READER COMMENTS

High Performance Concrete and Reinforcing Steel with a 100-Year Service Life*

by Donald W. Pfeifer

Comments by Richard W. Burrows and Author

RICHARD W. BURROWS†

I read with much interest your paper on the performance of reinforced concrete specimens with a projected 100-year service life. I have the following comments.

The first paragraph of the synopsis directly contradicts the last paragraph of the paper. The first paragraph cites the crack-proneness of high strength concrete due to thermal and shrinkage contractions coupled with its high modulus of elasticity and lack of creep.

The last paragraph of the conclusions then proceeds to recommend high strength concrete as follows: "A low water-cementitious ratio (less than 0.40) is essential in producing high quality, durable concrete. The precast, prestressed concrete industry has had a long tradition in fabricating precast concrete products with low water-cementitious ratios which have resulted in excellent long-term durabilities in aggressive environments."

The statement that the precast, prestressed concrete industry has successfully used such concrete is true because the prestressing prevents the development of high tensile self-stresses from thermal contraction, autogeneous shrinkage, and drying shrinkage. However, cast-in-place concrete in bridge decks and parking structure decks is an entirely different "animal."

Here, the concrete is highly restrained against shrinking and is very subject to cracking from the above-mentioned three self-stresses. Mr. Pfeifer cites the WJE-prepared report, TRB Report 380, but does not mention its recommendation for preventing early-age cracking – which advocates using the minimum possible amount of cement – a concept that is completely incompatible with the recommendation that the water-cement ratio should be less than 0.40.

The Colorado Department of Transportation used concrete with a water-cement ratio of 0.31 in the 23rd Street Viaduct which cracked before it was finished. The increased cracking that occurs when the water-cement ratio is reduced has also been observed with bridge decks in Virginia, Kansas and Texas.

The cement also plays an important role. If one orders a Type II cement and is unlucky enough to get one with a high seven-day strength of 5200 psi (36 MPa) and then uses it at a water-cement ratio of 0.40, not even adding 20 to 30 percent fly ash will prevent cracking. In New York State, 51 percent of their high performance concrete bridge decks with 20 percent fly ash still crack. The problem is the water-cement ratio of 0.37. It should be increased to 0.48, the preferred value in Germany.

T.C. Powers is cited as writing that a 0.40 water-cement ratio concrete was desirable because the capillaries are closed, that is, the pore system is discontinuous. However, T.C. Powers also wrote that, with a 0.50 water-cement ratio concrete, the capillaries close in 14 days.

The ring test was recommended as a useful tool for the Departments of Transportation. However, this test evaluates the cracking from drying shrinkage, not the cracking from thermal contraction, which is a greater problem in bridge decks. Drying shrinkage cracking is not involved in early-age cracking since it occurs later. For evaluating thermal contraction cracking, RILEM TC 119 should be used.

Regarding autogenous shrinkage, a report by Igarashi,

* PCI JOURNAL, V. 45, No. 3, May-June 2000, pp. 46-54.

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Bentur, and Kovler, *Advances in Cement Research*, October, 1999, concluded that concrete with a 0.33 water-cement ratio, the induced stress-to-strength ratio could approach 50 percent, thereby bringing about a risk of cracking. If the ambient temperature around the stressed concrete dropped, or drying shrinkage began, then the added tensile stress could exceed the strength.

AUTHOR'S CLOSURE by DONALD W. PFEIFER*

I greatly appreciate your timely discussion of this paper. During my over 20 years of consulting activities at Wiss, Janney, Elstner Associates, Inc. (WJE), we became acutely aware of the unanticipated early-age and moderate-age cracking of high performance concretes having high compressive strength, high modulus of elasticity, low creep, and high heat of hydration properties.

WJE was selected by the National Cooperative Highway Research Program to undertake a three-year research study titled "Transverse Cracking in Newly Constructed Bridge Decks," a landmark cracking study that was published in 1996 as NCHRP Report 380. I am also well aware of the report, "202 Observations on Too-Quickly-Strong Concrete." Your 20-page document certainly supports our experience about the unanticipated cracking of high performance concretes as used in the last 15 years.

