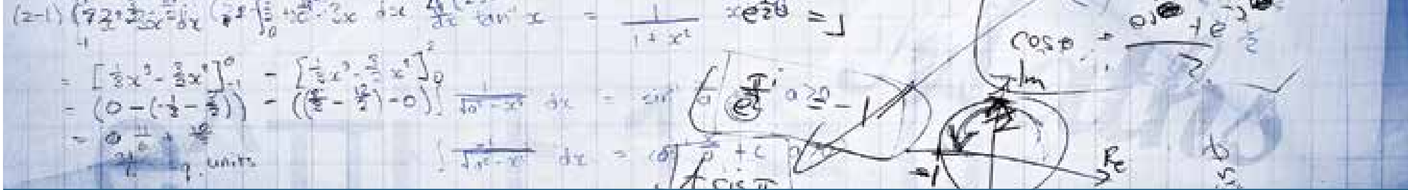

Discussion

AASHTO LRFD Bridge Design Specifications provisions for loss of prestress

I would like to compliment Brian D. Swartz, Andrew Scanlon, and Andrea J. Schokker on their exceptional paper titled "*AASHTO LRFD Bridge Design Specifications Provisions* for Loss of Prestress" in the Fall 2012 issue of *PCI Journal*. This is a resource that engineers will refer to for years to come.

The American Association of State Highway and Transportation Officials (AASHTO) T-10 committee on concrete is beginning the process of reorganizing and clarifying the requirements in section 5 of the *AASHTO LRFD Bridge Design Specifications*.[1] I would be curious to know specifically how the authors would modify the AASHTO LRFD specifications to clarify the prestress loss and elastic gain provisions.

The following statement in the Elastic Losses and Gains section of the paper seems to contradict the calculation procedure used in example 9.1b of the *PCI Bridge Design Manual*:[2] "Therefore, it would be a gross error to assume that the prestress gains due to the application of external loads or deck shrinkage act to further precompress the surrounding concrete. Such an error is likely to result if the true effective prestress force, found by summation of all losses and gains, is used to calculate concrete stress by a traditional combined stress formulation." In example 9.1b, a combined stress analysis is performed using gross section properties. The effective prestress used to compute the final concrete tensile and compressive stresses includes an allowance for elastic gains. The concrete stresses compare favorably with those computed in example 9.1a, which performs the stress analysis with transformed section properties for an identical structure to the one analyzed in example 9.1b. How is this apparent contradiction reconciled?

Richard Brice, PE
Bridge software engineer, Washington State Department of Transportation, Bridge and Structures Office
Olympia, Wash.

The authors have prepared an excellent paper that describes in detail the time-dependent analysis method for determining loss of prestress in pretensioned bridge girders given under "Refined Estimates of Time-Dependent Losses" in the AASHTO LRFD specifications.[1] The authors should be congratulated for providing clear and useful information on the fundamental mechanics on which this method is based. This is acknowledged by the discusser, who would also like to thank *PCI Journal* for the opportunity to offer the following comments, mainly about loss of prestress at transfer and the age-adjusted effective modulus.

First, the authors are right to describe the sources of prestress losses between initial jacking and the time just before transfer of prestress: friction, anchorage seating, prestressing steel relaxation, and elastic shortening. It should be clear that elastic shortening is the only source of prestress loss that occurs exactly at prestress transfer, whereas the other three sources take place beforehand. In particular, prestress losses by prestressing steel relaxation (lowered tensile stress under sustained elongation) are time dependent and are strongly influenced by the manufacturing process. The concrete member is cast around the prestressing tendon while relaxation loss of the prestressing tendon occurs, and additional factors, such as curing temperature, increase relaxation prestress loss. An example of overjacking prestressing steel strands to counteract relaxation losses has been done to evaluate creep and shrinkage prestress losses in prestressed concrete prismatic specimens.[3-5] By following the manufacturer's recommendations, prestressing reinforcement (a low-relaxation seven-wire steel strand of 13 mm [0.5 in.] in diameter) was overtensioned at 82% of the nominal ultimate reinforcement strength over a 10-minute period prior to anchoring. Afterward, tension was reduced to the desired prestress level of 75%.

