REFLECTIONS
or
FICTION AND FACT FROM ED'S ALMANAC

Forty-six years ago this coming August, I reported to the Bridge Division, Tennessee Highway Department (figure 1). After a greeting, I was placed at a drafting table equipped with a back transfer rotary calculator, electrically driven (figure 2), a T-square, pencils and paper, to which I would add a Chemical Rubber Company book of trigonometric and logarithmic tables, an AISC Book of Influence line ordinates and the PCA Book of Frame Constants (figure 3).

Thus oriented, I was given a set of handwritten bridge design notes to follow and a preliminary layout of a 3-span bridge of about 50-ft spans each and told to design alternates for a continuous mild reinforced concrete t-beam bridge and a precast, prestressed concrete bridge continuous for live load. I was to follow the notes, ask questions and also design the bridge by hand. It only took me 18-weeks. Perhaps how design was done back then explains why older engineers are content with approximately correct answers, not exact answers.

Now to fully disclose, at the end, I filled out coding sheets for a continuous beam run to be sent to a key punch operator who produced a stack of IBM punch cards which I checked and returned. The next afternoon, I received the printouts of all the moments, shears and reactions. Thus began my bridge design career.

At that time I was given a copy of the AASHTO Bridge Design Specifications 9th edition, 1965. The Concrete Section was 18 pages in length and the Prestressed Section was 9 pages long.

By comparison, the 17th and final addition of the Standard Specifications, 2002 edition, contained 35 pages on Reinforced Concrete, both working Stress and Load Factor, and 25 pages for Prestressed design. The 2010 LRFD Bridge Design Specifications has 255 pages of Combined Reinforced and Prestressed/Post-tensioned Design for concrete, but assuming half is commentary, it would be 128 pages.

To say the least, life for the designer has become more complicated. Yet, amazingly, the bridges I designed, as well as those designed 90-years ago or more, stood up then and many still serve today. I've seen to it that none of mine have been replaced.

I'm standing here, not of my own volition, but rather because Maher Tadros asked me if I would talk about what I've learned in 46 years. The answer is "not much". Especially, I've not learned to keep my mouth shut (figure 4). On my first report card in first grade, my astute teacher sized up my entire life. In the "Remarks" column next to my "F" in "Conduct" appeared the words "Eddie continually demands more than his share of attention". And like Dilbert, I can't resist telling the truth; truth as I know it, anyway.

But back to my assignment, I want to offer a few thoughts; truth as I know it.

Collaborate/Volunteer

Seek out and cultivate relationships with those who are more informed and have more expertise, then ally yourself with them. Examples are:

- PCI Bridge Committee
- AISI Bridge Task Force
- NCHRP Committee Membership
- AASHTO Technical Committees

The real satisfaction in engineering is helping solve problems in, and improving the practice of design and construction. For some with superior intellect, the goal can be achieved alone. For most of us, however, the satisfaction of contributing to advancement in engineering is not found in self aggrandizement but in being part of significant achievement.

Don’t Let Design Specifications or Lack of Research Paralyze Innovation

Too often engineers fail to embolden themselves to act upon new ideas and practices because current specifications do not cover or expressly permit what are otherwise intuitively sound ideas. Likewise, action on sound ideas is often postponed awaiting validation from exhaustive research. Don’t be reluctant to try new ideas or to push the limits because of a lack of proof. Successful practice is its own proof. Some examples in my life:

- Simple Span for Dead Loads, Continuous for Live Loads (prestressed, steel) - During the Illinois Road Test Program in the 1950’s, several prestressed bridges were designed and constructed with concrete diaphragms at intermediate supports. The composite slabs were poured continuous over the supports, containing nominal crack control reinforcement. The beams were designed as simple spans, however. The Bridge Division of the Tennessee Highway Department noted the good performance of the test bridges, and saw an opportunity to gain more structural effectiveness by designing their prestressed beams as continuous for Live Loads and Composite Dead Loads taking advantage of the continuity. Not waiting for any research to confirm their ideas, nor specifications to be in-place in order to sanction their theories, the Department set out to design and construct the dual bridges carrying I-40 over the Big Sandy River (figure 5). The move was not half-hearted as the bridges were 700-ft in length and would be subjected to heavy traffic. Constructed in 1963, the bridges served as the prototype for thousands of similar designs to follow. The longest such structural system designed by the Department is the Long Island Bridge in Kingsport, Tennessee (figure 6). Constructed in 1980, the dual bridges are 2,700-ft in length with joints at the abutments only. Funding was provided for the Long Island Bridge to be instrumented to measure the performance of the bridge as a continuous for composite load structure and assess the thermal affects over time. These tests verified the design theory as well as the limited impact of thermally induced stresses.