While your review states that 0.40 water-cement ratio concretes usually have a 28-day strength exceeding 9000 psi (62 MPa), that statement does not agree with numerous 0.40 water-cement ratio concretes WJE has reported in publications by the Federal Highway Administration, the PCI JOURNAL and other sources. Experience shows that 0.40 water-cement ratio concretes have nominal 28-day strengths of 6000 psi (41 MPa), with 0.32 water-cement ratio concretes having nominal 28-day strengths of 7500 psi (52 MPa) concretes.

With respect to the differences in cracking problems of cast-in-place concrete versus precast concrete, it is clear from our experience that discreet, individual sections of precast concrete have little external restraint and subsequent cracking in their early-age period from the effects of heat of hydration and curing and the subsequent thermal drop from these factors. On the other hand, cast-in-place concrete has been subjected to significant cracking under the same early-age conditions created by the natural restraint from that construction method when subjected to thermal effects, compounded by the low creep of high performance concretes, which when under restraint produce a more brittle, crack-prone concrete.

Since the concrete industry is using high performance, low water-cement ratio concretes for corrosion protection of embedded reinforcing steel, the question of cracking of these special concretes and the immediate loss of the low chloride ion permeability properties of these high performance concretes must be addressed. Numerous owners have had serious cracking with their specified low water-cement or water-cementitious ratio concretes and as a result, any estimated service life from Fick's law of diffusion equations of cracked concrete is, at that point, highly questionable.

The corrosion study referenced in the reviewed paper was funded by the Federal Highway Administration (FHWA) over a five-year period from 1993 to 1998, and the final two-year period was utilized to evaluate crack-free and intentionally cracked concrete using black bars, galvanized bars, three bendable and three nonbendable epoxy coated bars, copper-clad bars, and Type 304 and 316 stainless steel bars. The goal was to determine if a reliable 75- to 100-year design life could be calculated after these exhaustive two-year tests, even with initially cracked concrete.

The most important conclusion from the five-year FHWA-funded study was that Type 304 and 316 stainless steel reinforcing bars could achieve a 100-year design service life, based on corrosion, current calculations, even when tested in intentionally cracked concrete. These stainless steel bars were able to achieve a 99.8 percent reduction in corrosion current density, when compared to the black bar.

The concrete selected for these exhaustive FHWA tests was a standard AASHTO Class A (AE) concrete with a nominal 0.45 water-cement ratio. The actual target water-cement ratio was 0.47. The 28-day strength for this air-entrained concrete was 5700 psi (39.3 MPa). The curing period was a realistic three-day period under wetted burlap and polyethylene, not the seven-day AASHTO requirement.

The most important question that arises from the reviewed paper is why the industry is using high performance concretes that have such a significant cracking potential, when conventional concretes with a much lower cost can be combined with stainless steel bars to produce a 100-year service life, even when used in precracked concrete.

I sincerely hope that readers of this discussion, who are interested in the 20-page list of 202 published references to cracking problems of high strength, high performance concretes, will contact Mr. Burrows so that he could send them this most informative compilation to all interested parties.

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READER COMMENTS

Torsional and Other Properties of Prestressed Concrete Sections*

by Chai H. Yoo

Comments by Alex Aswad, Mostafa A. Hassanain, and Author

ALEX ASWAD†

The author is to be congratulated on a carefully executed investigation that furnished some valuable data in a field where data on torsional constants are short. He is also to be commended for making a software program freely available for use in personal computers.

In the concluding section, the author hints that the accuracy of the St. Venant torsional constants for the six types of bridge girders (AASHTO Types I to VI) in PCI's Bridge Design Manual (Ref. 7) is questionable. As principal author of Chapter 7 in that Manual, I disagree with the author's statement.

