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Excessive Deflections of Record-Span Prestressed Box Girder

Lessons learned from the collapse of the Koror-Babeldaob Bridge in Palau

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Because full-size testing of very large structures is impossible, progress in structural engineering largely depends on drawing lessons from major disasters. Not every failed bridge provides a progress-stimulating lesson, but the Koror-Babeldaob (KB) Bridge in the Republic of Palau does. This bridge, built in 1977, connected the islands of Koror and Babeldaob (Fig. 1(a)). It had a main span of 241 m (790 ft), which once set the world record for a segmentally built prestressed concrete box girder.[1] After 18 years, the deflection became so excessive[2,3] that it was decided to install additional prestressing and eliminate the bridge's midspan hinge. The retrofit began on October 17, 1995, starting with removal of a concrete overlay. But on September 26, 1996, 3 months after the reopening, the bridge suddenly collapsed (with two fatalities) under negligible traffic load and with no apparent external trigger (Fig. 1(b)).[4-6]

In design, the final midspan deflection accumulated since the end of cantilever erection, measured from the design camber of –0.30 m (–12 in.), was expected to be in the tolerable range from 0.76 to 0.88 m (30 to 35 in.), which would have led to a final sag of only 0.46 to 0.58 m (18 to 23 in.) compared to the design elevation. Although these predictions[7,8] were based on the 1970-72 CEB-FIP model[9] for creep, the 1972 ACI model[10] (reapproved in 2008[11]) would have predicted about the same values.[5]

During the first 2 years, the deflections were benign but then accelerated unexpectedly. After 18 years, the deflection increase, measured since the installation of the midspan hinge, reached 1.39 m (55 in.) and kept growing[3] (Fig. 1(c)). Compared to the design camber, an additional creep deflection of 0.22 m (9 in.) was accumulated earlier during segmental erection, and so the total deflection was 1.61 m (5.3 ft).

As a result of litigation, the technical data collected on this disaster by the investigating agencies have been unavailable to the engineering community for many years. In light of this, the first author, acting in the name of a worldwide group of 47 experts, proposed at the Third Structural Engineers World Congress in Bangalore a resolution[12] that called, on the grounds of engineering ethics, for the release of all the technical data necessary for analyzing major structural collapses, including the bridge in Palau. The resolution (see the sidebar "Resolution of Third Structural Engineers World Congress") passed on November 6, 2007, and was widely circulated. In January 2008, the Attorney General of the Republic of Palau permitted the release of the technical data.

The present study (updating a previous preliminary report[13]) has a two-fold objective: 1) explain the excessive deflections, and 2) compare the predictions obtained with the main existing creep and shrinkage prediction models currently used in practice. Investigation of the collapse itself will be presented in a later article. Due to

The list of references is available with the online version of this article at www.concreteinternational.com

length limitations, the details of the creep and shrinkage analyses cannot be presented here, but they can be found in a recent report,[12] available at **www.civil.northwestern.edu/people/bazant/PDFs/Papers**.

MAIN CHARACTERISTICS OF THE BRIDGE

The KB Bridge comprised three spans. The main span consisted of two symmetric simultaneously erected cantilevers connected at midspan by a horizontally sliding hinge (Fig. 1(d)).[1,5] Each cantilever contained 25 cast-in-place segments of depths varying from 14.17 to 3.66 m (46.5 to 12 ft) (Fig. 2). The segmental erection took about 6 to 7 months. The two end spans were partially filled with rock ballast to balance the moments from the cantilevers.

The web thickness was 356 mm (14 in.)—unusually small compared to the girder depth. The thickness of the bottom slab varied from 1153 mm (45.4 in.) at the main piers to 178 mm (7 in.) at the midspan. The thickness of the top slab ranged from 432 mm (17 in.) at the main piers to 280 mm (11 in.) at the midspan. The top slab was covered by concrete overlay with an average thickness of 76 mm (3 in.).

The prestress was generated by 32 mm (1.25 in.) diameter threaded alloy bars with 1030 MPa (150 ksi) nominal tensile strength. The bars were extended by couplers, anchored by nuts, and grouted in 47.6 mm (1.9 in.) diameter ducts. The jacking force for each tendon was about 0.60 MN (135 kips). The horizontal, longitudinal force above the pier, provided by 316 densely packed tendons in four layers within the top slab, was about 190 MN (42,606 kips). Similar threaded bars were used to provide vertical prestress in the webs and horizontal transverse prestress in the top slab. The nonprestressed steel reinforcement was also taken into account in calculations. Despite the tropical marine environment, the post-collapse examination revealed no signs of significant corrosion of steel, prestressed or nonprestressed.