Do the authors have any information on overjacking practices to counteract relaxation or other prestress losses? The transfer length of prestressing reinforcement can increase with time.[6,7] However, changes in prestressing reinforcement stress (influenced by concrete creep and shrinkage and steel relaxation) are not directly related to changes in transfer length. Therefore, the sections between the initial and final transfer lengths show additional prestress losses. A transfer length model with a factor accounting for transfer length changes with time has been presented,[7] and a comparative study on European[8,9] versus North American[10] practices for transfer length, regarding the effective stress considered (initial effective stress—just after prestress transfer—or effective stress after allowing for all prestress losses) has been done.[11]

Second, as stated by the authors, the time-dependent material property models and the methods for estimating prestress loss are independent of each other. Thus any suitable model can be used. Concrete properties, such as modulus of elasticity, shrinkage strain, and creep strain, are considered in the AASHTO LRFD specifications prestress loss provisions, and a method for calculating each concrete property is required. Determination of prestress losses usually involves laborious procedures because time-dependent prestress losses are interdependent:[12]

- prestressing reinforcement relaxation is continuously altered by changes in stress due to concrete shrinkage and creep
- concrete creep, in turn, constantly alters by changes in prestressing reinforcement stress
- concrete shrinkage and creep movements are partially restrained by prestressing reinforcement[3]

To account for this, an age-adjusted effective elasticity modulus of concrete is obtained by using a dimensionless multiplier to the creep coefficient,[13,14] the aging coefficient, which depends on age at loading, load duration, development of concrete modulus of elasticity with time, and the creep coefficient. Aging coefficient values ranging from 0.5 to 1 have been reported by ACI Committee 209[15] (Table 5.1.1) and by the Euro-International Concrete Committee[16] (appendix 3 adimensional diagrams). In other cases, values varying from 0.6 to 0.9,[14] from 0.7 to 0.9,[13] or 0.8 as a mean value for most cases[8,9,13,14] have been established. However, the AASHTO LRFD specifications establish a unique age-adjusted effective modulus, which includes an aging coefficient equal to 0.7. Can the authors explain this?

Moreover, measured prestress losses exceed the losses predicted by code specifications in some cases,[17,18] and the measured prestress losses that are in line with the values expected by current codes have been obtained in prestressed concrete girders that exceeded the allowable compressive stress limit.[19] Perhaps the aging coefficient should be also considered by means of a model, rather than as a constant value, to account for the specific characteristics of a pretensioned concrete member and to obtain a better prestress loss estimation.

Last, as the prestressing force is transferred instantaneously and the time-dependent prestress losses are gradual, the concrete compressive stress diminishes with time.

Therefore, and complementarily to Fig. 4, stress changes by means of decrements are more representative for the case of pretensioned concrete members.

José R. Martí-Vargas
Associate professor, Institute of Concrete Science and Technology, Universitat Politècnica de València
Valencia, Spain

Authors' response

The authors appreciate the insightful and constructive discussion comments related to this paper. Brice correctly notes that the AASHTO T-10 committee is moving toward reorganizing section 5 of the AASHTO LRFD specifications,[1] including the provisions related to loss of prestress. The authors have prepared a detailed Portland Cement Association (PCA) research report[20] presenting an alternative approach to determining prestress losses, currently being called the direct method. The direct method intends to mirror the format of the traditional method (published in the AASHTO LRFD specifications until 2004) while making similar technical advancements for higher-quality concrete and the inclusion of deck shrinkage seen in the refined method. The direct method does not divide the time periods into pre- and post-deck placement ranges as the refined method has because the service-level stresses are largely insensitive to the age at deck placement, as substantiated in the PCA report. Also, the creep and shrinkage model is inherent in the direct method, rendering it less flexible but more straightforward to apply. The direct method has been presented to AASHTO Committee T-10 for consideration and formalized as working agenda item 157.

The authors' concern with the refined method is not its technical accuracy but rather the confusion engineers have experienced through its implementation. Any improvements made through the rigor of the approach are quickly negated if it is applied incorrectly.

Brice's discussion further asks for clarification on elastic gains and losses. In part, this point has been made confusing by the endorsement of transformed section properties for stress calculations in prestressed girders. Again, the use of transformed section properties offers some technical advantages compared with gross section properties, but the procedure is often misunderstood and applied incorrectly.

