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Pervasiveness of Excessive Segmental Bridge Deflections: Wake-Up Call for Creep

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*A recent study revealed that the main cause of the gross underestimation of the observed 18-year deflections of the Koror-Babeldaob (KB) Bridge in Palau was the use of an obsolete standard recommendation for creep design. Motivated by this finding, a search for data on similar bridges was undertaken. The results amount to a wake-up call: 56 other large-span, prestressed concrete, segmentally erected box girders (66 by the time of proof) have already been found to exhibit excessive long-time deflections and it is likely that many more such girders exist. The observed deflections give no sign of approaching a finite bound, as implied in the empirical ACI Committee 209, CEB-*fib*, and GL models for creep, and are found to evolve approximately logarithmically beginning at approximately 1000 days after span closing. Whereas sufficient data for the finite element (FE) creep analysis of these deflections proved impossible to obtain, it is demonstrated by comparisons with accurate deflection solutions for the KB Bridge that the terminal logarithmic deflection trend can be predicted quite well by a simple extrapolation of the measured 1000-day deflection under the hypothesis of proportionality to the compliance function increment since the time of span closing. Comparisons of the extrapolations according to various creep models show that, for the theoretically based Model B3, which is a 1995 RILEM recommendation, the underestimation of long-time deflections is much less severe than it is for the ACI Committee 209, CEB-*fib*, and GL models, and that the terminal trend is correct—that is, logarithmic. Because Model B3's terminal trend can be separately controlled, a simple update of this model that gives the same mean terminal trend as the 56 bridges is devised. The use of this updated Model B3 should allow for the improvement of the durability of segmental bridges and other structures that are highly sensitive to creep.*

Keywords: box girders; creep prediction models; design code; multi-decade creep; shrinkage.

INTRODUCTION

A recent article[1] reported a detailed analysis of the data on the 1996 collapse of the Koror-Babeldaob (KB) Bridge in Palau released in 2008. Built in 1977 and designed according to the CEB 1972 creep model,[2] this prestressed, segmentally erected box girder had the world-record span of 241 m (791 ft). Within 18 years, it had deflected by 1.61 m (5.28 ft), as measured from the design camber, and its tendons (bonded bars) suffered an average prestress loss of 50%, as revealed by measurements. Remedial prestressing undertaken in 1996 caused a sudden collapse after a 3-month delay (Fig. 1).

In 2008, the data from the investigation and litigation of this collapse were released to Northwestern University. This made possible a three-dimensional (3-D) finite element (FE) step-by-step creep analysis of the KB Bridge, taking into account the effects of cracking, concrete aging, shear lags in slabs and walls, nonuniform shrinkage, nonuniform drying creep properties, temperature, and gradual stress relaxation in the prestressing steel. The results showed that the excessive deflections and prestress loss can be explained and even closely matched if the theoretically based Model B3,[3,4] which became a 1995 RILEM recommendation,[3] is used to characterize the creep and shrinkage properties, provided that a compliance function agreeing with the 30-year laboratory creep tests of Brooks[5] is considered.

Models other than Model B3, which include the current ACI Committee 209, CEB-*fib*, and GL models[6-10] for creep and shrinkage prediction, have a mathematical form that does not allow recalibration by laboratory data. The same FE program for 3-D creep analysis was also run for the ACI Committee 209, CEB-*fib* and GL models, and for Model B3, as originally calibrated by a worldwide laboratory database.[4] The 18-year midspan deflections computed from these models were 31%, 34%, 43%, and 57% of the measured deflection, respectively, and the computed 18-year prestress losses were 44%, 48%, 54%, and 80% of the mean measured loss, respectively.

Are the dismal predictions of the standard recommendations of engineering societies merely a coincidental bad experience? Or are they endemic to the segmental box girders, many hundreds of which have been built around the world?

One objective of this paper is to answer these questions. A second objective is to exploit the available long-time bridge deflection measurements to improve the material model for creep. This should benefit not only the design of prestressed box girders, but also the design of other creep-sensitive structures, such as cable-stayed concrete bridges, nuclear containments, large arches and roof shells, and especially the super-tall, high-strength concrete buildings for which no long-time experience yet exists.

It should be noted that not all large-span, prestressed segmental box girders deflected excessively, even though a realistic creep model was unavailable in design (one noteworthy example is the 140 m [460 ft] span Pine Valley Creek Bridge in California, which was built in 1975). Various conservative and cautionary design measures explain why the deflections need not become excessive—six of them are listed at the end of Reference 1. Compared to designs based on a realistic creep model, however, these measures may severely limit the maximum feasible span, prevent elegant slenderness, and be uneconomical.

RESEARCH SIGNIFICANCE

Awareness of the excessive multi-decade creep deflections and their poor predictions by standard design recommen-

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dations will help the development and adoption of improved creep prediction models.

NEED TO CIRCUMVENT SHORT-TIME BIAS OF LABORATORY DATABASE

Currently, the largest existing laboratory database available for the calibration of creep and shrinkage models is the database of the Infrastructure Technology Institute at Northwestern University (refer to Reference 11). It is an enlargement of the previous RILEM 1993 database,[12] which was an enlargement of the original Northwestern University database compiled in 1978.[13] This database includes approximately 680 test curves and approximately 12,000 data points for creep.