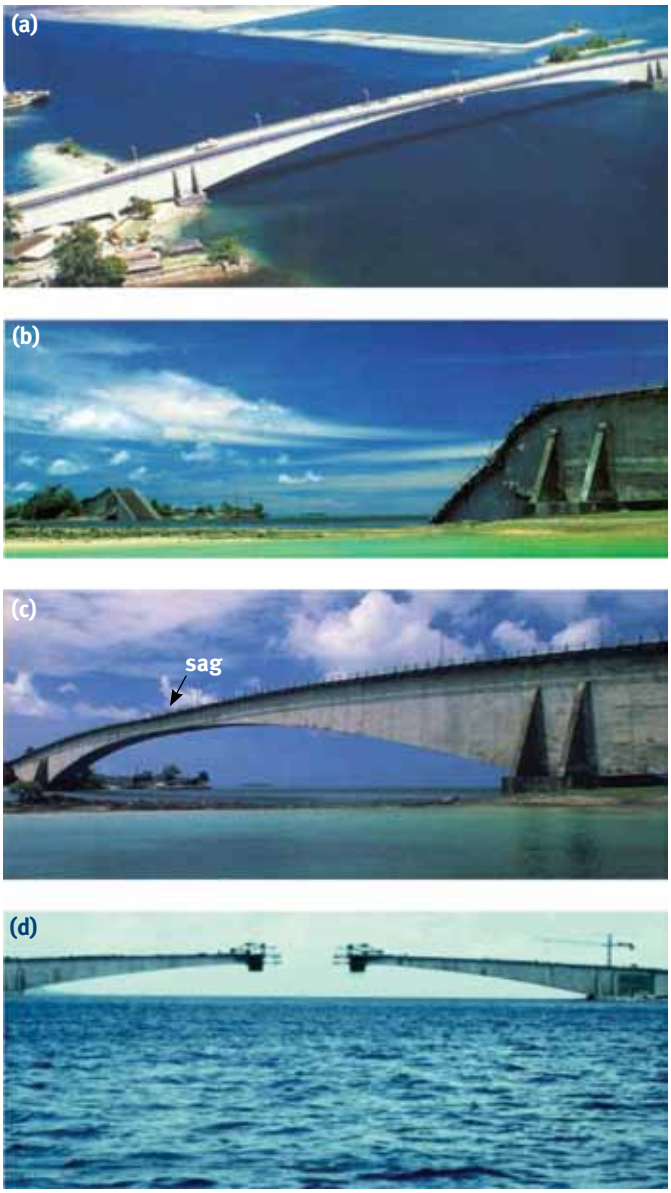

Fig. 1: (a) KB Bridge, connecting Koror and Babeldaob Islands in the Republic of Palau; (b) collapse of KB Bridge; (c) excessive deflection of KB Bridge; and (d) erection of the cantilever box girders of KB Bridge

CREEP AND SHRINKAGE MODELS AND METHOD OF ANALYSIS

For this study, creep and shrinkage were evaluated using the ACI,[11] CEB (CEB-FIP or *fib*),[14] JSCE,[15] GL,[16,17] and B3[18-20] models. The B3 model is the third and latest version of the theoretically based models developed at Northwestern University since 1978.[21,22] The bridge was analyzed using the commercial finite element code ABAQUS,[23] a three-dimensional finite element system,

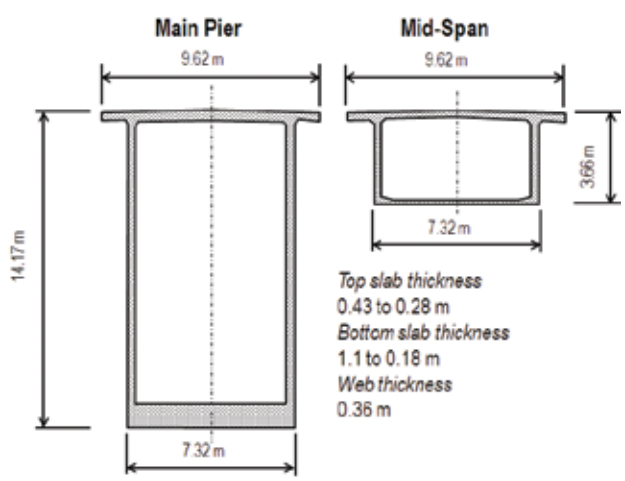

Fig. 2: Main span cross sections (1 m = 3.28 ft)

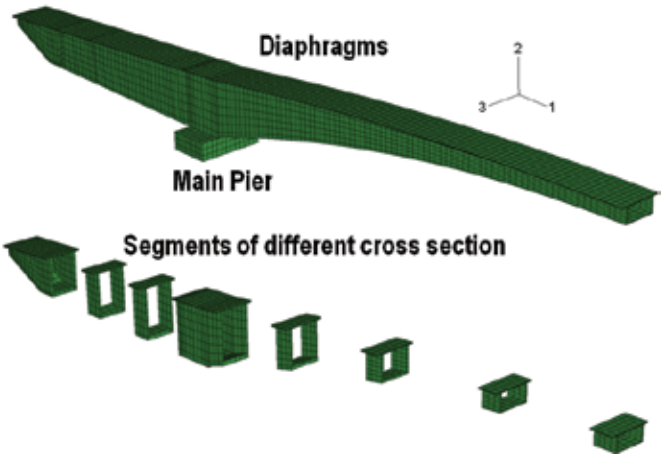

Fig. 3: Model of KB Bridge built in ABAQUS showing diaphragms and segments of the box girders.

with Kelvin-chain-based step-by-step integration in time. Because of symmetry, only one half of the bridge, subdivided into a mesh of 5036 eight-node hexahedral isoparametric elements for concrete and 6764 bar elements for prestressed and unprestressed steel bars, was analyzed (Fig. 3). Elastic analyses verified that a finer mesh would give no significant accuracy improvement.