Examples 9.1a (transformed section properties) and 9.1b (gross section properties) in the *PCI Bridge Design Manual*[2] have attempted to provide clarity. They arrive at nearly identical numerical results after selectively including elastic gains that the authors believe to be inconsistent. Inclusion of the elastic gains numerically compensates for the reduced stiffness of the gross section (where the increased modulus in the area occupied by steel is ignored) that would be considered in transformed section properties. This approach is reasonable only in sections with small reinforcement ratios. The elastic gains included seem selective because only elastic gains due to dead load are included, not elastic gains due to live load. In this case, gravity loads from both live load and dead load would have the same effect on stress in the prestressing steel and flexural stress in the concrete. The fact that one may be more gradual and sustained than the other is not relevant. If prestressing gains due to gravity dead load act to resist flexural tensile stresses caused by the same loading, it seems the same should apply to live loading. Certainly the additional stiffness due to the presence of prestressing steel is included when stress evaluation is done by transformed section properties. Example 9.1 in the *PCI Bridge Design Manual* illustrates the potential for confusion regarding inclusion of the effect of deck shrinkage and the corresponding prestressing gain in accordance with the AASHTO LRFD specifications. The effect of deck shrinkage cannot be termed a prestress gain without causing considerable confusion. The explanation provided in 9.1a.8.5 is suitable but should be made more prominent so that it is not so easily overlooked.

Martí-Vargas correctly points out that overjacking is a commonly used approach to counteract losses, particularly relaxation losses, that occur prior to transfer. Unfortunately, the authors do not have experience or particular expertise related to overjacking and its effects on time-dependent losses.

Martí-Vargas also correctly asserts that the AASHTO LRFD refined method for loss of prestress adopts a constant value of 0.7 for the aging coefficient. That decision is documented in National Cooperative Highway Research Project report 496,[21] referencing work done by Dilger.[22] It seems apparent that the sensitivity of service-level stress calculations to the choice of aging coefficient value is small compared with other factors, particularly the choice of creep, shrinkage, and elastic modulus models. Considering that the refined method is already sufficiently complex, in the authors' opinions, for the LRFD specifications, calculating the aging coefficient by some mathematical model is unnecessary and would not be fruitful.

Brian D. Swartz, PhD, PE
Assistant professor of Engineering, Messiah College
Mechanicsburg, Pa.

Andrew Scanlon, PhD
Professor, Penn State University
University Park, Pa.

Andrea J. Schokker, PhD, PE
Professor and head of Civil Engineering, University of Minnesota
Duluth, Minn.

Significant changes from the 2008 to the 2011 edition of ACI 318

S. K. Ghosh
PCI Journal, Winter 2013, pp. 142–154.

I am reviewing concrete construction for the Structural Engineering exam. After reading Dr. Ghosh's article "Significant Changes from the 2008 to the 2011 Edition of ACI 318" in the Winter 2013 issue of *PCI Journal*, I was given the impression that shear is allowed to be calculated either per method (a) or (b) in section 21.3.3, based on wording in the paper: "Section 21.3.3 of ACI 318-08 provided two choices for the calculation of the required shear strength of a column of an intermediate moment frame.". The code has an "and," which requires that both (a) and (b) be calculated and the smaller of the two values be used. Can you please clarify if either (a) or (b) may be used or if (a) and (b) are to be checked per the code? For the designer to have a choice, the code should state (a) or (b), or allows an option, and requires both to be addressed.

Thank you for any assistance you can give me.

Steve Pancrazio
Goodyear, Ariz.

Author's response

The exact language in ACI 318-11 section 21.3.3 is: "shall not be less than the smaller of (a) and (b)."

So it can be equal to the smaller of (a) and (b). Thus, if you calculate *Vu* by (a) and ignore (b), you are fine. If (a) is smaller than (b), your *Vu* is the smaller of values given by (a) and (b), which is allowed. If (a) is larger than (b), your *Vu* is the larger of (a) and (b), which is allowed as well. It works the same way if you calculate *Vu* by (b) and ignore (a). I hope this clarifies the issue.

S. K. Ghosh, PhD, FPCI
President, S. K. Ghosh Associates Inc.
Palatine, Ill.

Residual strength assessment and destructive testing of decommissioned concrete bridge beams with corroded pretensioned reinforcement

The following comments relate to "Residual Strength Assessment and Destructive Testing of Decommissioned Concrete Bridge Beams with Corroded Pretensioned Reinforcement," by Rhys A. Rogers, Liam Wotherspoon, Allan Scott, and Jason M. Ingham, on pages 100 through 118 in the Summer 2012 issue of *PCI Journal*.