Only 8% of the existing database[14] includes creep curves for durations of more than 6 years and only 5% includes creep curves for durations of more than 12 years. The only available data with durations exceeding 12 years are the aforementioned 30-year tests of Brooks,[5] the 18-year tests of Burg and Orst[15] and Russell and Larson,[16] the 12-year tests of Browne and Bamforth[17] and the Bureau of Reclamation,[18,19] and the 23-year tests of Troxell et al.[20] The last two, however, were made on 1930s concretes, which are of lesser relevance today. Brooks's data,[5] which represent only 3% of the database creep curves, are thus the only multi-decade source for modern concretes. The scope of the existing long-time tests is quite limited in terms of concrete types, specimen thicknesses, environmental humidities, and ages at loading.

Therefore, the only possible way to fill the data gap is to identify concrete creep properties by the inverse analysis of multi-decade deflections of structures. Large-span, segmental box girders are best suited for this purpose because many such structures have been built, are old enough, are highly sensitive to creep, and are dominated by self-weight. The recent super-tall concrete buildings may also be highly sensitive to creep and shrinkage because the columns may shorten unequally; however, they are currently generally not old enough yet to judge the long-time deformations.

COLLECTION OF EXCESSIVE DEFLECTION HISTORIES OF 56 LARGE BRIDGE SPANS: WAKE-UP CALL

Prompted by the release of data on the KB Bridge in Palau and their analysis, an effort to collect data on other bridges has begun in the Infrastructure Technology Institute of Northwestern University in collaboration with the recently established RILEM Committee TC-MDC. Private communications from some construction firms and consultants and the scanning of various papers and reports[21-27] led to a collection of histories of excessive deflections of 56 bridge spans, as shown in Fig. 2 (by the time of proof, 66 spans). It is likely that hundreds of such cases exist around the world. All the bridges in Fig. 2, except one (the Gladesville Arch), are large-span, segmental prestressed box girders, mostly with midspan hinges; however, at least six of them (the Parrots Ferry, Grubbenvorst, Wessem, Empel, Hetern, and Ravenstein Bridges) are continuous. The elimination of a midspan hinge reduces deflection, but often not enough, as documented in detail by the Labe Bridge in Děčín. Many of the listed sources on bridge deflections provide only sketches and limited cross-sectional information that does not suffice for FE modeling.

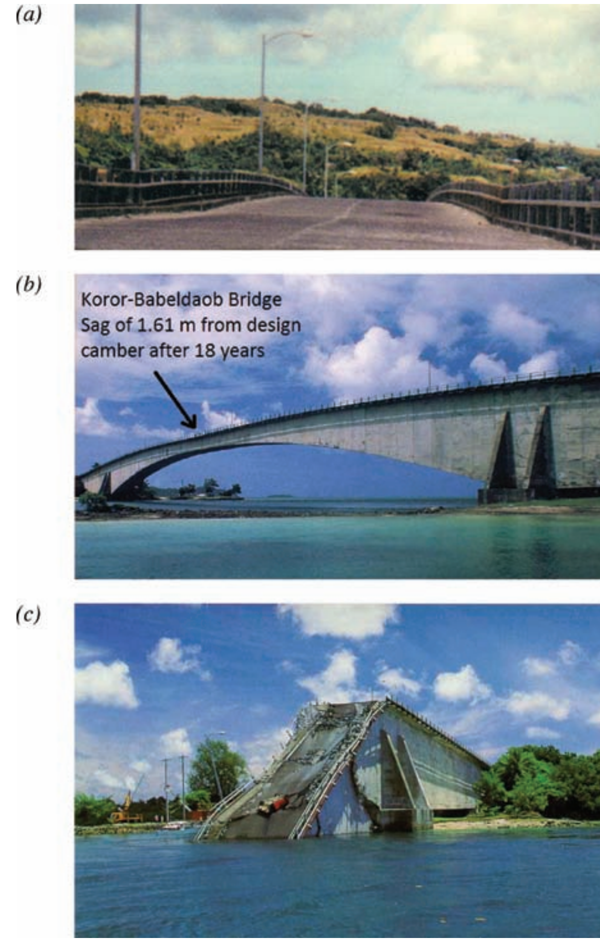

Fig. 1—(a) ACI authorized reprint from cover of ACI SP-194[4] *taken by Adam Neville before retrofit showing deck view of KB Bridge at midspan hinge before retrofit; (b) side view of KB Bridge before retrofit (main span: 241 m [790.7 ft] with two symmetric cantilevers consisting of 25 cast-in-place segments); and (c) collapse of KB Bridge on September 26, 1996—3 months after retrofit with remedial prestressing. (Note: 1 m = 3.28 ft.)*

What is most interesting is that all of these deflection histories terminate with a straight or nearly straight line in the logarithmic time scale, which corresponds to a logarithmic curve in the actual time scale. This feature, which was introduced in 1975 in an analysis of nuclear containment (refer to Fig. 4 in Reference 28) on the basis of L'Hermite et al.'s[29,30] and L'Hermite and Mamillan's[31] test data, agrees with the prediction of the theoretically based Model B3[3,4] and is also supported by other existing long-time laboratory