The linear aging viscoelasticity underlying the creep analysis implies the principle of superposition in time and direct application gives the stress-strain relation in the form of a history integral. However, accurate and efficient creep analysis requires replacing the history integral by a rate-type creep law, which can be based on the Kelvin chain model. For the ACI, CEB, JSCE, and GL models, the set of elastic moduli of the Kelvin chain model, called the retardation spectrum, is age dependent.[24,25] Within each time step, though, the aging can be neglected, and so the spectrum can be considered as constant and identified for each time step separately.[26] For the B3 model, which is based on the solidification theory, the creep properties are described by the nonaging properties of the hardening constituent whose volume growth accounts for aging.[27] Hence, for this model, the spectrum of elastic moduli of the Kelvin chain units of the nonaging constituent is constant,[18] simplifying analysis.

The step-by-step integration algorithm reduces the creep problem to a series of elastic structural analyses with general inelastic strains, one for each time step. ABAQUS was called in each time step to carry out the elastic analysis. The tangential elastic moduli matrix, supplied in ABAQUS to each finite element integration point, was evaluated from the exponential algorithm for time integration, and was different for each integration point of each finite element in each time step.[25,28] The introduction of prestress of each tendon was simulated in ABAQUS by assigning the initial stress to the tendons. To capture the time sequence of segmental construction, some finite elements were initially inactive and were activated at the time of their casting, which also introduced a different age of concrete for each segment. The prestress losses due to steel relaxation were imposed in each time step; those due to sequential prestressing were automatically captured by ABAQUS. Tensile cracking in the top slab had to be taken into account because large tensile strains arose.

INPUT PARAMETERS

The input parameters required for creep and shrinkage analysis are: 1) the 28-day elastic modulus of concrete E_c or the design strength f_c'; 2) the age at start of drying, taken as t_0 = 7 days, which is the average segmental erection cycle; 3) the average environmental humidity h = 0.7; 4) the effective thickness of cross sections, $D = 2V/S$, where V/S = volume/surface ratio; 5) temperature T for the extended B3 model[20,25]; and 6) additional concrete composition parameters for the B3 model. In the absence of mixture proportion data, default or alternative values based on educated guesses can be assigned to each B3 parameter.[20] The ACI, CEB, and GL models use the standard 28-day compressive strength f_c' as the only input characteristic of concrete.

No report on E_c is available. Two independent investigating agencies[2,3] conducted core sample tests and reported E_c values of 22.1 GPa (3200 ksi) in 1990 and 21.7 GPa (3150 ksi) in 1995. Both are lower than the E_c of 28.3 GPa (4110 ksi) predicted using the specified f_c' and the ACI formula.[29] This discrepancy might be attributed to high porosity and random variations of local properties within the structure. Nevertheless, what matters for the deflections is the overall average E_c that can be evaluated from the deflection in a load test. In 1990, the bridge was loaded by two trucks at midspan.[2] The three-dimensional finite element analysis of this load test, followed by an adjustment for the increase of E_c with age according to the standard ACI formula, yields, for the age of 28 days, $E_c \approx 22.0$ GPa (3190 ksi). This is almost 23% less than predicted from f_c' by the ACI empirical formula, but is in close agreement with the core sample data.

Two sets of input parameters are considered for computations:

- Set 1 aimed to check how good the prediction is when only f_c' and, for B3, the default values of unknown mixture proportions, are used; and
- Set 2 aimed to check whether the observed deflection can be explained or, more specifically, whether there exist realistic input parameter values leading to a good fit of the observed deflections (realistic means that Set 2 must also give a good fit of the long-time creep tests of a similar concrete in the database;[30] otherwise, Set 2 calculations would explain nothing).