In a similar vein, the Structures Division started experimental development of simple span for dead load, continuous for composite load construction of steel deck girder bridges. Again, without benefit of lengthy and costly research, or awaiting specification guidance we designed our first steel rolled I-beam bridge constructed as simple span for non-composite loads and continuous for composite loads, in Maryville, Tennessee (figure 7). From that learning experience, we've forged ahead to extend the range of spans for steel beams by utilizing essentially prismatic beams sized based on the demands of simple span for beam weight and continuous for all other non-composite and composite loads in the maximum positive moment area of a span and providing the additional moment capacity at interior supports via beefed up compression flanges and bolted top flange splice plates, extended to serve as cover plates as well. The compression flange force transfer is provided by driven wedge plates, while cast-in-place concrete diaphragms replace web plates (figure 8). This reduces the number and complexities of field splices, provides safer erection over and minimizes delays of traffic.

- Integral Abutments/Jointless Bridge Development –
  At the same time the Department began experimenting with continuous prestressed concrete beam construction another area of experimentation was initiated; jointless bridges with integral abutments. The first bridge was built in 1963, a 3-span cast-in-place t-beam bridge 168-ft. in length (figures 9 and 10). Thus began a 48-year practical research experiment culminating in our longest jointless concrete bridge, State Route 50/Happy Hollow Creek, 2000 Harry Edwards Award Winner from PCI (figure 11). The bridge is 46-ft. wide and 1,175-ft. in length.

  Through the years, Tennessee has steadily pushed the boundaries of jointless construction for concrete and steel bridges. This work was always "seat-of-the-pants" exploration. Interestingly, it was only in 1996 that a design procedure was formalized. The American Iron and Steel Institute requested Tennessee to prepare a chapter for their Highway Design Handbook. Reluctantly, we accepted the challenge, and after several discussions on how we would design an integral abutment if we were of a mind to do so, we set to work. There had been a number of papers by other researchers but their conclusions and limitation on design always fell short of our experiences. Several years after publication of our recommended procedures, we had occasion to fund full scale field testing of integral abutments supported on both steel and prestressed concrete piling. The test performed by the University of Tennessee confirmed the design methods we had published earlier (figure 12). The report can be found at www.tdot.state.tn.us/longrangereports.htm.

- Implementation of the Use of High Performance Steels In Highway Bridges – Starting in 1992, a consortium that included FHWA, the American Iron and Steel Institute and the US Navy, had been working to produce new steel grades in 70 and 100 ksi strengths that would exhibit greater toughness to resist fracture growth. The primary aim was directed toward producing these steels for use in submarines for the Navy. However, recognizing the need to expand the use of such steels in other markets to reduce production costs, it was hoped that such steels would be adaptable to the highway bridge market. Having reached the production stage, the consortium was experiencing difficulty in finding an agency to design a bridge utilizing the New High Performance Steels. Tennessee volunteered to design and construct the first bridge specifically taking advantage of the HPS-70W properties, using the Load and Resistance Factor Bridge Design Specifications (figure 13). Further, funds obtained from an Innovative Construction Grant awarded to the Tennessee DOT were turned over to the Turner-Fairbank Laboratories for th purpose of documenting the fabrication process, particularly the welding applications. The first such bridge, State Route 53 over Martin's Creek was completed in 1998. Lessons learned from the fabrication of this bridge along with that of a second Tennessee bridge that utilized hybrid girders of ASTM A572 grade 50 webs and HPS 70W flanges (figure 14), formed the basis for the AASHTO Guide Specifications for the Fabrication of Highway Bridges with HPS-70W Steel. The efforts by Tennessee contributed materially to the consortium being awarded the Charles Pankow Award for Innovation presented by the Civil Engineering Research Foundation.

- Development of Design for Integral Concrete Bent Caps for Steel Bridges – In 1977, the Department of Transportation was busily engaged in designing the re-construction of the Interstate System in Knoxville for the 1980 World's Fair. Ray Whitaker of Wilbur Smith came to the Division of Structures with a proposal that would eliminate some difficult skew conditions as well as avoid adverse roadway grades at a three level interchange. The proposal was to design selected intermediate substructures using integral post-tensioned cast-in-place bent caps for some horizontally curved steel bridges. The principles to be employed would be similar to those used to design integral bent caps employed in cast-in-place hollow box girder bridges. Heretofore, such design methods had not been used, particularly addressing issues dealing with girder reaction load transfer and torsion under live load. However our confidence in Mr. Whitaker and the appeal of potential gain in knowledge of how to further exploit the concept convinced us to allow the designs to proceed. No long drawn out research was proposed as proof of concept nor did we await specifications modifications to sanction the application. Time was precious and an opportunity was at hand. The methods of design and construction were a success and gave us a new tool in our bridge design box that has been used many times hence (figures 15, 16 and 17).

Don't fear to fail, for as much is learned by mistake as is learned by success. If after thoughtful consideration one thinks an innovation is workable, go ahead.