Based on destructive flexural tests performed on 19 decommissioned pretensioned concrete bridge beams with different corrosion conditions of prestressing strands, the authors present an excellent paper that proposes a methodology for the assessment of residual strength. This methodology provides an effective means of estimating the number of corroded strands, which should be disregarded when calculating the residual strength of beams with corroded pretensioned reinforcement. The authors should be congratulated for this paper. In addition, the discusser would like to thank *PCI Journal* for the opportunity to offer the following comments, mainly about the background on residual strength assessment.

Corrosion of the prestressing reinforcement strongly affects its mechanical properties, mainly resulting in degradation of tensile strength, ductility, modulus of elasticity, and fatigue.[23] Besides, in the case of pretensioned, prestressed concrete members, prestressing force is transferred from the prestressing reinforcement to concrete by bond. Therefore, the influence of corrosion on these material properties and bond performance must be considered in the long-term structural strength and safety of pretensioned, prestressed concrete structures.

In the discusser's opinion, it might not come over clearly for readers that in the background section on residual strength assessment, the authors have stated that "corrosion can cause relaxation of the steel and also compromises the integrity of the bond between the steel and concrete," so more information is needed.

According to the manufacturing process for pretensioned, prestressed concrete members,[4,5] first the prestressing reinforcement is tensioned in a casting bed. Next, while relaxation loss of prestressing reinforcement occurs, the concrete member is cast around the prestressing reinforcement. Last, when sufficient strength is attained by the concrete, the prestressing reinforcement is released and the prestressing force is applied to the concrete by bond at the ends of the members. Along the transfer length,[10,24] the prestressing reinforcement stress varies from zero at the free ends of the member to the effective stress in the central zone. Instantaneous prestress losses due to the elastic shortening of concrete occur. As time elapses after the prestress transfer, time-dependent prestress losses (by concrete creep and shrinkage and prestressing reinforcement relaxation) gradually occur in the member,[3] causing effective stress variation from its initial value, just after prestress transfer, to its final value after allowance for all prestress losses.[11] As stated, prestressing reinforcement relaxation exists from the first moment after tensioning.

Do the authors have any information on how corrosion causes relaxation of steel? One source of relaxation of steel in an indirect manner can be explained from loss of the cross-sectional area of the prestressing reinforcement owing to corrosion. At tensioning, prestressing steel reaches a high stress level (75% to 80% of its specified minimum ultimate tensile strength). Prestress losses reduce this stress and thus the amount of prestress transferred to the concrete, but external applied loads increase prestressing steel stress. Moreover for the same loading stage, cross-sectional area loss due to general or pitting corrosion incrementally increases the net stress in the prestressing tendon. In this case, the higher stress leads to further relaxation of the steel. In addition, the higher stress accelerates corrosion,[25] in turn resulting in a further loss of cross-sectional area of prestressing tendon and, consequently, greater prestressing steel stress, and increases the risk of structural collapse.

Furthermore, adequate bond behavior between the tendon and the concrete is essential for the structural performance of pretensioned, prestressed concrete members.[26] Corrosion can improve bond strength as follows: the bond mechanisms based on friction and mechanical action, which are caused by radial compressive stresses around the prestressing reinforcement[7,27] when it slips into the concrete,[28] improve because the corrosion products expand in volume and cause higher pressure between prestressing reinforcement steel and concrete. However, corrosion also can degenerate bond strength if the higher pressure causes cover concrete cracks. In this case, the amount of prestress transferred to the concrete reduces more drastically if the cracks are parallel to the prestressing reinforcement and the transfer length of the prestressing reinforcement increases (similar to a debonding length along the crack length). Then the pretensioned, prestressed concrete member weakens in both structural capacity because the transfer length remains within the required development length at loading[10,29] and durability because cracks can accelerate corrosion. Differences between the bond behaviors of corroded and uncorroded prestressing reinforcement have been reported,[30] but the effect of prestressing reinforcement corrosion was evident in tensile strength degradation, and not in bond strength. Therefore, the bond between the prestressing steel reinforcement and concrete is significant for the residual strength of the corroded pretensioned beams when bond integrity is compromised because of the concrete cracking due to corrosion.