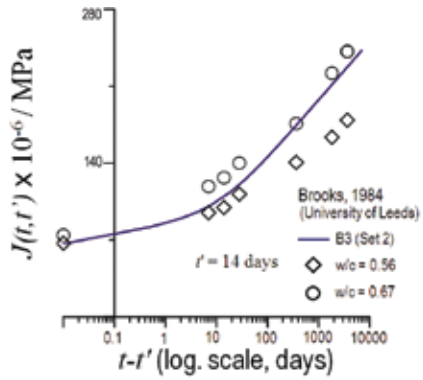

Fig. 4: B3 model using adjusted q_3 and q_4 compared with the creep tests by Brooks (1 MPa = 145 psi)

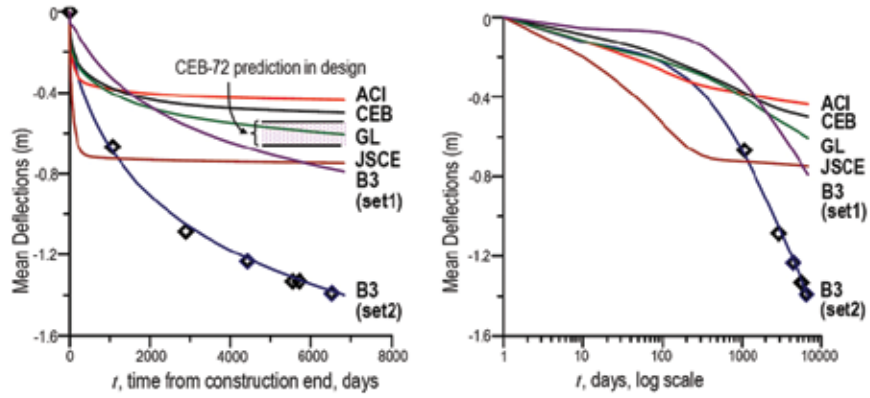

Fig. 5: Calculated mean deflections by B3, ACI, CEB, JSCE, and GL models, plotted using normal and logarithmic time scales (1 m = 3.28 ft)

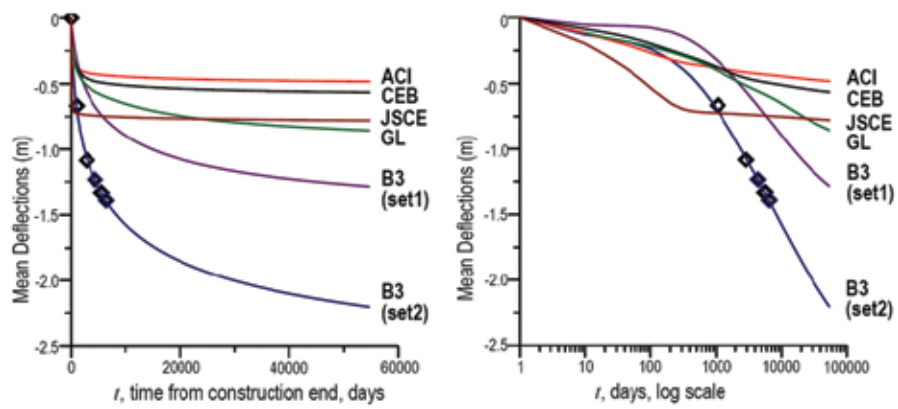

Fig. 6: Calculated mean deflections by B3, ACI, CEB, JSCE, and GL models, plotted using normal and logarithmic time scales and with time extended up to 150 years, assuming that no retrofit and no collapse have taken place (1 m = 3.28 ft)

In Set 1, the compressive strength at 28 days is estimated as f'_c = 35.9 MPa (5200 psi),[3,5,7] according to the design. Based on the ACI empirical formula, E_c = 28.3 GPa (4110 ksi) at 28 days. According to the concrete density discovered in core sample tests, the concrete mixture was assumed to have a cement content of 700 lb/yd^3 (415 kg/m^3), an aggregate-cement ratio of 4, and water-cement ratio (w/c) of 0.62. According to the B3 model, the corresponding Set 1 input parameters are q_1 = 0.146, q_2 = 1.04, q_3 = 0.045, q_4 = 0.053, q_5 =1.97 (all × 10^{-6}/psi); $\varepsilon_{k\infty}$ = 0.0013; and k_t = 19.2.

Set 2 parameters cannot be generated for the ACI, CEB, and GL models because their only free input parameter is the design strength f'_c, which is fixed. The aforementioned Set 1 parameters for B3 were adjusted for Set 2 as follows: q_1 changed from 0.146 to 0.188; q_3 changed from 0.045 to 0.262; and q_4 changed from 0.053 to 0.140 (all × 10^{-6}/psi). Here q_1 is adjusted according to an E_c deduced from the truck load test, and parameters q_3 and q_4, which affect mainly the long-time creep, are obtained by optimizing the fit of the measured deflections.

Subsequently, the corresponding compliance function $J(t, t')$ for Set 2 is generated (t = current age, t' = age at loading). Checking the database,[30] this $J(t, t')$ matches well the 10-year creep test data of Brooks[31] (Fig. 4) (later extended to 30 years[32]) and is also close to the 18-year data of Russell and Burg and the 23-year data of Troxell et al.[33] This agreement proves that Set 2 is realistic and that the excessive deflections are explicable.

SIMULATION RESULTS AND MODEL COMPARISONS

To ensure comparability, the computations for all five models were made with the same finite element model and the same step-by-step time integration algorithm. Figures 5 and 6 show, both in linear and logarithmic scales, the deflection curves computed for the five models. The measured deflections, reported by two independent investigating agencies, are plotted as the diamond points. As many bridges are now expected to last more than 100 years, Fig. 6 shows the same curves extended up to 150 years under the assumption that there has been no retrofit, in which case the bridge would not have collapsed.