In fact, the cited values in Ref. 7 are based on exactly the same procedure, namely, a finite difference solution of the partial differential Eq. (1). The values were reproduced from Ref. 9 by Eby et al. Refs. 9 to 11 all used the same procedure of finite differences starting in 1965. Ref. 9 mentions that the mesh pattern used 160 to 185 points for which $z \neq 0$. This is a lesser

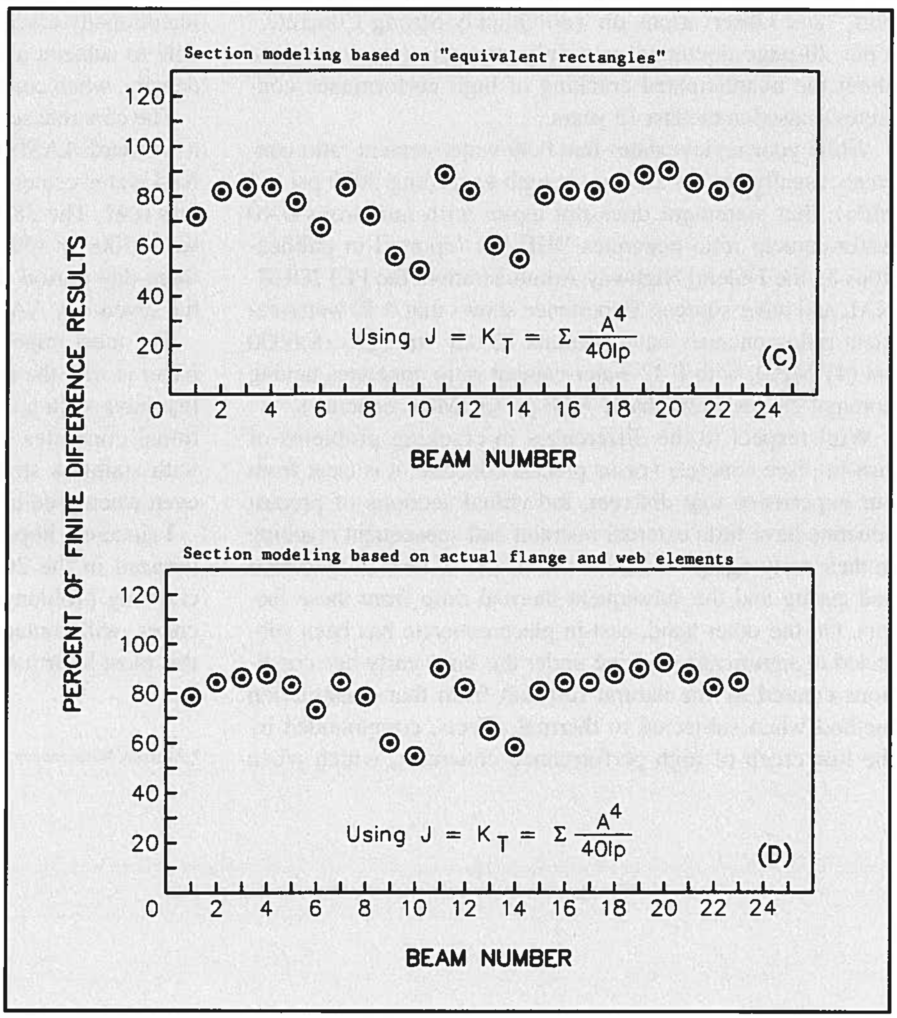

Fig. A. Comparison of results for St. Venant torsional constant *J* using the approximate and the finite difference procedures (from Ref. 9).

* PCI JOURNAL, V. 45, No. 3, May-June 2000, pp. 66-72.

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number of points than what the author had used, but sufficiently accurate in computational mechanics practice. As a matter of fact, the average absolute relative deviation in Table 2 is 4.3 percent only.

Though one would expect a smaller difference because of the large number of points used by both Ref. 9 and the author, I suspect the 4.3 percent is mostly due to the differing treatment of mesh points close to the boundary. The author may want to comment on that aspect of the analysis since Fig. 2 is not detailed enough and its scale is too small.

