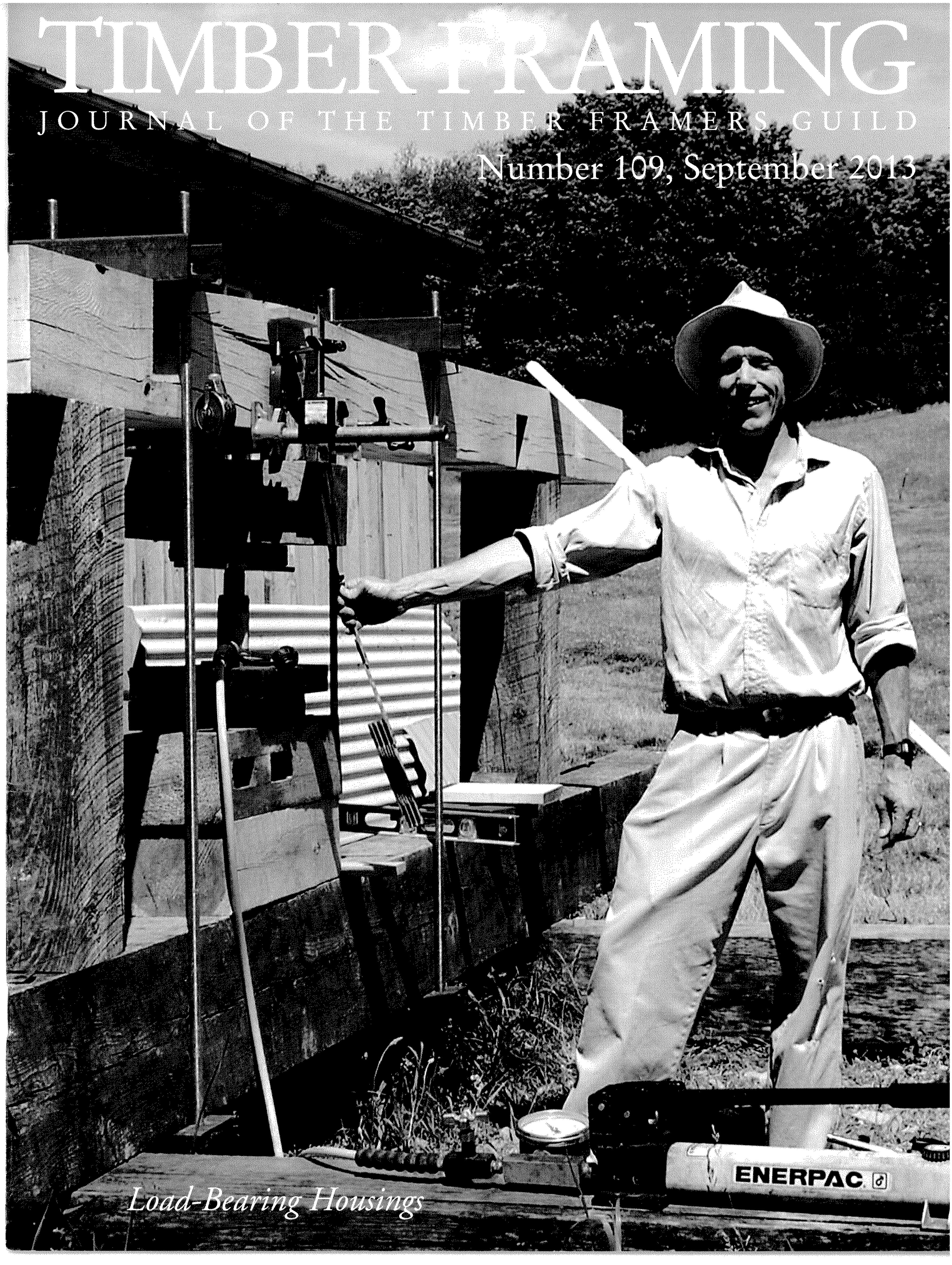
TIMBER FRAMING

JOURNAL OF THE TIMBER FRAMERS GUILD

Number 109, September 2013



Load-Bearing Housings

Reflections on Load Capacity of Historic Covered Bridges

IN nearly 40 years of work on timber buildings and bridges, I have repeatedly encountered in historic covered bridges the mystery of apparent reserve capacity for live load. Routine analytical evaluations using current specifications indicate these bridges should have fallen down long ago, yet they continue to support vehicles with no apparent distress. Why?