Aside from deflection magnitudes, note the erroneous shapes of the deflection curves—particularly the errors in the final slope (the logarithmic scale is necessary to make these errors conspicuous, whereas linear scale plots, often favored by engineers, obfuscate these errors). These discrepancies indicate that something is fundamentally inadequate and

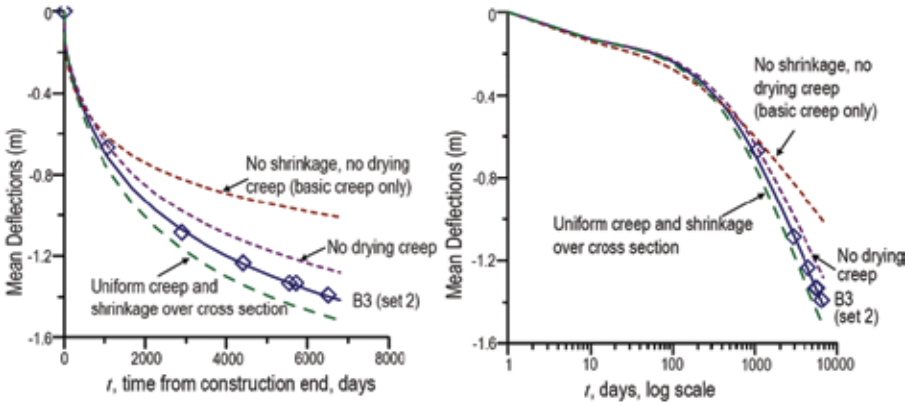

Fig. 7: Deflections predicted using the B3 model with uniform creep and shrinkage over the cross section, no drying creep, and no shrinkage or drying creep (1 m = 3.28 ft)

support previous criticism,[19,20,34,35] and they show that it is impossible to extrapolate short-time observations into the future. This aspect (which is discussed in more detail in recent studies[34,36-38]) is a significant problem for the ACI and CEB models, and especially the JSCE model, which, for concretes with a high w/c, happens to give higher long-time deflections than the ACI and CEB models but also provides the worst shape of the deflection curve.

Recent research[37] has revealed an extreme sensitivity of deflections to the differences in the rates of shrinkage and drying creep between the top and bottom slabs (also see Reference 34). Unlike the basic creep, the shrinkage and the additional creep due to drying are bounded. Their rates and half-times are proportional to the square of the effective thickness of each slab and to the diffusivity of concrete. Figure 7 clarifies these features by comparing the midspan deflection curves computed for three cases in which: 1) the drying creep is neglected; 2) both the shrinkage and drying creep are neglected; and 3) the creep and shrinkage properties are considered to be uniform over the cross section and are based on the effective thickness $D = 2V/S$ for the whole cross section.

Because of the big thickness difference between the top and bottom slabs in the cross section at the pier, moisture diffusion analysis indicates the shrinkage and drying creep of the top slab will proceed about nine times faster than the bottom slab. This becomes about 270 times faster if one also takes into account that, because of direct sunshine, the top slab is on average about 20°C (36°F) warmer than the bottom slab during the day,[39] and that concrete of the top slab and the pavement slab have a higher moisture diffusivity because of tensile cracking.[40]

Consequently, during the initial years, the faster shortening of the top slab pulls the midspan up, mitigating deflections. Later, though, after drying of the top slab has terminated, the bottom slab begins drying significantly, which in turn accelerates the midspan deflections. These phenomena, which are captured by the B3 model (as discussed in detail in Reference 37) and are apparent in the logarithmic time scale in Fig. 5 to 7, are crucial for the extrapolation of early deflection measurements.

Accuracy in predicting the prestress loss is particularly important because the bridge deflection is a small difference of two large but uncertain numbers representing downward and upward deflections respectively caused by the selfweight and the prestress.

The KB Bridge is unique in that the prestress loss in grouted tendons was measured. Once the retrofitting was agreed upon, three tendons were sacrificed. Three segments of each tendon were exposed and cut after strain gauges were placed. Tendon stress was determined by measuring the strain relief.

These strain relief tests showed an average prestress loss of about 50%, much higher than that normally assumed in prestressed concrete design (in this case, 22%). It is also much higher than the prestress losses obtained from the finite element analyses using the ACI, CEB, JSCE, and GL models (22, 24, 29 and 27%, respectively).

By contrast, for the B3 model, the calculated prestress loss is 40% for Set 1. For Set 2, it is 46%, close enough to the mean of measurements and well within their scatter range. A comparison of the results is seen in Fig. 8, in both the linear and logarithmic scales.