Finally, I would like to reassure the users of PCI's Bridge Design Manual that the listed J values are quite adequate for input to a refined method of analysis such as when the model assumes precast beam elements rigidly attached to horizontal shell elements (simulating the deck slab). Pilot studies have shown that deviations in the J value as high as 12 percent affected the resultant sectional moment by less than 1 percent. Sectional forces are not very sensitive to J values. For users facing other geometric shapes (such as bulb tees) a rapid algebraic procedure has been developed by Eby et al. (Ref. 9) and is summarized in Ref. 12.

On the other hand, I fully agree with the author that St. Venant's approximate Eq. (19) should not be used for I-shapes in bridges since the equation is limited to sections without re-entrant corners. Eby et al. (Ref. 9) show a comparison between the results from Eq. (19) and the finite difference procedure. This is graphically displayed in Fig. A. The underestimation can be as large as 45 percent! Why the equation is still in the AASHTO-LRFD Specification is a mystery. Anyway, under-estimating the torsional stiffness of a precast beam element in a refined analysis is generally conservative.

ADDITIONAL REFERENCES

9. Eby, C. C., Kulicki, J. M., Kostem, C. N., and Zellin, M. A., "The Evaluation of St. Venant Torsional Constants for Prestressed Concrete I-Beams," Fritz Engineering Laboratory Report No. 400.12, Department of Civil Engineering, Lehigh University, Bethlehem, PA, September 1973.
10. El-Darwish, E. S., and Johnston, B. G., " Torsion of Structural Shapes," *Journal of the Structural Division*, Proceedings of ASCE, V.91, No. ST1, February 1965, pp. 203-228.
11. Tamberg, K. G., "Elastic Torsional Stiffness of Prestressed Concrete AASHO Girders," ACI Two-Part Paper, *ACI Journal*, April 1965.
12. Chen, Y., and Aswad, A., "Stretching Span Capability of Prestressed Concrete Bridges Under AASHTO LRFD," *ASCE Journal of Bridge Engineering*, V. 1, No. 3, August 1996, pp. 112-120.

MOSTAFA A. HASSANAIN*

I would like to comment on the comparison made by the author in Table 2 of the above-noted paper. The table gives values for the St. Venant torsional constant, J, of AASHTO bridge girders as obtained from three sources. The first source is a computer program developed by the author based on the finite difference method. The second source is AASHTO LRFD approximate Eq. C4.6.2.2.1-2.[3] And the third source is the PCI Bridge Design Manual (Table 7.6.3-1).[7]

Readers of the paper will get the impression that the values obtained from Ref. 7 were computed from some approximate formula, especially that the author questioned their accuracy in his concluding remarks along with the accuracy of AASHTO approximate formulas. These values, in fact, were obtained numerically using the finite difference method in a study that was carried out at Lehigh University during the early seventies.[9] It seems that the major source of difference between the values of J computed by the author and those obtained in that study is due to mesh refinement.

With today's high-power computers, it is possible to use a finer mesh than what was practically possible in the early seventies. This could make the author's J values closer to the "exact" values. However, it is important to point out that the torsional constants currently listed in the PCI Bridge Design Manual are not based on some approximate formula, as would be understood from the paper.

It might be interesting for readers to know that there is an approximate formula for J that is more accurate than AASHTO LRFD Eq. C4.6.2.2.1-2. Eby et al.[9] considered the results obtained by the finite difference method as an "exact" solution, and they compared them to the results obtained using several approximate methods. One of those methods is based on the work of El-Darwish and Johnston.[10] Their method showed the best correlation with the "exact" solution.

An algebraic solution to the problem was then developed by modifying the method used in Ref. 10. With reference to Fig. B, the torsional constant of an I-girder can be obtained as follows:

$$J = \frac{1}{3}\left(b_1 t_1^3 + b_2 t_2^3 + d_3 b_3^3\right) + \alpha_1 D_1^4 + \alpha_2 D_2^4 - 0.21\left(t_1^4 + t_2^4\right) \tag{A}$$

where

$$t_1 = d_1 + \frac{d_2(b_1 + b_3)}{2b_1} \tag{B1}$$

$$t_2 = d_5 + \frac{d_4(b_3 + b_2)}{2b_2} \tag{B2}$$

$$\alpha_1 = -0.042 + 0.2204\frac{b_3}{t_1} - 0.0725\left(\frac{b_3}{t_1}\right)^2 \tag{C1}$$

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Table A. Bridge girder torsional constants, J.

