Letters

Run test product when using inexperienced drafter

I enjoyed the article in the *PCI Journal* Winter 2010 issue starting on page 139, "Use Lean Principles to Improve Drafting Quality," by Mike Strain.

This article had many good points. The one item I would add is to run a test widget if you have a large order to produce.

When we start a new product line or have an inexperienced draftsperson drawing a product line that is new to him or her, we have the draftsperson do a typical ticket or elevation. We do a complete drafting, engineering, and sometimes production check on this typical ticket, generally with many markups required. The draftsperson quickly learns what is expected and required without having the same 10 markups on 100 or more tickets. While we do a 100% check of all shop drawings, seldom does the draw-check cycle have to be repeated.

Thank you for addressing a subject seldom written about in *PCI Journal*.

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Author's response

These are good suggestions from Retzner on effective drafting practices.

We notice, in our lean transformation projects with a variety of producer engineering departments and engineering consultants across the country, that a good opportunity exists to develop and share industrywide best practices. As engineering and drafting teams adopt lean methods, they understand that standardized best practices for critical processes are necessary to achieve their goals to improve quality, cost, and speed. Production departments across the industry, through PCI's annual productivity tour and other events, have found mechanisms to share best practices.

Perhaps in the future similar forums will be available for engineering departments to exchange and advance best practices.

Mike Strain
President, MJS Management Services

Clarification requested on seating wedge use

This letter comments on "Behavior of Unbonded Post-tensioning Monostrand Anchorage Systems under Monotonic Tensile Testing" on pages 97 through 117 in the Winter 2010 issue of *PCI Journal*.

Regarding the seating of the wedges the article states, "Prior to seating, the anchor wedges were tapped in lightly by hand using the back of a spare anchor barrel before a preload of about 750 lb (3340 N) was applied to the strand."

It is my experience that, in current (field) construction practice, stressing uses jacks that incorporate the mechanism for seating wedges at the stressing end as part of the release when the pre-

scribed jacking force is reached. Most often, the strands are shipped with the anchors and wedges preinstalled at the dead end.

Also, it has been my experience that broken strands almost always are a result of uneven wedge seating. I would appreciate the authors' comments comparing actual field practice versus their laboratory testing procedures.

Furthermore, the article states that "the wedges were assumed to seat about 0.5 in. (13 mm) in each of the top and bottom anchors." It is my understanding that wedge seating loss is normally assumed to be 0.25 in. (6.4 mm).

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Authors' response

The authors thank the reader for taking the time to read the article and sending his comments on the seating of anchors using a post-tensioning jack and on his experience regarding broken strands with uneven seating. Both of these important issues are addressed in a second paper (titled "Effects of Loading Parameters on the Behavior of Unbonded Post-tensioning Strand/Anchorage Systems in Seismic Regions") accepted for publication in *PCI Journal* as well as in the final project report submitted to PCI in April 2009, *Behavior and Design of Unbonded Post-tensioning Strand/Anchorage Systems for Seismic Applications*.[1]

As stated, the primary objective of the initial paper was to experimentally investigate the differences among strand, anchor, and wedge configurations tested in conformance with the static testing requirements of the International Code Council Evaluation Service (ICC-ES) *Acceptance Criteria for Post-Tensioning Anchorages and Couplers of Prestressed Concrete*.[2] The ICC-ES testing procedure does not require the use of a prestressing jack to seat the anchors, and thus none of the tests described in the current paper used a jack to seat the anchors.

However, as investigated in the final project report (and in the second paper), the difference between prestressing with a jack compared with simply allowing the movement of the testing machine crosshead to stress the strand is twofold. First, the jack creates a live-end effect at one of the anchors by reducing the diameter of the strand due to the tightening of the strand wires before power seating the wedges, possibly resulting in deeper wedge seating at one end. Second, prestressing with a hydraulic jack occurs at a faster strain rate than is commonly achieved by the movement of the testing machine crosshead.

To examine these factors, additional tests were conducted by applying a prestress to each strand using a hydraulic jack with a power seater at one end before continuing to load the strand to failure with the testing machine. A small reduction in the average fracture strain of the total data sample pool occurred in the tests with the prestressing jack compared with the control procedure described in the first article. This reduction was mostly governed by the greatly reduced fracture strains for one of the six strand/anchor/wedge configurations tested using a prestressing jack. The second paper and the final project report not only expound upon these results but also quantify the effects of several other important loading parameters.