Creep and shrinkage are notorious for their relatively high random scatter. It is prudent to explore the range of responses by considering the likely scatter of all the influencing parameters, and then design the structure for the most unfavorable realistic combination. This can be done statistically, by generating a Latin hypercube table of the random influencing parameters according to their approximate statistical distributions and running the bridge analysis for each sample (or each row of the table).[41] Figure 9 shows the deflection ranges obtained in this way after eight deterministic runs based on the B3 model, Set 2. After calculating the mean response and the coefficient of variation from these eight runs, a table of the normal distribution can be used to determine the 95% confidence limit on the maximum deflection, on which the design should properly be based (for a more refined statistical prediction of deflections, see References 27, 42, and 43).

The three-dimensional simulations

reveal that the shear lag effect cannot be adequately captured by the classical concept, used in the design of the KB Bridge, of effective width of the top slab.[42] Shear lag occurs in several different ways—in the transmission of vertical shear force due to vertical reaction at the pier, in the transmission of the concentrated forces from tendon anchors, and for each of these in the horizontal slabs and in the vertical walls. Only full three-dimensional analysis can capture these behaviors. The use of one-dimensional beam-type analysis causes a large error in deflections as well as the prestress loss (see the curve for beam analysis in Fig. 10).

SIMILAR EXPERIENCE WITH OTHER BRIDGES

Is the KB Bridge an anomaly? No, it is not. Although very little has been reported in technical journals, rumors about excessive deflections of many large segmentally erected box girders abound. Revealing such data to allow analysis would benefit progress. If deflection data for many large bridges could be compiled, their inverse analyses, based on a computer program such as the one discussed here, could be used to devise a more realistic multidecade creep prediction model, which is essential for ensuring multidecade serviceability.

This goal is impossible to achieve on the basis of the existing database because the vast majority of laboratory creep tests were only 3 to 5 years in duration. Among more than 621 creep test series that have been conducted at many labs around the world,[44] only three test series give information for load durations greater than 10 years, and even those data are far from complete in terms of the ages at loading, humidities, specimen sizes, and water loss.

Y. Watanabe, the Chief Engineer of Shimizu Corp., Tokyo, Japan, graciously made available to us the data on excessive deflections on some large Japanese bridges that epitomize

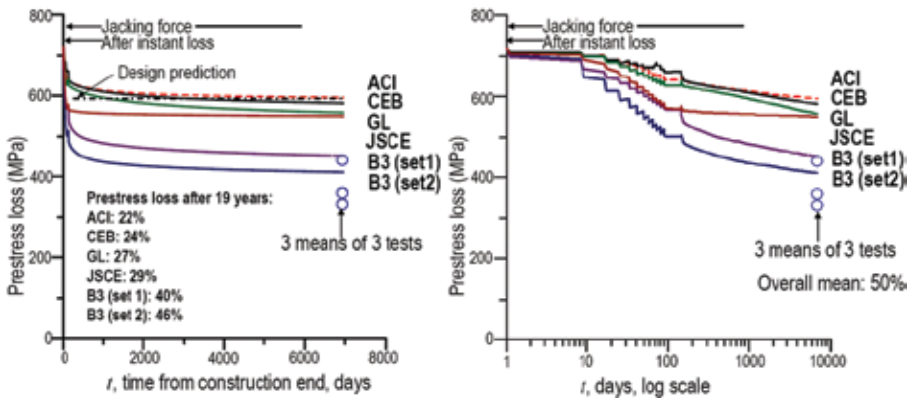

Fig. 8: Prestress loss in tendons at main pier, predicted using B3, ACI, CEB, JSCE, and GL models (1 MPa = 145 psi)

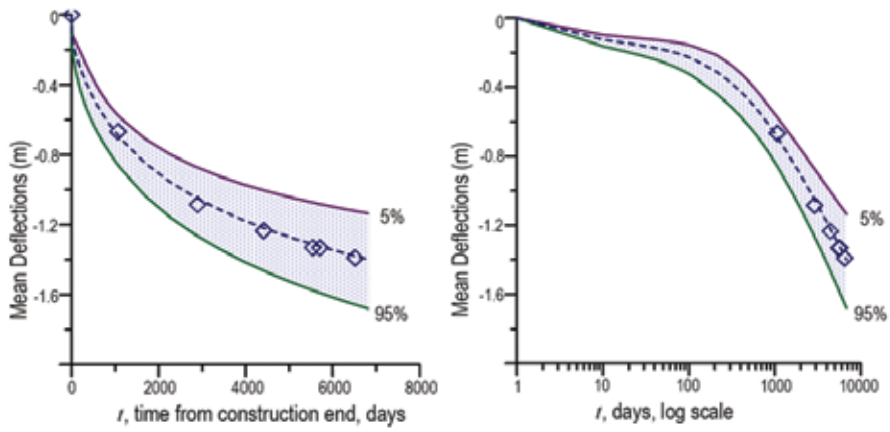

Fig. 9: Mean response and 95% confidence limits for B3 model (Set 2) (1 m = 3.28 ft)