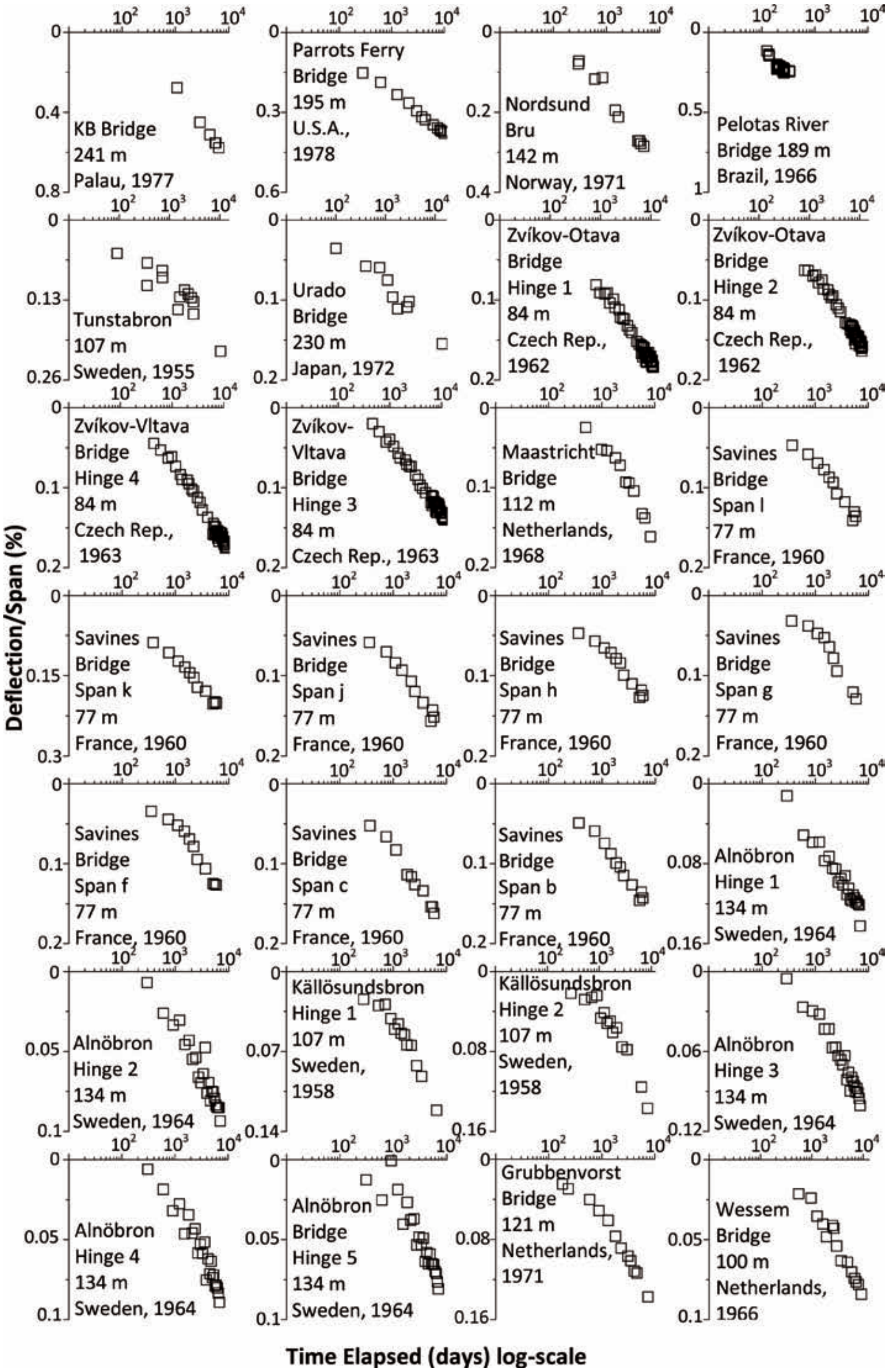

Fig. 2 (first part)—Histories of excessive deflections of 56 large bridge spans. (Note: 1 m = 3.28 ft.)

tests[5,15-20,32,33] (for example, refer to Fig. 2.2, 2.7, 2.10, 2.24, and 2.28 in Reference 34 or Fig. 1 through 4 in Part 2 and Fig. 1, 3, and 4 in Part 2 of Reference 35). Note that, similar to the aforementioned laboratory creep tests, there is no sign of the deflection curves approaching a finite bound.

By contrast, the existing creep prediction models of engineering societies, including the ACI Committee 209, CEB-*fib*, and GL models[2,6-8,10,36] (as well as the Japanese JSCE and JRA models[37,38]) have a form that implies a horizontal asymptote or a finite upper bound on creep. This erroneous assumption has doubtlessly been caused by the habit in most of the engineering literature to plot the creep curves only in the actual time scale and with an elongated time axis. When plotted that way, even the logarithmic curve gives an illusion of approaching a bound, although none exists.

The horizontal dashed lines in Fig. 3 represent the deflection equal to 1/800 of the span, which is considered to be the acceptable limit in bridge design specifications.[39] This limit is exceeded within the time range of available measurements by 16 of the 36 studied bridges and 26 of the 36 bridges based on the straight-line extrapolations to 100 years, which is the generally required design lifetime (note that 36 bridges were analyzed but only 35 could be shown in the figure). Based