Regarding the reader's comment on anchors with uneven wedge seating, all of the test results presented in the initial paper had proper seating between the anchor wedges. Any test with more than 0.10 in. (2.5 mm) of differential wedge seating was discarded from the main test-data pool but was recorded for the purposes of a follow-up investigation. Through this follow-up investigation, which is also included in the second paper and the final project report, it was demonstrated how poor wedge seating may be associated with low fracture strains (as low as 0.01 in./in.) but that low fracture strains are not entirely dependent on poor wedge seating. At least 100 specimens had differential wedge seating indiscernible to the naked eye but fractured at very small strand strains.

Thus, contrary to the reader's experience and as shown in the initial paper, broken strands are not necessarily a result of uneven wedge seating, even though uneven wedge seating does seem to increase the likelihood of the strand wires' fracturing at low strains. Preinstalled anchors would significantly reduce the likelihood of differential wedge seating at the dead end, but this does not necessarily eliminate the possibility of an early wire fracture at either end of a strand.

With regard to the reader's comment on the assumed wedge seating of 0.5 in. (13 mm), 13 different anchors ranging in height from 1.31 in. to 3.11 in. (33.3 mm to 79.0 mm), and 15 different wedges ranging in height from 1.17 in. to 1.97 in. (29.7 mm to 50.0 mm) were tested in this research project. A range of wedge seating depths—some lower than 0.25 in. (6.4 mm) and some higher than 0.5 in. (13 mm)—were observed, and the assumed value of 0.5 in. was deemed to be as acceptable as any other similar value. It should be emphasized that the strand strains were measured using two extensometers placed within the free length of each strand, and the readings from these extensometers were not affected by the assumed wedge seating depth. The only effect of the assumed seating depth was on the strain rate applied to the test strands, which was controlled by the relative position of the two crossheads of the testing machine. Because the initial free length of all test strands was at least 72 in. (1800 mm), it is unlikely that a difference of 0.25 in. (6.4 mm) in wedge seating would have noticeably affected the outcome of any test.

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Predictive equations OK for high-strength concrete

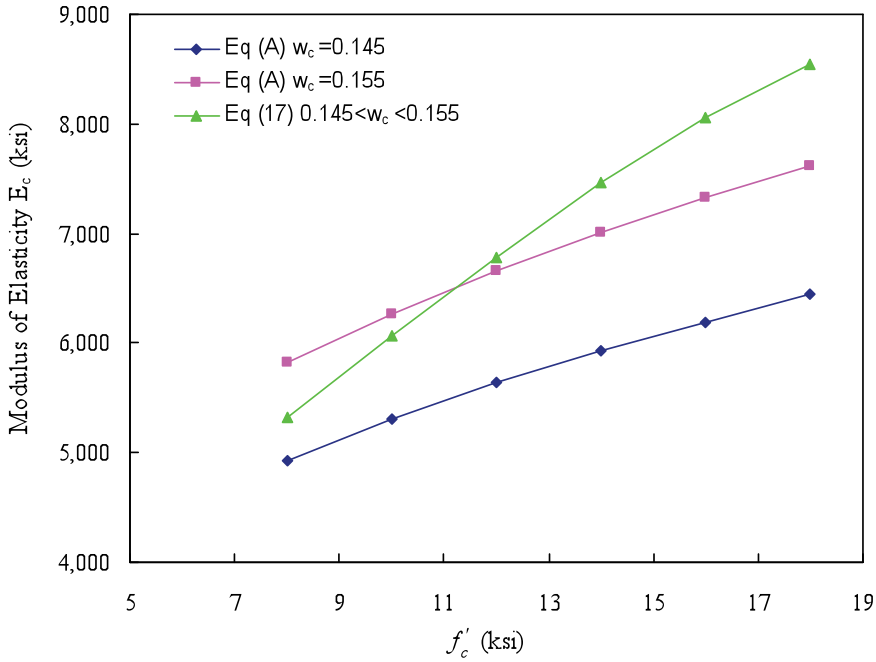
In "Elastic Modulus, Shrinkage, and Creep of High-Strength Concrete as Adopted by AASHTO" on pages 44 through 63 in the Summer 2009 issue of *PCI Journal*, the authors have made an important contribution by refining the predictive equations for modulus of elasticity, shrinkage, and creep of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] for applications of high-strength concrete up to 15 ksi (100 MPa). In the past, designers have often used codified estimates of these properties and found them to be higher than values reported or observed from the field.