Girder Type	J (in.^{4}) from Eq. (A)	J (in.^{4}) by the author	Percentage difference
I	4584	4820	- 4.90
II	7767	7372	+ 5.36
III	16,845	16,210	+ 3.92
IV	32,265	30,229	+ 6.74
V	33,371	35,044	- 4.77
VI	34,907	37,347	- 6.53

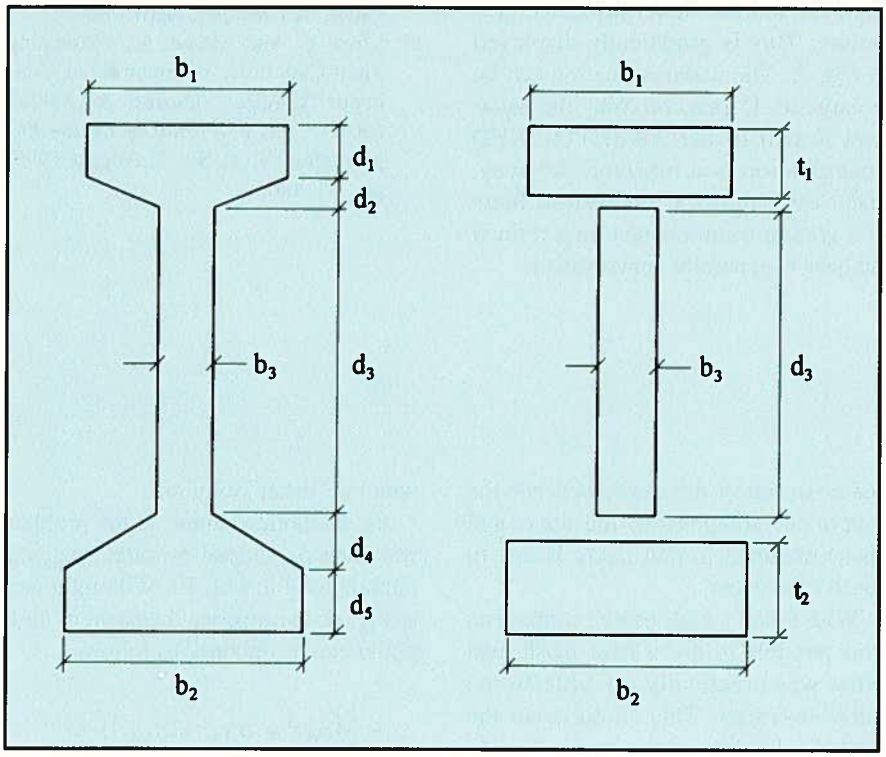

Fig. B. General I-girder section and discretization.

$$\alpha_2 = -0.042 + 0.2204\frac{b_3}{t_2} - 0.0725\left(\frac{b_3}{t_2}\right)^2 \qquad \text{(C2)}$$

$$D_1 = t_1 + \frac{b_3^2}{4t_1} \qquad \text{(D1)}$$

$$D_2 = t_2 + \frac{b_3^2}{4t_2} \qquad \text{(D2)}$$

Eq. (A) might look cumbersome to be evaluated by hand. However, it could be easily evaluated using a program like Mathcad®. For other precast I-girders that do not conform in shape to the girder shown in Fig. B, such as AASHTO Type V and Type VI sections, their dimensions should be idealized to fit the basic I-shape shown in the figure. The results obtained for these sections should be regarded as somewhat more approximate than those for the basic I-sections. Eby et al. reported that the torsional constants computed using Eq. (A) were within ±5 percent of their respective finite difference values.

Table A compares the J-values obtained using Eq. (A) to those computed using the author's finite difference computer program. It can be seen that the percentage differences for all girder types are quite reasonable for practical applications, which gives much confidence in Eq. (A) over AASHTO LRFD Eq. C4.6.2.2.1-2.