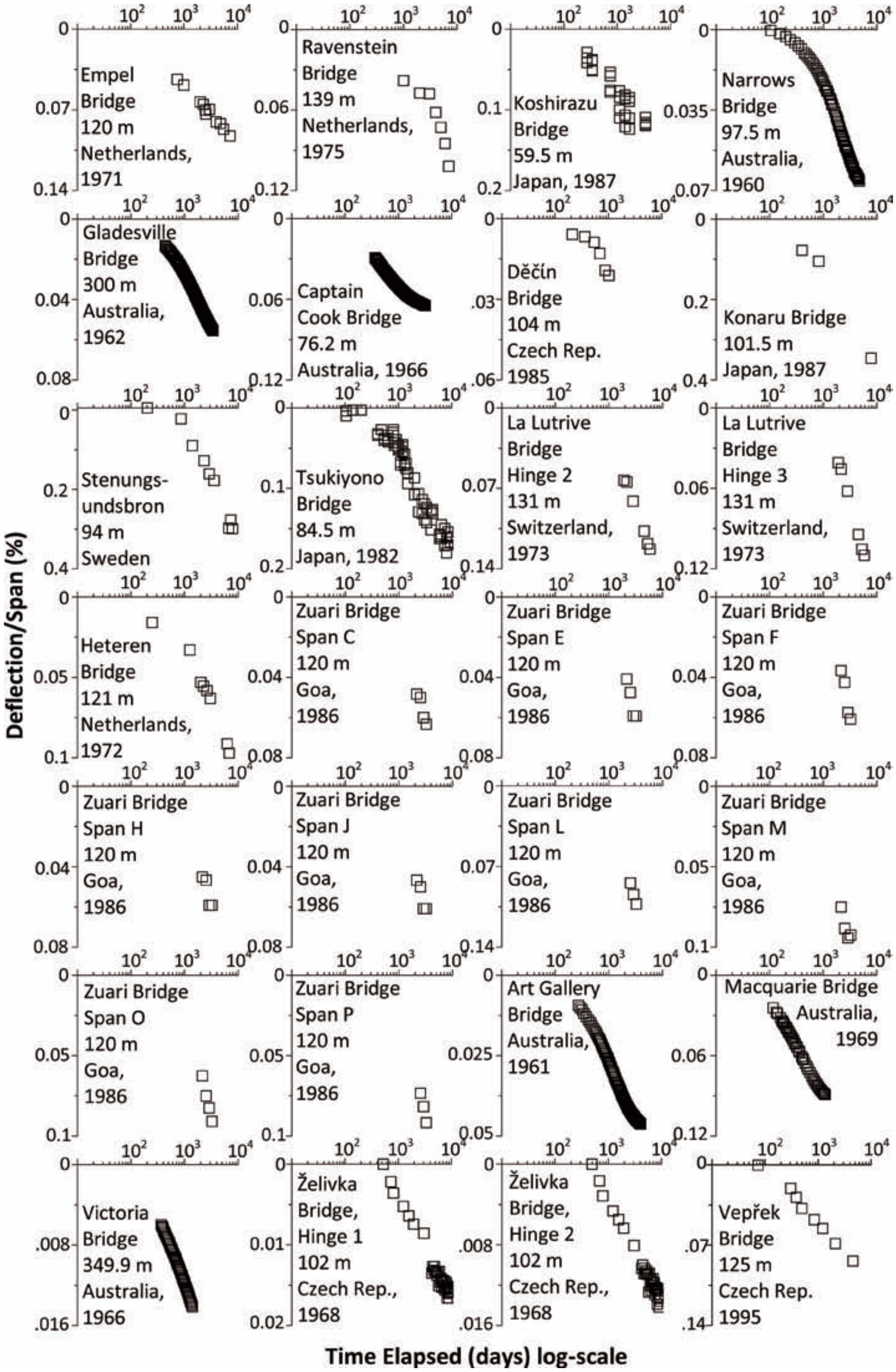

Fig. 2 (cont.)—Histories of excessive deflections of 56 large bridge spans. (Note: 1 m = 3.28 ft.)

on the data in Fig. 2 and their straight-line extrapolations, the limit of 1/800 is exceeded by 36 spans within 24 years, 39 spans within 40 years, and 50 spans within 100 years among the 56 spans in Fig. 2.

APPROXIMATE MULTI-DECADE EXTRAPOLATION OF MEDIUM-TERM DEFLECTION

The creep properties of concrete are characterized by the compliance function $J(t, t')$, which represents the strain at time t caused by a sustained unit uniaxial stress applied at age t' (refer to Reference 40 for an example). According to Model B3[28] and the preceding 1991 BPKX model[13,35] developed at Northwestern University, the long-time asymptote of the compliance curve at fixed t' is logarithmic. This feature is supported by the aforementioned laboratory data. The time at which the creep curve becomes a straight line in the logarithmic time scale depends on many factors—on average, it is approximately 3 years.

It is interesting that, after several years, the bridge deflection curves also become straight lines in the logarithmic time scale (Fig. 2). The reason for this must be that the effects of the age differences between segments, the variation of the self-weight bending moment during cantilever construction, the differences in the slab thicknesses, and the change of the structural system at span closing nearly die out. Also, the transient processes, particularly the drying effect on