The discussers also conducted another research project, National Cooperative Highway Research Project (NCHRP) no. 12-64, to extend the application of flexure and compression provisions of the AASHTO LRFD specifications[3] to high-strength structural concrete up to 18 ksi (120 MPa).[4] The following discussion addresses several aspects of the authors' recommended predictive equations:

1. Regarding the equation for modulus of elasticity, the authors made no changes to the basic form of the equation except for introducing a factor K_1 to account for the effect of aggregate stiffness and an auxiliary equation for w_c to relate its value to the specified concrete strength f'_c. Based on their test data combined with test results reported in the literature (more than 4400 data points), the discussers proposed an alternative equation as follows:

$$E_c = 310{,}000\, K_1 (w_c)^{2.5} (f'_c)^{0.33} \quad \text{(A)}$$

A comparison of Eq. (A) with the authors' Eq. (17), taking $K_1 = 1$, is shown in the following figure:



It is clear that the authors' Eq. (17) gives an increasingly higher estimate of E_c than the discussers' Eq. (A), especially for concrete strength greater than 12 ksi. Such a trend does not seem realistic in the face of available test data.

2. It is an improvement by introducing K_1 to account for the effect of aggregate stiffness on the modulus of elasticity. Given that creep and shrinkage are known to be affected by aggregate type, such effects could also be accounted for by introducing adjustment factors (such as K_2 and K_3) in the predictive equations for creep coefficient and shrinkage strain, provided that sufficient research data are available to establish these factors. Such an approach may be particularly useful for precast, prestressed concrete producers and state departments of transportation when certain types of aggregates are regularly used. Using such adjustment factors would provide better predictions for modulus of elasticity, creep coefficient, and shrinkage strain for design.

3. For predictions of both shrinkage and creep, the authors introduced a new time-development correction factor as

$$k_{td} = \frac{t}{61 - 4f'_{ci} + t} \tag{24}$$

where

t = age of concrete between time of loading for creep calculations or end of curing for shrinkage calculations and time being considered for analysis of creep or shrinkage effects

The authors also suggested that Eq. (24) should not be used for concrete strength f'_{ci} at release of prestress in excess of 12 ksi (83 MPa) or at service in excess of 15 ksi (103 MPa). Eq. (24) produces erroneous values for small values of t if the concrete strength f'_{ci} is greater than 15 ksi (103 MPa).

For example, if f'_{ci} is 16 ksi (110 MPa), the value of k_{td} would be negative for t less than three days. For t equal to three days, the value of k_{td} would become infinity. In terms of design, although concrete compressive strength of 16 ksi (110 MPa) is unlikely to be used as transfer strength for pretensioned concrete members, it is possible that the strength could be achieved at the time of loading for cast-in-place concrete columns or for post-tensioned concrete members.

To alleviate this mathematical anomaly, the discussers proposed the following equation as a replacement of Eq. (24):

$$k_{td} = \frac{t}{12\left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20}\right) + t} \qquad \text{(B)}$$

A comparison of Eq. (B) with Eq. (24) is shown in the following table:

Time *t*, days	k_{dt}								
	Eq. (B)						Eq. (24)		
	f'_{ci}, ksi						f'_{ci}, ksi		
	8	10	12	14	16	18	8	10	12
1	0.033	0.040	0.049	0.060	0.077	0.102	0.033	0.045	0.071
7	0.194	0.226	0.264	0.311	0.368	0.442	0.194	0.250	0.350
14	0.325	0.368	0.418	0.474	0.538	0.613	0.326	0.400	0.519
28	0.490	0.538	0.589	0.643	0.700	0.760	0.491	0.571	0.683
90	0.755	0.789	0.822	0.853	0.882	0.911	0.756	0.811	0.874
365	0.926	0.938	0.949	0.959	0.968	0.976	0.926	0.946	0.966

Note: 1 ksi = 6.895 MPa.

The table indicates that Eq. (24) provides slightly greater values of ktd than Eq. (B) provides, but only Eq. (B) can be applied when the concrete strength f☞c i exceeds 15 ksi (103 MPa).