One final comment is that I believe it would be useful if the author would explain how he dealt with the different skewed boundaries in Type V and Type VI girders. Were they simulated with a series of steps as for the other girder types, or were there certain approximations? These bulb-tee shaped girders are more common today in bridge applications than the other types. It would be of interest to provide further explanation.

ADDITIONAL REFERENCES

9. Eby, C. C., Kulicki, J. M., Kostem, C. N., and Zellin, M. A., "The Evaluation of St. Venant Torsional Constants for Prestressed Concrete I-Beams," Fritz Engineering Laboratory Report No. 400.12, Department of Civil Engineering, Lehigh University, Bethlehem, PA, September 1973.

10. El-Darwish, I. A., and Johnston, B. G., "Torsion of Structural Shapes," *Journal of the Structural Division*, American Society of Civil Engineers, V. 91, No. ST1, February 1965, pp. 203-228.

AUTHOR'S CLOSURE by CHAI H. YOO*

The author wishes to thank Dr. Alex Aswad and Dr. Mostafa Hassanain for their thoughtful comments and interest in the paper.

Table 1 shows that the finite difference procedure used in this study converges monotonically from the lower bound. The author was concerned with the scattering (positive and negative) of the percentage differences in Table 2.

Dr. Aswad is invited to examine the convergence rate introduced in Table 1. Although the accuracy of a finite difference solution is not linearly proportional to the number of internal mesh points used, it does provide a reasonable indication of the convergence trend. The 16-subdivision case (Table 1) results in a total of 225 internal mesh points [(16-1) x (16-1) = 225, considerably more points than those used in Ref. 9]. The corresponding percentage error is 1.27. In fact, the number of internal node points used ranges from 1,030 to 2,176 for the six types of girder shapes in Table 2.

Skewed boundaries usually present a difficulty of the section being accurately represented by finite difference mesh equations in a rectangular Cartesian coordinate system. The author replaced the skewed lines by a series of small rectangular blocks as shown in Fig. 2, instead of introducing special interior node points for integration as was done in Ref. 9.

Maintaining a symmetric narrowly banded coefficient matrix (or unsymmetric when modeling only one-half the section to take advantage of the symmetry as shown in Fig. 2) is important for computational efficiency. Because the coefficient matrix is narrowly banded, it is possible to use a very large number of internal mesh points to model the AASHTO type girders.

The author agrees with Dr. Aswad that the St. Venant torsional constant is not very sensitive to the girder forces in the two-dimensional grid analysis of a typical right angle bridge superstructure where no torsional loading is applied. However, in the case of the fascia girder of a sharply skewed bridge subjected to eccentrically loaded utility pipelines and/or the spandrel beam of a building carrying reactions from eccentrically placed precast panels, the girder/beam forces might be affected a lot more than the discusser implied.

As the author has stated in the concluding remarks, this numerical method does not always give an exact value for the St. Venant torsional constant. The accuracy of the solution entirely depends on the mesh refinement; however, an opportunity exists to evaluate the torsional constant as close to the exact value as desired or needed by executing a desktop computer program with straightforward, simple data input system.

Table B. Torsional Constant, AASHTO Type VI Girder

No. of Interior Nodes	J (in.^{4})	Percent difference
513	37,437	5.07
2109	38,448	2.51
8580	39,252	0.47
34,470	39,438	–

The author wishes to thank Dr. Hassanain for his interest in the paper. Eq. (A) appears to yield reasonably accurate values for the St. Venant torsional constant, J, that can be used for all practical analyses where torsion is not the primary concern. A modeling technique for skewed boundaries adopted in the procedure is presented earlier in the response to Dr. Aswad.

It should be cautioned that the finite difference numerical solution should not automatically be considered exact. The convergence rate for various shapes with many skewed boundaries can be very slow. Table B shows the convergence rate for the AASHTO Type VI girder. The number of interior nodes is for one-half the cross section, as shown in Fig. 3.

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DISCUSSION NOTE

The Editors welcome discussion of reports and papers published in the PCI JOURNAL. The comments must be confined to the scope of the article being discussed. Please note that discussion of papers appearing in this issue must be received at PCI Headquarters by May 1, 2001.