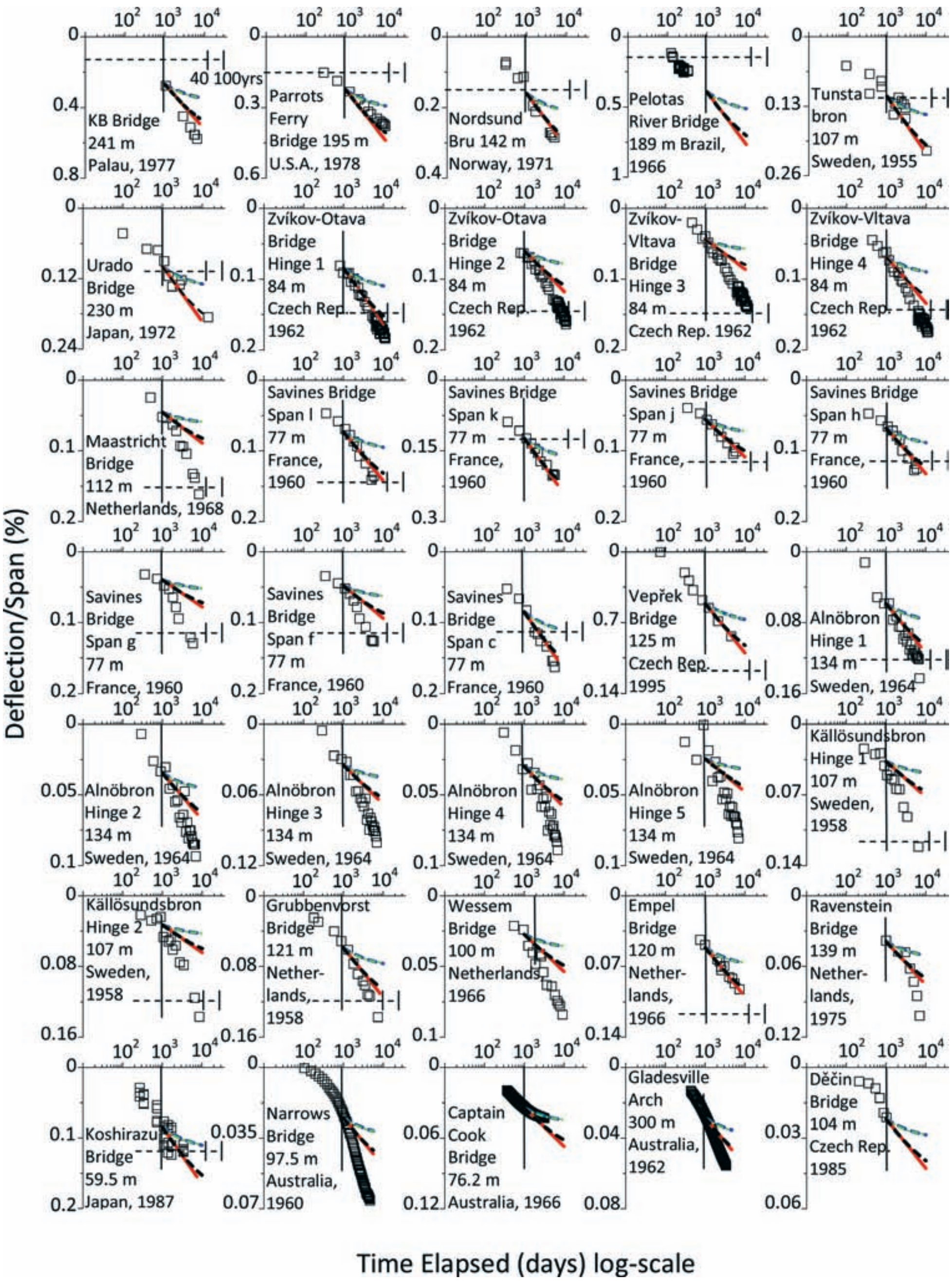

Fig. 3—Extrapolations of creep data for 35 bridges based on Eq. (1) using estimated average strength and composition of concrete. (Note: 1 m = 3.28 ft.)

creep and shrinkage, the gradual filling of capillary pores by cement hydration products, the acceleration of creep by drying, and the prestressing steel relaxation rate, greatly attenuate within a few years.

At earlier times, the drying effects greatly distort the deflection curve. Because the top slab of a segmental box girder is much thinner near the support than the bottom slab, its shrinkage and drying creep become accelerated. This reduces the midspan deflection and may even cause a temporary upward deflection.[41] Further complications of the short-time deflection history are caused by the gradual rise of the bending moment at the pier during the segmental erection and the age differences among the segments of the box girder. Therefore, the prediction of deflections during the first few years requires sophisticated FE creep analysis.[1,42,43]

Nevertheless, the straight-line trends of long-time deflections in the logarithmic scale suggest that if the deflection w_m (deflection at time t_m) at a certain medium time, such as t_m = 1000 days, is known, it could be simply extrapolated to long times by assuming similarity to $J(t, t')$. To keep the extrapolation easy, two simplifications of the regime prior to span closing need to be introduced: 1) the age differences among the box girder segments must be ignored and the age of the concrete must be characterized by one common effective (or average) age t_c at the span closing; and 2) instead of the gradual increase of the bending moment in the cantilever segments during the erection, one common effective (or average) age t_a at which the self-weight bending moments are introduced in the erected cantilever must be considered. In the following, the values t_c = 120 days and t_a = 60 days are considered for all the bridges.