4. The authors compared the experimental results of shrinkage and creep with three different prediction methods (Fig. 11 and 12 in the article) and made it clear that the methods prescribed by the pre-2005 AASHTO LRFD specifications[3] and American Concrete Institute (ACI) 209[5] greatly overestimate the amount of shrinkage and creep. The methods proposed by the authors provide a much closer estimate for both shrinkage and creep. However, upon close examination of the authors' own test data contained in NCHRP report 496,[6] it is observed that the predictive equations proposed by the authors also overestimated their own test data by a substantial margin. A reason explaining why the equations could not provide closer predictions for their own more controlled data would be helpful.

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Authors' response

The authors appreciate the interest and the thoughtful comments by Zia, Rizkalla, Mirmiran, Russell, and Mast. We would like to offer the following responses to the comments in the order they were listed.

Before we do so, we would like to make a general statement. The NCHRP project[6] on which the article was based had certain specified limitations. Among them was the edict to limit the design strength of the concrete in service to 12,000 psi (83 MPa). Also, we were expected to extend the existing AASHTO LRFD specifications[3] loss prediction methods to concrete strengths beyond the 5000 psi (34 MPa) level for which they were developed.

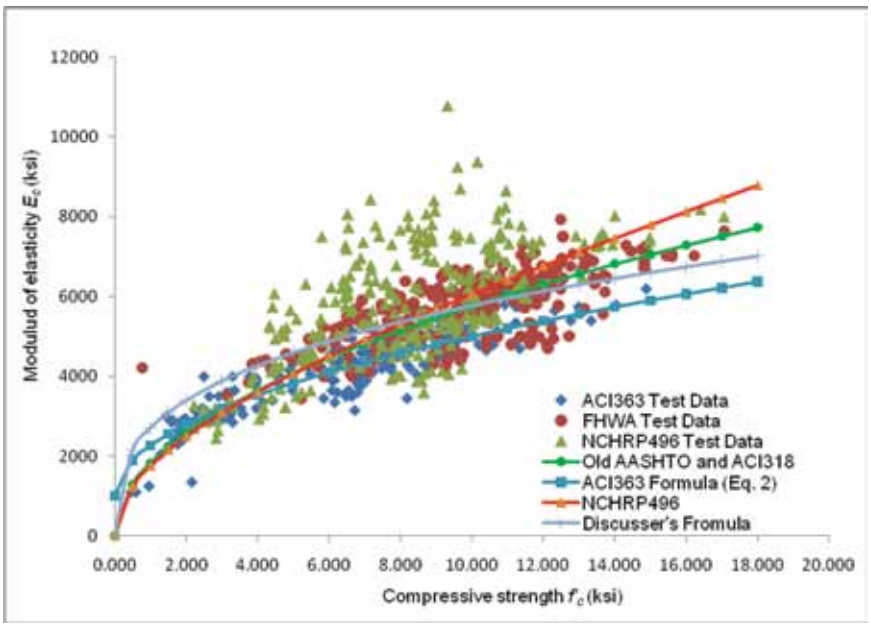
Consequently, one of our goals was to have the estimated losses default to the values predicted by the existing methods at lower levels of concrete strength. The concrete strength at prestress release was assumed to be on the order of 80% of the final strength, so for 12,000 psi (83 MPa) concrete, the maximum concrete release strength was expected to be about 9600 psi (66 MPa).

While our project was being conducted, the project referred to by the discussers had not been started. The specified concrete strength ranges were different for the two projects, and the contributions by the discussers for concrete strengths in the range of 12,000 psi to 18,000 psi (83 MPa to 124 MPa)are expected to be valuable when such values become commonplace in future practice.