<u>Support Research to Improve Understanding but Don't Be Obsessed With a Quest for Accuracy, Nor Fail to be Wary of Results</u>

I give you two examples:

1. Stress Loss in Prestressing Strands -
   The first estimate of prestress losses appeared in the 1961 8th edition of the AASHO Standard Specifications for the Design of Highway Bridges based on the work of Kent Preston. Losses were identified for ASTM 250 ksi 7-wire strand as 35 ksi. In the 1969 10th edition, a more refined method was introduced. A fixed reduction for stress losses was eliminated; however, the fixed reduction was re-introduced in the 1975 Interims to the 11th edition as 45,000. The latter was the average value obtained using the refined method, and both the refined and standard loses remained constant through the 17th and final editions of the Standard Specifications (figure 18).

   The NCHRP 12-33 final draft of the AASHTO LRFD Bridge Design Specifications through the 3rd edition (2004) had a different method for time dependent loss calculations but allowed standard loss values between 30 and 33 ksi. However in the 2005 Interims to the 3rd edition of the LRFD Specifications, a new set of time dependent loss calculations based on the NCHRP Report 496, authored by Tadros, Gaalt and others. The standard loss deductions remained at 30-33 ksi. Finally, the 2010 4th edition eliminated fixed loss values and substituted a "simplified" set of equations for a less refined estimate.

   Lost with the introduction of the 2005 Interims was an important message from the commentary to the section on "Loss of Prestress". That commentary stated "…..undue refinement is seldom warranted or even possible at the design stage since many of the component factors are either unknown or beyond the control of the designer".

   Let's apply the original 1961 suggestion and compare its affect on one of our most efficient beams, a BT 72 vs. the affect of the latest suggested time dependent calculations from CONSPAN. Let's presume 40 strands (figure 19).

   With the1961 suggested value of 35,000, we get a 17.28 percent loss of stress from the initial stress in the strand.

   With the improved, more precise calculations, the loss is between 20 and 24 ksi or about 11 percent.

   The net gain is 6.28 percent which equates to 2 to 3 less strand.

From one of our producers, the cost to install two 0.6 in diameter strands is \$0.50/ft for strands and \$1.25 for labor or \$1.75 per foot.

The average price of a BT-72 is \$162/LF, so the savings by using the time dependent method is 1.08 percent, for two (2) strands and 1.4% for three (3). Further, the time dependent losses are based on assumptions over which the designer has no control. So have we made any significant progress in 50-years with the research done?

2. Development Length of Strand –

In 1993 a research report entitled “An Investigation of Shear Strength of AASHTO Type II Girders” was published by the Structures Research Center of the Florida DOT. The Report concluded that debonding more than 25 percent of the strands in a beam could lead to a significant reduction in the shear capacity at the end of the beam. Though not specifically addressed by the AASHTO T-10 Committee, the Report recommendations were adopted into the first edition of the AASHTO LRFD Bridge Design Specifications, 1994. Interestingly, PCA research was published under PCA R & D Serial No. 1171 in 1963, entitled “Effect of Strand Blanketing on Performance of Pre-tensioned Girders”. The research by Karr and Magura was instituted at the request of the Bridge Division of the Tennessee Highway Department, as a means of reducing the pre-compression in box beam ends at interior supports of continuous bridges. The research used ½-scaled AASHTO Type III beams that ranged from no blanketing to up to 83 percent blanketing (figure 20). Test included samples to failure in bending and tests to failure of under-reinforced beams in shear. The conclusion of that research was that blanketing had no affect on the shear capacity of the beams.

Which study is correct? Perhaps both. The LRFD points out that many states have successfully designed beams with considerably more blanketing than 25 percent. There are four considerations associated with the decision to blanket strands:

- Reduce the stress at selected beam locations to the allowable at the Service Limit State.
- Provide the Moment Capacity required at the Service and Strength Limit States.
- Provide the Shear Capacity required in the end regions of a beam at the Service Limit State.
- Prevent splitting of the bottom flange in the end regions of a beam due to the Hoyer Effect.