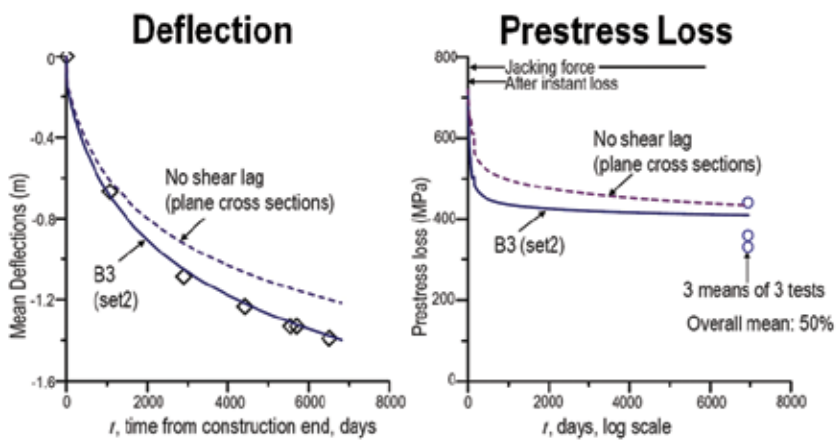

Fig. 10: Deflections and prestress loss obtained using three-dimensional analysis, compared with values found using bending theory with cross sections remaining plane (no shear lag) (1 m = 3.28 ft; 1 MPa = 145 psi)

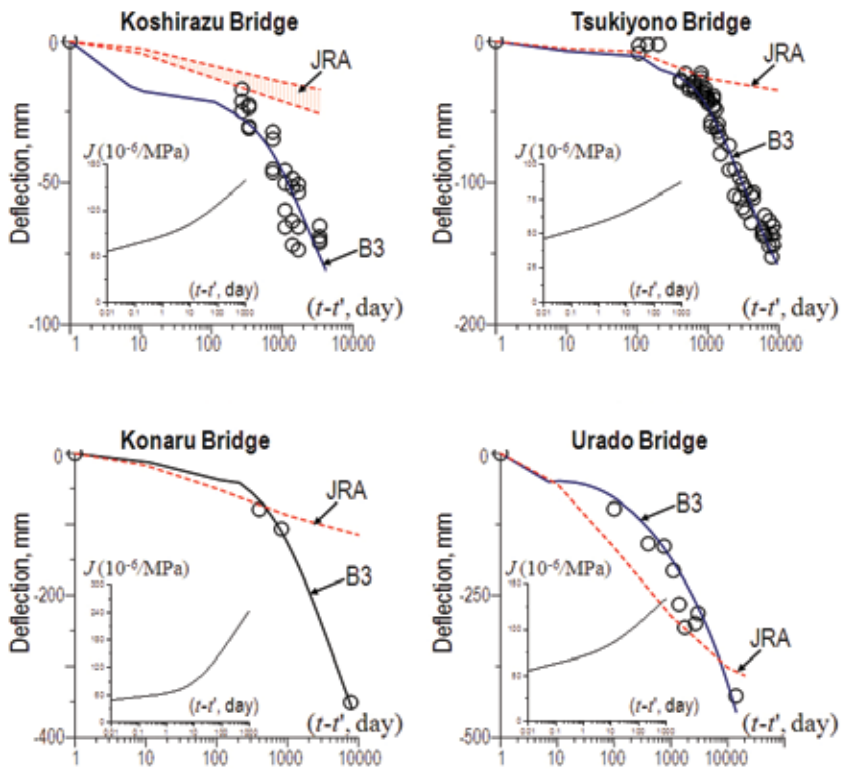

Fig. 11: Four examples of excessive deflection reported in Japan for prestressed cantilever box girders. Measured data are compared against predictions made using the B3 model and the Japan Road Association (JRA) specification (1 mm = 0.039 in.; 1 MPa = 145 psi)

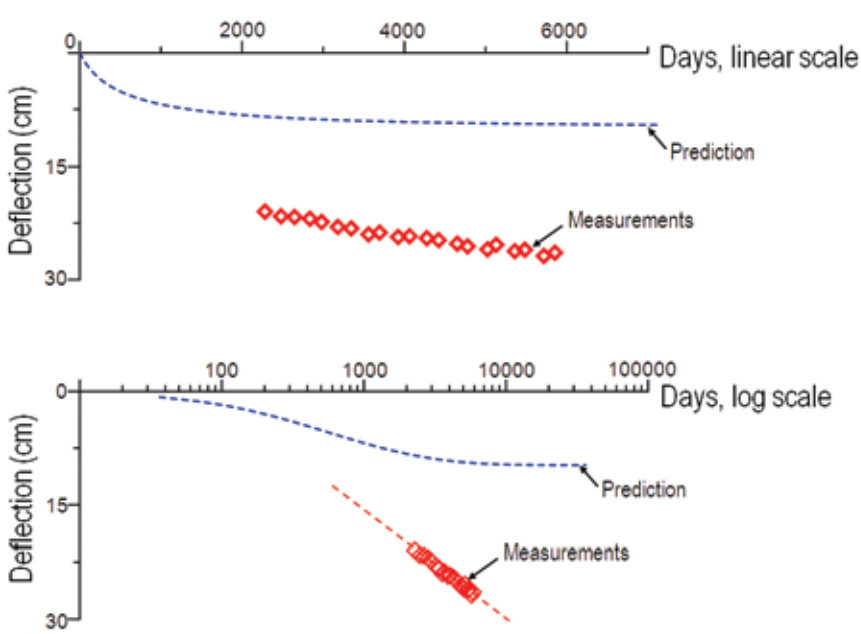

Fig. 12: Excessive deflection recorded in a prestressed box girder bridge in Děčín, compared against design prediction (1 cm = 0.39 in.)