José R. Martí-Vargas
Associate professor, Institute of Concrete Science and Technology, Universitat Politècnica de València
Valencia, Spain

Authors' response

The authors would like to thank José R. Martí-Vargas for his useful clarifications and comments on our paper and research. The explanation provided on the construction process for pretensioned, precast concrete members and the explanation of instantaneous and long-term prestress losses are useful additions to the paper. The indirect mechanism described by Martí-Vargas for steel relaxation as a result of corrosion is indeed the mechanism that was referred to in the original 2012 paper. The authors acknowledge that this is not the same mechanism as is usually referred to by the term steel relaxation; however, the term does accurately describe the effect, and relaxation of steel by any mechanism has a similar effect on the performance of a prestressed concrete member. The discussion and references provided by Martí-Vargas on the bond behavior of corroded and noncorroded strands in concrete are particularly interesting, and the authors are grateful for this contribution.

Rhys A. Rogers
Structural engineer, Construction Techniques Ltd.
Auckland, New Zealand

Liam Wotherspoon
Research fellow, University of Auckland
Auckland, New Zealand

Allan Scott
Lecturer, University of Canterbury
Christchurch, New Zealand

Jason M. Ingham
Professor, University of Auckland
Auckland, New Zealand

References

1. AASHTO (American Association of State Highway and Transportation Officials). 2012. *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington, DC: AASHTO.
2. PCI Bridge Design Manual Steering Committee. 2011. *PCI Bridge Design Manual*. MNL-133. 3rd ed. Chicago, IL: PCI.
3. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2013. "Prestress Losses Evaluation in Prestressed Concrete Prismatic Specimens." *Engineering Structures* 48: 704–715.
4. Martí-Vargas, J. R., P. Serna-Ros, M. A. Fernández-Prada, P. F. Miguel-Sosa, and C. A. Arbeláez. 2006. "Test Method for Determination of the Transmission and Anchorage Lengths in Prestressed Reinforcement." *Magazine of Concrete Research* 58 (1): 21–29.
5. Martí-Vargas, J. R., C. A. Arbeláez, P. Serna-Ros, M. A. Fernández-Prada, and P. F. Miguel-Sosa. 2006. "Transfer and Development Lengths of Concentrically Prestressed Concrete." *PCI Journal* 51 (5): 74–85.
6. Martí-Vargas, J. R., L. Caro, and P. Serna. 2013. "Experimental Technique for Measuring the Long-Term Transfer Length in Prestressed Concrete." *Strain*, 49: 125-134.
7. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2012. "Time-Dependent Evolution of Strand Transfer Length in Pretensioned Prestressed Concrete Members." *Mechanics of Time-Dependent Materials*, doi: 10.1007/s11043-012-9200-2.
8. CEN (Comité Européen de Normalisation). 2004. *Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings*. European Standard EN 1992-1-1:2004:E. Brussels, Belgium: CEN.