Providing the required Moment Capacity at the Limit State at required locations and providing the Service and Strength Limit Stresses at key locations sufficient for loads but not exceeding limit requirements, is in part a function of the embedment or development length of the prestressing strands, l_d. In the 1961 edition of the AASHTO Bridge Design specifications, l_d was a set length, 135-inches for ½-diameter ASTM 250k or 270k grade strand. In the 1965 edition of the AASHTO Standard Specifications for Highway Bridges, the equation, $l_d = \left(f_s^* - \frac{2}{3} f_{se}\right) d,$ was imported from the ACI 318-63 Building Specifications. This equation remained unchanged until the 1989 edition of the Bridge Specifications, when a "k-factor" was introduced at the insistence of the Federal Highway Administration. The action was precipitated by a series of events. In the early 1980's, Dr. Paul Zia of the University of North Carolina was studying the effects of epoxy coated prestressing strands on beams. As a control, one or more beams with bare strands were tested. It was observed that prior to reaching Ultimate Moment Capacity, the bare strands experienced slip. The report from Dr. Zia's work concerned FHWA and PCI, as well s the States. There followed a concerted research effort by the FHWA Turner-Fairbank Laboratories and at several universities, some of the research at the latter funded in part by PCI. At the end of the testing it was found results were not uniform and that in many cases, strand was found to be "contaminated" with oil from the drawing process in manufacture. Dr. Dale Buckner was brought in to assess all the research and make recommendations. The result was the introduction of the 1.6 k-factor.

Interestingly, the work of Karr and Magura, discussed earlier, had observed slip of strand. To quote "There is evidence that the 1963 ACI Building Code) ACI 318-63) requirement for bond embedment length of strand in section 2611 cannot be directly applied to blanketed strand. However, the performance in the exploratory tests of blanketed strand girders with embedment lengths twice those required by Section 2611 closely matched the flexural performance of a similar pretensioned girder entirely without blankets." Perhaps the occurrence of slippage was coincidence, but perhaps not.

It is of interest that if one calculated l_d by the current AASHTO equation and used 0.7 f_{pu} for 270 ksi strand as equal to f_{ps} and took 2/3 of the 35 ksi losses suggested in the 1961 Specifications, l_d would equal 132.53-inches; practically the same length suggested in 1961, 135-inches.

The commentary to ACI 318 on development of prestressing Strand emphasizes that the tests used to establish the l_d equation "may not be representative of the behavior of strand in low water-cementitious ratio materials, that clean strand and controlled strand release were important.

What do these examples of conflicting conclusions from research tell us? Perhaps it tells us that research programs should start by examining the realities of how the products, prestressed beams in this case, are mass produced on a daily basis, from manufacture, handling, storage and assembly. The research at least should mimic the current processes in devising test procedures from which to derive results and recommendations. Certainly other controlled procedures can be investigated to demonstrate that improved means and methods can lead to enhanced product quality. However the market place will determine the worth of tighter controls. In the final analysis, it is important to remember that engineering is not exact science, it is empirical science. Perhaps the researchers who determined the development length of strand in 1961 used strand not thoroughly cleaned of oil from the drawing of wire and concrete mixed that had higher w/c ratios. They were looking at conditions as they were to document the process, not specifically to improve the process. Perhaps the research upon which the ACI formula was based were looking to demonstrate a gain in efficiency by using an improved quality control process, but failed to emphasize the importance of having the greater controls in place. Perhaps the two research projects on the affects of debonding and shear created different circumstances of support conditions that, by chance, changed the outcome of the results.

Complexities of Design Solutions

I am concerned that researchers tend to express results in complex analysis solutions when more simple and intuitive approximations can suffice. The best example of this in Concrete Design is the Modified Compression Field Sectional Analysis Procedures for Concrete Shear Design. It is accurate but far from intuitive whereas the Vci, Vcw Method is perhaps less accurate, but more intuitive and actually provides a greater margin of safety and could be assigned a higher Phi factor. If you recall, Dennis Mertz and Dan Kuchma reported the Phi factor could be raised from 0.90 to 0.95.

Summary

What do these ramblings suggest?

- First it is important to participate in Technical Committee work to learn from others, share your knowledge and to jointly achieve meaningful progress toward solving common problems in design. I've noticed that tangential discussions at these meetings reveal that many perceived problems by one have already been solved by another or others.
- Engineers need more encouragement to be bold in trying new ideas without waiting for research or specification changes to validate innovation. Build on what concepts you've proven and push the envelope from time-to-time.
- Support of meaningful research to solve problems is a key component of advancing engineering knowledge. However, it is a waste of time and money to keep re-visiting the same issues every 5 years or so for the sole purpose of supporting research that has no expectation of significant gain. Discuss carefully the potential significant gains in scientific knowledge, improvements in design or monetary payoff in research outcomes. Also be sure the research is grounded in real world conditions.
- Lastly, I think we should all strive for design and analysis methods that can be boiled down to be more transparent, intuitive and simplified solutions for designers. Adequately approximate equations yielding safe results should replace complex solutions. The more elegant solutions appeal to researchers, but not to those who have to use them on a daily basis. True, complicated

methods can be programmed to solve complex equations, but designers who use the programs lose sight of or never grasp what fundamental problem they are solving.

In closing, allow me to say that serving on the PCI Bridge Committee, AASHTO T-10 and a host of other committees with you and others has been one of the most rewarding parts of my career. Thank you for your help and friendship along the way. Hopefully, I can find a way to continue my relationship with you all. Best wishes for your continued successes.

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