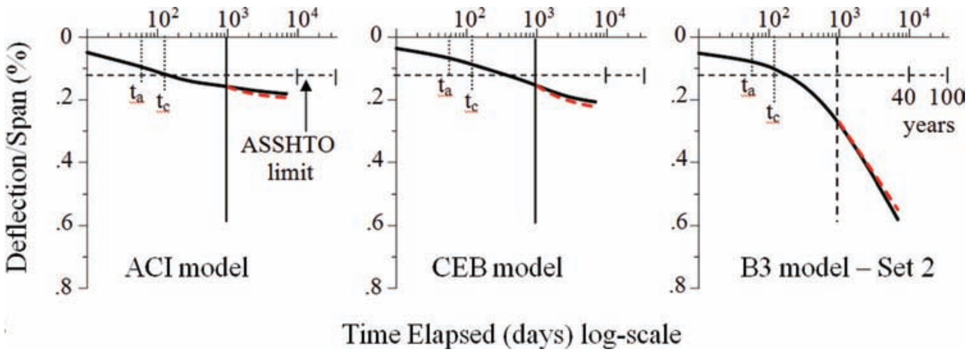

Fig. 4—Extrapolations of 1000-day deflections of KB Bridge by Eq. (1) based on concrete strength composition (dashed lines), documenting a good agreement with KB Bridge deflections accurately calculated in Reference 1 by FE creep analysis.

Because of these simplifications and the complexity of the drying and hydration processes in the early years, the long-time deflections cannot be assumed to grow in proportion to $J(t, t_a)$. Nevertheless, for the additional deflection w that develops after the span closing time t_c, the errors in approximating the early loading history by t_a and t_c must decay with time and eventually become negligible when $t >> t_c$—that is, after the lapse of a sufficient time t_m. As shown in the following, the aforementioned time t_m = 1000 days (measured from span closing) seems appropriate.

Before the span closing and for a few years afterward, the drying process and the differences in concrete age make the box girder response very complicated. After these effects nearly die out, however—that is, for $t > t_m$—the box girder begins to behave as a nearly homogeneous structure, for which the growth of deflection w should be approximately proportional to the increment of the compliance function that has developed since the closing time t_c—that is, $w = C[J(t, t_a) - J(t_c, t_a)]$, where C is a certain stiffness constant.

The values of C or w_m can vary widely and their calculation would require a detailed FE analysis, considering creep with drying and the construction sequence. Unfortunately, for most of the bridge deflection curves in Fig. 2, it turned out to be impossible to obtain the data necessary to calculate C from the material properties, geometry, and construction sequence. Therefore, only the extrapolation from time t_m can be examined, assuming that w_m is known.

Therefore, C can be calibrated experimentally from w_m using $C = w_m/[J(t_m, t_a) - J(t_c, t_a)]$. For the extrapolation of deflection after time t_m, the following approximate formula can thus be obtained

$$w(t) = w_m \frac{J(t,t_a) - J(t_c,t_a)}{J(t_m,t_a) - J(t_c,t_a)} \qquad (1)$$

To check how good this formula is, the deflection curves accurately calculated by FEs for the KB Bridge using the B3, ACI Committee 209, and CEB-*fib* material models can be used to an advantage.[1,42,43] For each curve, Eq. (1) can be used to extrapolate w_m at 1000 days from the computed deflection using the same compliance function $J(t, t')$ as that from which the curve was computed. The resulting extrapolations are shown in Fig. 3. It is astonishing how close each extrapolation is to the computed curve for the corresponding model; therefore, it makes sense to compare the extrapolations according to this formula to the observed long-time deflection curves of various bridges.

In theory, Eq. (1) should be applied only if the bending moments caused in the girder after time t_a by the self-weight and the prestress are approximately constant. Because the additional prestress loss after time t_m = 1000 days is very small, assuming the constancy of the bending moments should be a very good approximation for bridges with a midspan hinge. For a segmental bridge that was made continuous through the midspan, the internal forces redistribute so as to approach the moment distribution for an elastic continuous bridge. This redistribution after time t_m could be taken into account by generalizing Eq. (1) according to the age-adjusted effective modulus method (refer to References 40 and 44 for examples). However, complete information on the bridge geometry and prestress would be needed for this purpose. It is, unfortunately, unavailable for most of the bridges with no hinge in Fig. 2, except the Děčín and Vepřek Bridges. Even for these two bridges, however, the degree of redistribution after 1000 days must have been very small, which can be explained by the relative shallowness and flexibility of the cross section at the midspan.

COMPARISON OF DEFLECTIONS EXTRAPOLATED USING MODEL B3, ACI COMMITTEE 209, CEB-*fib*, AND GL COMPLIANCE FUNCTIONS

The input characteristics required by all the creep prediction models are the average compressive strength of concrete $\bar{f}_c$, the environmental relative humidity H, and the effective cross-sectional thickness D. In addition, RILEM Model B3 uses the water-cement ratio (w/c), the specific cement content c, and the aggregate-cement ratio (a/c) (the a/c value is implied by the specific weight ρ of concrete) as input, and if these additional input values are unknown, the recommended default values are used. Although the drying creep term of Model B3 has little effect on the deflection rate after time t_c, it affects the creep from t_o to t_c; therefore, it must be included in calculating $J(t, t')$ from Model B3. Because Eq. (1) cannot take into account the effect of the variation of the slab thickness within box girders, an approximation in which a single average or effective thickness D is used to calculate $J(t, t')$ must be used ($D = 2v/s$, where v/s is the volume-surface ratio of an average cross section).

To apply Eq. (1), the mean concrete strength $\bar{f}_c$ and w/c, c, and ρ for Model B3 must be specified. Unfortunately, these parameters are known for only six bridges among the 36 that were analyzed. Therefore, individual comparisons for each bridge are impossible. Nevertheless, a useful comparison, at least in the mean sense for all the bridges combined, can be made.