the experience rumored in other countries.[45] Figure 11 shows the prediction according to the standard design recommendation (dashed curves), the B3 model prediction after adjustment of composition parameters (solid curves), and the data points representing the measured deflection. The plots in logarithmic time scale make similarity with the KB Bridge conspicuous. One reason that one of these bridges, Urado, does not show excessive 30-year deflection is that it was designed using the Japan Road Association (JRA) specification[46] in which the creep curve is set about 60% higher than in the JSCE code. However, the deflection slope at 30 years portends a problem for the future.

Although a continuous segmentally erected bridge might be more expensive to build and more difficult to analyze, the absence of a hinge has been known to reduce deflections. However, it is not a panacea. Even bridges without a midspan hinge can suffer excessive deflections. This is documented by the data on the Děčín Bridge over Elbe in North Bohemia (Fig. 12).[47]

LESSONS LEARNED

None of the existing creep and shrinkage prediction models are satisfactory as purely predictive tools, although the B3 model, which has a strong theoretical basis,[18-20,35] gives a significantly better long-time prediction than the others. After parameter adjustment according to test data, the B3 model fits the measured deflections closely.

It is thus imperative to improve the creep and shrinkage prediction models, especially the prediction of model parameters from concrete strength and composition. The inadequacy of the creep and shrinkage prediction from concrete strength and composition is the greatest source of error.

Second in terms of the error magnitude is the oversimplification

of creep structural analysis as currently used. The one-dimensional beam-type analysis leads to great errors in deflections as well as prestress loss. To capture shear lags in slabs and webs due to dead load and to prestressing tendons, box girders must be analyzed as three-dimensional shells (for wide box girders of multi-lane bridges, the error of one-dimensional analysis must be expected to be still much larger than demonstrated for the relatively narrow box girder in Palau).

The effects of the differences in slab thicknesses within the cross sections on the shrinkage and drying creep rates must be considered. This is particularly important for extrapolating short time observations. The temperature differences, tensile cracking, and concrete diffusivity (which governs the rates of shrinkage and drying creep), should also be included in the analysis.

The classical estimates of prestress loss are inadequate. The prestress loss must be computed as part of the overall three-dimensional time-step creep analysis of the structure.

For large bridges, short-time tests of creep and shrinkage should always be carried out. According to the procedure developed in recent studies,[19,20] even tests of 1 month's duration greatly improve the long-time prediction. The short-time tests of shrinkage and drying creep are helpful only if accompanied by measurements of water loss due to drying.

The statistical scatter of the predictions (which depends on the quality control) should be estimated, and the design should be based on the 95% confidence limits.

The experience in Palau underscores the old wisdom that it is prudent to adopt measures that minimize creep deflections and prestress losses. In particular: 1) use no midspan hinge; 2) choose a concrete with low creep, especially a low long-time creep; 3) use a deflection-reducing layout of tendons[38] and optimize it; 4) apply a higher level of prestress, preferably so high that an upward deflection is predicted; 5) give concrete more time to gain strength before prestressing; 6) use stiffer girders of low slenderness; and 7) install empty ducts into which additional tendons can be placed later.

THE PUBLIC INTEREST

The present lessons from the catastrophe in Palau could have come to light a decade earlier if the technical data were not sealed. In commercial aviation, concealment of technical data from disasters is prohibited by law as well as international treaties. While enactment of a similar law for structural engineering would be most desirable, it might be difficult to achieve. It should be easier for engineering societies to expand their ethics codes. Any concealment of technical data from a major failure or serviceability loss after the official investigations are completed should be labeled as a violation of engineering ethics. Often, of course, the legal settlement is reached without the participation of an engineer, but then the ethics code should require the engineer to inform the lawyers and the judge that data sealing would be against the public interest, which is what every judge is obliged to defend.

Acknowledgments

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Resolution of Third Structural Engineers World Congress

(1) The structural engineers gathered at their Third World Congress deplore the fact the technical data on the collapses of various large structures, including the Koror-Babeldaob Bridge in Palau, have been sealed as a result of legal litigation. (2) They believe that the release of all such data would likely lead to progress in structural engineering and possibly prevent further collapses of large concrete structures. (3) In the name of engineering ethics, they call for the immediate release of all such data. (Bangalore, November 6, 2007)

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