1. The discussers' chart shows no test data. It is difficult to evaluate their conclusion based only on a plot of Eq. (17) versus their Eq. (A). It should be observed, however, that their plot shows the impact of density of concrete on the modulus of elasticity. When the discussers use a density equal to 155 lb/ft^3 (24 kN/m^3), the two prediction methods are very close for the concrete strength range used in the authors' study. Due to the lack of information on higher-strength concretes in our study, it would be helpful if the discussers provided more persuasive experimental justification for their proposed equation. As a side note, ACI committees 318 and 363 have struggled for many years to justify changing the power of the term f'_c from 0.5 to another power, as is seemingly adopted in some other countries. So far there has been resistance to change the power from 0.5 as given in the authors' equation. The following figure compares predicted values of modulus of elasticity of concrete with test data reported in the literature and in NCHRP report 496,[6] on which this paper was based. The plot confirms that the discussers' proposed formula overestimates E_c for f'_c less than 8 ksi (55 MPa). The authors' formula appears to overcorrect for the effect of concrete unit weight for the very-high-strength concrete, which was beyond the scope of the authors' study. It is interesting to note that the existing ACI 318 formula,[7] which is the same as the previous AASHTO equation, may still give reasonable averages for the very-high-strength concrete.
2. The authors agree that aggregate type and proportion in concrete mixtures affects creep and shrinkage coefficients. Even though two factors were originally recommended for the E_c formula, AASHTO elected to retain only K_1 for coarse aggregate type. To our knowledge, however, no state department of transportation (DOT) has opted, in the past five years since the authors' method was introduced, to do their own experiments to quantify K_1 for their sources of coarse aggregates. This is understandable because of the cost-to-benefit ratio of such experimental work, and the DOTs' inability to limit aggregate usage to pits included in the testing. As it is,

E_c is highly variable and its most visible impact is on camber, which can vary from the average by plus or minus 25% in real-world applications. Even if the DOTs tried to establish K_1 values, each state must conduct studies for all potential sources of aggregates and keep it up to date. The same argument applies to creep and shrinkage, which are even more variable than E_c.

3. As previously stated, the time-development equation was not intended for service concrete strength beyond 12,000 psi (83 MPa) or release strength greater than 9600 psi (66 MPa). In subsequent AASHTO Subcommittee T-10 deliberations, of which Russell was a participant, it was decided to extend the prestress loss provisions to service concrete strengths of up to 15,000 psi (103 MPa). Based on these limitations, it is difficult to accept the assertions by the discussers that Eq. (24) is erroneous or to confirm that Eq. (B) is better. It is simply beyond the scope of our work. In addition, the authors know of no precast concrete producers who realistically expect to manufacture members with a concrete strength at prestress transfer of 16,000 psi (110 MPa), nor do we have experience with three-day-old cast-in-place columns or post-tensioned members with a strength of 16,000 psi. These scenarios are not ordinary by any means and would, presumably, require special attention to all properties of the relevant concrete mixtures. Thus, we cannot endorse either our formula or the discussers' formula for such cases. We would just point out that Eq. (24) does produce implausible answers when the concrete strength reaches a certain level at an early age.
4. Please refer to Table 4 and Fig. 11 and 12 of the paper for a response to the discussers' comment. Note the high variability of creep and shrinkage. Different sites and sources produced significantly different results. However, when test data from the NCHRP 496[6] study and data gathered from the literature such as the Federal Highway Administration showcase projects[8] were examined, the overall average was close to unity. Table 4 illustrates that the NCHRP 496 methods produce predicted-to-measured ratios of 1.05 and 0.98 for shrinkage strain and creep coefficient, respectively. Figures 11 and 12 show that the proposed method gave better correlation with test results than did previously published methods.



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Comments?

The editors welcome discussion of *PCI Journal* content. Comments must be confined to the scope the article under discussion. All discussion of articles in this issue must be received by July 1, 2010.

Please address reader comments to *PCI Journal* editor-in-chief Emily Lorenz at elorenz@pci.org.

Letters should include the writer's name, title, company, city, and email address or phone number and may be sent to the respective authors for comment. All letters become the property of *PCI Journal* and may be edited for space, clarity, and style.

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Errata

In Fig. 15 on page 130 of the paper "Erecting Long, Vertical Precast Concrete Members with One Crane and Two Operating Lines" in the Winter 2010 issue of *PCI Journal*, some notation within two equations at the bottom of the figure is incorrect.

Instead of

$$R_{R\Delta} = W * e_{\Delta}/(CG(L)_{\Delta} + e_{\Delta})$$

$$R_{L\Delta} = W * CG(L)_{\Delta}/(CG(L)_{\Delta} + e_{\Delta})$$

the equations should read as follows:

$$R_{R\Delta} = W * e_{\Delta}/(CG(R)_{\Delta} + e_{\Delta})$$

$$R_{L\Delta} = W * CG(R)_{\Delta}/(CG(R)_{\Delta} + e_{\Delta})$$