It is assumed that the concrete design strength in these older bridges was, on average, 31 MPa (4500 psi), which implies (according to CEB-*fib*)[8] that the mean strength was at least 39 MPa (5660 psi). Furthermore, the average effective cross-sectional thickness of $D = 0.25$ m (10 in.) and the environmental humidity of 70% for the Scandinavian bridges (the Norsund Bru, Tunstabron, and Alnöbron Bridges) and 65% for the other bridges is assumed. For the other parameters, it is assumed that $w/c = 0.5$, $c = 400$ kg/m^3 (25 lb/ft^3), and $\rho = 2300$ kg/m^3 (143 lb/ft^3). Of course, the deflection curve extrapolated in this way from w_m will likely be incorrect for each particular bridge. Nevertheless, because the errors should be of alternating signs, compensating each other, the mean of the extrapolations for all the bridges should still be approximately equal to the mean of the correct extrapolated long-term trend of the deflection curve that would be obtained if the properties of each individual concrete were known.

The last 19 of the 56 bridge spans in Fig. 2 (counted from the bottom) were omitted from the extrapolation exercise for three reasons: 1) not enough measurements were made; 2) the deflections were not too excessive; and 3) the straight-line regime has not yet been entered at 1000 days, which means that the drying effects still continued for the Konaru, Stenungsbron, Tsukiyono, Želivka, and Victoria Bridges. Moreover, one more figure had to be omitted to obtain in Fig. 2 a rectangular array, and it was the Savines Bridge Span b because its plot is essentially identical to Span c. This reduced the number of extrapolations according to Eq. (1) to 36.

The extrapolations obtained with the B3, ACI Committee 209, CEB-*fib*, and GL models are shown in Fig. 4 by lines—continuous lines for Model B3; light, dashed lines for the ACI Committee 209 model; dash-dot lines for the CEB-*fib* model; and dark, dashed lines for the GL model. None of these models is considered satisfactory because they all systematically and significantly underestimate the measured long-time deflections. Nevertheless, RILEM Model B3 does not perform as poorly as the others.

UPDATING LONG-TIME PREDICTION CAPABILITY OF RILEM MODEL B3

RILEM Model B3 has two important advantages:

- The long-time form of Model B3 is a logarithmic curve (shown as a straight line in the figures), which agrees with the long-time trend of the deflection data, whereas the long-time curves for the ACI Committee 209, CEB-*fib*, and GL models[6-10] (as well as the JSCE and JRA models[37,38]) level off as they approach a horizontal asymptote; and
- Model B3 is the only model that can be updated without compromising the short-time performance because the slope of the straight long-time asymptote can be separately controlled.

From Fig. 3, one can determine for each bridge span i (i = 1, 2, …N, N = 36) the ratio of the actual observed terminal slope r_i to the deflection slope extrapolated with Model B3. The mean ratio

$$\bar{r} = \sum_{i=1}^{N} r_i \,/\, N \tag{2}$$

may then be applied to modify Model B3 such that it would not systematically underestimate the long-time extrapolation of creep deflections.

According to RILEM Model B3 as described in References 2, 6, and 40, the terminal asymptotic deflection slope in the log($t - t_c$) scale is proportional to $q_4 + nq_3$, where n is an exponent of the viscoelastic term equal to 0.1; and q_3 and q_4 are Model B3 parameters obtained from empirical formulas as a function of w/c, c, a/c, and mean concrete strength $\bar{f}_c$, for which the aforementioned default values are used.

The parameter values resulting from these formulas are now proposed to be updated by factor $\bar{r}$, yielding corrected parameters

$$q_3 \leftarrow \bar{r}q_3,\; q_4 \leftarrow \bar{r}q_4 \;\text{ with } \bar{r} = 1.6 \tag{3}$$

The coefficient of variation of $\bar{r}$ is $w_r = 0.45\%$, but only the average value of $\bar{r}$ can be considered to be realistic because the same mean properties had to be assumed for all the bridges.

Figure 5 compares the lines of the corrected extrapolations with the terminal series of deflection data points. Note that the extrapolation errors are significantly reduced and that the deviations from the measurements now lie nearly equally below and above the measured data point series. An improved Model B3 is thus obtained. The other parameters of Model B3 have no effect on the long-time bridge deflection slope and thus cannot be improved in this way.

It would hardly be possible to obtain such an improvement of long-time performance by model calibration with the laboratory database alone. Because the database is biased toward short creep durations,[13] large changes in q_3 and q_4 cause only a very small change in the sum of squared deviation from the laboratory data. This causes high uncertainty in the q_3 and q_4 values obtained solely by minimizing the database errors.

CONCLUSIONS

1. The current empirical creep prediction models of ACI Committee 209, CEB-*fib*, and GL lead to a gross underestimation of multi-decade creep deflections. These models give an incorrect shape of the long-time creep curves and incorrectly imply the existence of a final creep value. A fundamental revision of all the engineering society recommendations is inevitable.

2. Beginning at approximately 1000 days after span closing, the segmental bridge deflections are approximately proportional to the increment of the compliance function from the moment of span closing, as described by Eq. (1).

3. With parameters predicted by the empirical formulae from the strength and composition of concrete, the theoretically based Model B3, which became a standard recommendation of RILEM in 1995, also underestimates the multi-decade deflections. The underestimation is not as severe, however, and the logarithmic shape of the terminal portion of the creep deflection curve perfectly agrees with the observations of large-span segmental bridges.