9. *fib* (International Federation for Structural Concrete). 2010. *Model Code 2010*. First complete draft, v. 1, bulletin 55. Lausanne, Switzerland: *fib*.
10. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete and Commentary (ACI 318-11)*. Farmington Hills, MI: ACI.
11. Martí-Vargas, J. R., and W. M. Hale. 2012. "Predicting Strand Transfer Length in Pretensioned Concrete: Eurocode versus North American Practice." *ASCE Journal of Bridge Engineering*, doi:10.1061/(ASCE)BE.1943-5592.0000456.
12. Au, F. T. K., and X. T. Si. 2011. "Accurate Time-Dependent Analysis of Concrete Bridges Considering Concrete Creep, Concrete Shrinkage and Cable Relaxation." *Engineering Structures* 33 (1): 18–126.
13. ACI Committee 435. 2003. *Control of Deflection in Concrete Structures*. ACI 435R-95, appendix B. Farmington Hills, MI: ACI.
14. Youakim, S. A., A. Ghali, S. E. Hida, and V. M. Karbhari. 2007. "Prediction of Long-Term Prestress Losses." *PCI Journal* 52 (2): 116–130.
15. ACI Committee 209. 1997. *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures (ACI 209R-92)*. Farmington Hills, MI: ACI.
16. CEB (Euro-International Concrete Committee). 1993. *Structural Effects of Time-Dependent Behaviour of Concrete*. Bulletin 215. Lausanne, Switzerland: CEB.
17. Baran, E., C. K. Shield, and C. E. French. 2005. "A Comparison of Methods for Experimentally Determining Prestress Losses in Pretensioned Prestressed Concrete Girders." In *Historic Innovations in Prestressed Concrete*. Edited by B. W. Russell and S. P. Gross, 161–179. Farmington Hills, MI: ACI.
18. Al-Omaishi, N., M. K. Tadros, and S. J. Seguirant. 2009. "Estimating Prestress Loss in Pretensioned, High-Strength Concrete Members." *PCI Journal* 54 (4): 132–159.
19. Hale, W. M., and B. W. Russell. 2006. "Effect of Allowable Compressive Stress at Release on Prestress Losses and on the Performance of Precast, Prestressed Concrete Bridge Girders." *PCI Journal* 51 (2): 14–25.
20. Swartz, B. D., A. J. Schokker, and A. Scanlon. 2010. *Simplification of Prestress Loss Provisions for the AASHTO LRFD Bridge Design Specifications*. Skokie, IL: Portland Cement Association.
21. Tadros, M. K., N. Al-Omaishi, S. J. Seguirant, and J. G. Gallt. 2003. *Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders*. National Cooperative Highway Research Program report 496. Washington, DC: Transportation Research Board, National Academy of Sciences.
22. Dilger, W. H. 1982. "Creep Analysis of Prestressed Concrete Structures Using Creep Transformed Section Properties." *PCI Journal* 27 (1): 89–117.
23. ACI 222. 2001. *Corrosion of Prestressing Steels (ACI 222.2R-01)*. Farmington Hills, MI: ACI.
24. Martí-Vargas, J. R., P. Serna, J. Navarro-Gregori, and J. L. Bonet. 2012. "Effects of Concrete Composition on Transmission Length of Prestressing Strands." *Construction and Building Materials* 27: 350–356.
25. Li, F., Y. Yuan, and C. Q. Li. 2011. "Corrosion Propagation of Prestressing Steel Strands in Concrete Subject to Chloride Attack." *Construction and Building Materials* 25: 3878–3885.
26. Martí-Vargas, J. R., P. Serna, J. Navarro-Gregori, and L. Pallarés. 2012. "Bond of 13 mm Prestressing Steel Strands in Pretensioned Concrete Members." *Engineering Structures* 41: 403–412.
27. Martí-Vargas, J. R., C. A. Arbeláez, P. Serna-Ros, J. Navarro-Gregori, and L. Pallarés-Rubio. 2007. "Analytical Model for Transfer Length Prediction of 13 mm Prestressing Strand." *Structural Engineering Mechanics* 26: 211–229.

28. Martí-Vargas, J. R., C. A. Arbeláez, P. Serna-Ros, and C. Castro-Bugallo. 2007. "Reliability of Transfer Length Estimation from Strand End Slip." *ACI Structural Journal* 104 (4): 487–494.
29. Martí-Vargas, J. R., P. Serna, and W. M. Hale. 2013. "Strand Bond Performance in Prestressed Concrete Accounting for Bond Slip." *Engineering Structures* 51: 236–244.
30. Li, F., and Y. Yuan. 2013. "Effects of Corrosion on Bond Behavior between Steel Strand and Concrete." *Construction and Building Materials* 38: 413–422.

Comments?

The editors welcome discussion of the technical content of *PCI Journal* papers. Comments must be confined to the scope of the paper to which they respond and should make a reasonable and substantial contribution to the discussion of the topic. Discussion not meeting this requirement will be returned or referred to the authors for private reply.

Discussion should include the writer's name, title, company, city, and email address or phone number and may be sent to the respective authors for closure. All discussion becomes the property of *PCI Journal* and may be edited for space and style. Discussion is generally limited to 1800 words with each table or illustration counting as 300 words. Follow the style of the original paper, and use references wherever possible without repeating available information.

The opinions expressed are those of the writers and do not necessarily reflect those of PCI or its committees or councils.

All discussion of papers in this issue must be received by January 1, 2014. Please address reader discussion to *PCI Journal*l at journal@pci.org.