

Resting on the trestle bridge were two 150 Ton Manitowoc cranes which were used to place forms, caps, beams and deck panels. Since the trestle was located between the two new bridges, the cranes were protected from the power lines by distance and the ongoing construction.

Although this method proved to be very effective for the construction and for safety, it forced every structural element of the bridge onto the critical path. The time line ran through the columns, caps, beams and deck panels. The leapfrog movement of the trestle was critical to the schedule. Thus, every item was critical.

Normally, the time frame for construction of those elements was as follows. Drill shaft forms and concrete placement were not performed from the trestle bridge. Column forms could be set and concrete placed in one day with another day or two for curing and form removal. Normally, cast in place caps took a minimum of a day to set platforms, a second day to tie steel, and a third day to set forms and pour concrete. Normally, two days were required for form removal. Beams could be set the day they arrive, assuming the cap concrete had obtained the appropriate strength. Deck panels could be set as they arrived. It is obvious to all that the cast in place caps were going to slow the movement of the trestle bridge by at least 4 days per cap. With 45 caps on the Eastbound bridge, this added about 180 days to the project.

Overhead costs for an extra 180 days of several pieces of heavy construction equipment would have been cost prohibitive to the entire project. This led to the plan to produce precast caps to provide relief from that overhead.

The Bridge Division of TxDOT quickly provided a precast cap design. All the precast caps were identical and included hollow grout tubes which could be placed over the dowels extending from the columns to provide a sheer connection.

Traylor Bros., Inc. determined that the most cost efficient method of production was to cast the caps in the construction yard themselves. The area for this operation was surprisingly small. Less than an acre of area was needed for casting, curing and storing the caps.

Traylor Bros manufactured two cap forms. Steel was tied on the ground and lifted into the forms. Concrete was poured and the forms were insulated with blankets, since most of this work was done in cold weather. Maturity meters were used to verify the required 3500 psi compressive strength in 36 hours. Unfortunately, the State and the Contractor were unable to produce a mix design utilizing the GGBFS that would meet the high early strength requirements.

After curing and strength requirements were met, the contractor loaded the caps on barges from where they were placed upon the columns by crane. Pressure grouted connections were completed, and beams were allowed to be placed as early as the next day.

The benefits of precast caps for the second bridge on this project are numerous. Obviously, the time savings on the construction schedule was very important to the contractor and the State of Texas. The contractor was able to prevent the accumulation of added overhead, and the State of Texas will be able to provide the promised end product to the taxpayers on time.

Precast caps are inherently safer to construct. The form placement, steel tying concrete placement and form removal were all done on the ground rather than over water.

Although a time savings had been anticipated when using the precast caps, the production rates for caps from start to finish was fairly close. The rate of cap production for the Westbound bridge, with cast in place caps was about 2.7 units per week with 4 sets of forms. The rate of cap production on the Eastbound bridge was about 2.0 caps per week. It should be noted that the manufacture of the caps was not the controlling cap operation. At any given point, Traylor Bros. had 4 to 10 caps stored in their casting yard.

The addition of precast caps did not provide a cost savings to the State of Texas. In all, the cost to add the precast caps was about $140,000. Had the precast caps been planned at the time of bidding, it is likely that the cost would not have been this significant, but the fact that two distinct systems with two different forming and placing systems were incorporated drove the price up somewhat. It is not unreasonable to believe that the schedule delay, had the precast caps not been incorporated would have cost significantly more than $140,000 cap change order, thus justifying the changed design.

Summary

The Lake Ray Hubbard State Highway 66 Bridge offered many opportunities to evaluate new construction concepts. Some of these innovations were added for testing and research purposes and some were added for necessity. All the innovations proved to be effective in delivering to the taxpayers of Texas a more durable, more aesthetically pleasing, safer, stronger and timely project.