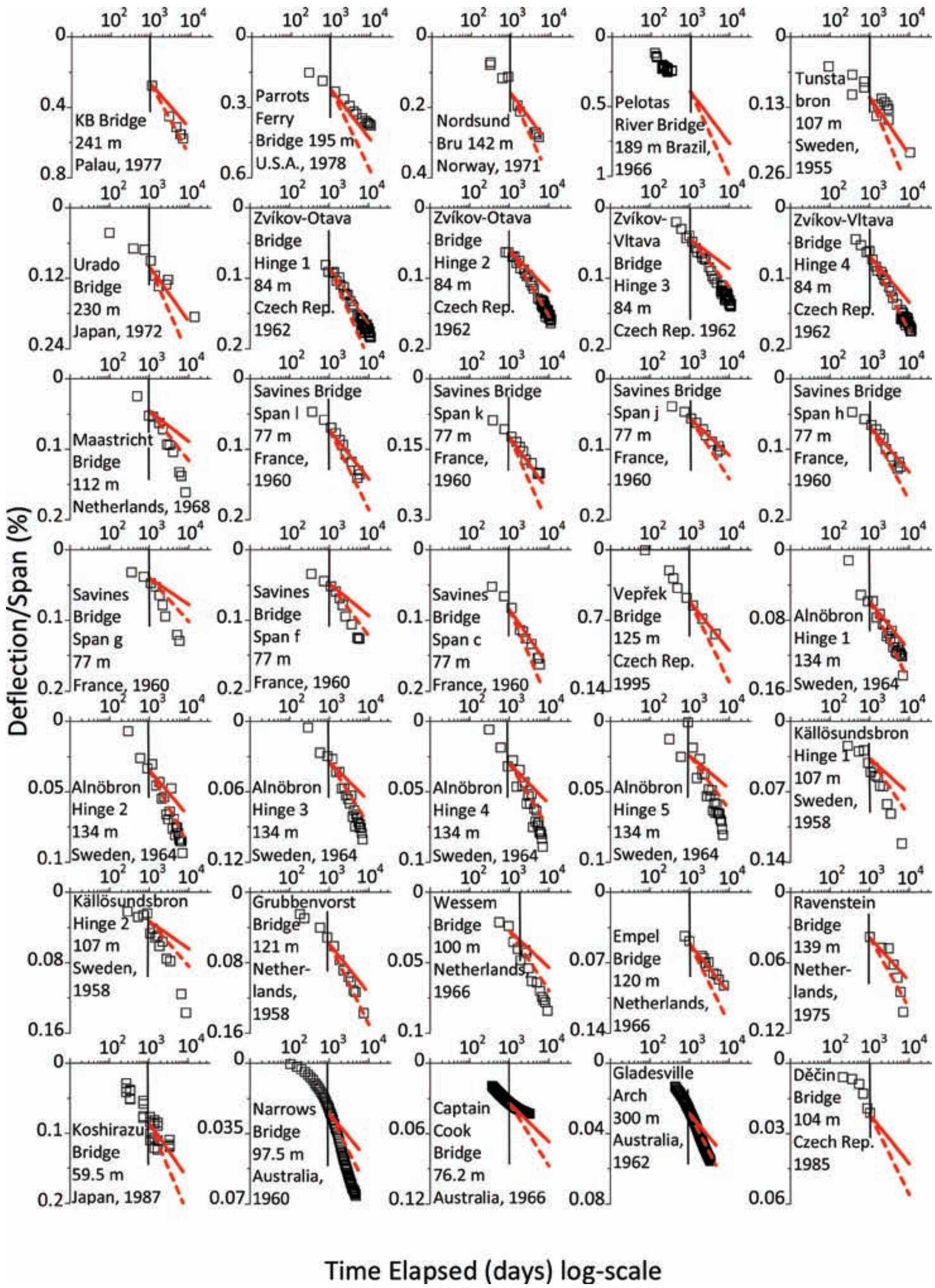

Fig. 5—Extrapolations (solid lines) of deflections of 36 bridges based on original Model B3 and its update with long-term factor $\bar{r}$ = 1.6 (dashed lines). (Note: 1 m = 3.28 ft.)

4. The aforementioned finding makes it possible to update a creep prediction model, provided that its parameters controlling the long-time creep rate are separate from the other model parameters. This is the way Model B3 is designed. A multiplier of these parameters identified from the long-time deflection trends of 36 bridge spans greatly improves the long-time predictions based on the updated Model B3.

5. Excessive long-time deflections of large-span, prestressed, segmental box girders are far more prevalent than previously thought. While a lifetime well in excess of 100 years is generally required in design, many of these bridges develop excessive deflections within 20 to 40 years. This may in turn cause cracking with corrosion, drainage problems, excessive vibrations, and car passenger discomfort. It may require either a bridge demolition or a retrofit with additional prestressing, which is a risky undertaking that may lead to a delayed collapse with fatalities (this is what did happen in Palau).

6. The prestressed box-girder bridges erected segmentally by the cantilever method are highly efficient and elegant structures. The present findings do not mean that they should be abandoned in new designs. Aside from updating the standard creep recommendations, the six deflection-mitigating design measures listed at the end of Reference 1 and in more detail in Reference 43 would have to be emphasized in new designs. Generally higher levels of prestress may be necessary such that the stress distribution produced in the cross sections by self-weight alone would be nearly uniform.

7. Although legal litigation stemming from deflection problems often blamed poor construction, the blame more likely rested on the standard design recommendations of engineering societies.

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