While most of our remaining covered bridges were built after the basics of engineering analysis had been established in the middle of the 19th century, no standard design specifications were available to the builders of these structures. Extant bridges may have survived for a variety of reasons, but not because they were built in accordance with modern design practices.

Standardization of timber specifications did not commence in earnest until the late 1930s (with a subsequent big push during World War II), after almost all of the remaining covered bridges had been built. Builders could size bridges based on past experience or instincts without the need for numbers to document the dimensions needed. They had access to great-quality old-growth timber and were astute enough to place the best pieces where they would be exposed to the highest forces.

Heated exchanges occur in public meetings about repair or replacement of covered bridges. Bridge lovers want no noticeable modification of these wonderful old structures and challenge the engineer to find a way to avoid any proposed changes. The engineer, saddled with responsibility for public safety, has to be able to document such safety in keeping with the current standard of care. A lack of appreciation of the complexity of the structure and the nature of timber is a major part of the problem.

If we examine how timber design specifications were developed, we may be able to shed some light on why historic covered bridges seem to have more capacity than analytical evaluations indicate they should.

Wood vs. lumber vs. timber Wood is the material of the tree and is used here in reference to small pieces prepared for testing. In the US, lumber is sawn wood in specified dimensions, and much of the specification and historical development to be cited is technically aimed at lumber elements. Generally speaking, timber is large sawn or hewn elements. (Wooden-bridge engineers habitually identify themselves as "timber engineers," not "wood engineers," a nuance not intended to confuse.)

Although this discussion is limited to timber elements without regard to their connections, it must be recognized that the structural capacity of a historic timber structure is almost invariably controlled by the connections. The analytical review of connections, however, is a world unto itself whose inclusion would not illuminate our particular question.

Material background Like metal, timber reacts to loading in a generally predictable linear elastic manner up to a certain point, after which the relationship of stress to strain becomes nonlinear as the element is loaded to failure. Unlike metal, however, timber is anisotropic, with significantly different stress/strain properties depending on the direction of loading with respect to grain. Timber variation—most notably the extreme difference between cross-grain and parallel-to-grain cell structure—affects its strength, as do the interruptions of knots and the slope of grain to loaded surfaces. Variations in density and moisture are also significant factors.

Testing and material variability One of the first challenges when developing design standards is to confront the variability of the material. The chosen method of wood technologists was an extensive testing program of small clear specimens of the species in question, roughly 2 in. x 2 in. x 30 in. long with straight grain and no knots or other apparent imperfections, which served as the starting point for predicted strength. Each species for which a design parameter would be developed had to have a sufficient number of samples and tests to produce a statistically reliable value.

Inasmuch as this effort aimed to establish values for design of new structures, another question must then be confronted. In a large number of specimens, one might expect a plot of the results to follow a normal distribution curve (bell curve) of strength values. What value should then be selected from the normal distribution curve to serve as the basis of design? The mean? The 25th percentile? The 10th percentile?

The 5th percentile, commonly known as the 5 percent exclusion value, was adopted. That is, if 100 tests were performed on small clear specimens, the desired value would be found on the normal distribution curve where 95 percent of the test results were higher. It would be adopted as the basis for the strength of the group being tested. In other words, statistically one would expect that 95 out of 100 elements would have greater strength than the value used for sizing of elements. Does that not seem appropriately conservative?

I have delved into the history of this decision because it seems a part of this exercise that may justify a difference between design of new versus evaluation of existing elements. The 5 percent exclusion value is a refinement of earlier work by John Newlin, chief of the Timber Mechanics Division at the Department of Agriculture's Forest Products Laboratory (FPL) from 1910 to 1939. Newlin recognized the need to account for "within-species variability." To do so, he chose to multiply the mean of the tests by 75 percent.

This approach produces results quite close to the 5 percent exclusion value for most species of wood. I find this more visually clear—25 percent less than the mean value—and still seemingly conservative. History shows that the 5 percent exclusion value has worked quite well for nearly a century. But keep in mind that this procedure was the basis for design, not evaluation.

Next, a related question: How confident must we be that the normal distribution curve adequately represents the true strength of the group? 100 percent? 90 percent? 75 percent? A number of curves could be used to represent the test results. A "normal" distribution curve (a bell curve) was settled on by the industry, and then that required a decision as to how closely one demanded the curve to represent the tests. (This part of the work is aimed at how many tests are required for an average—2, 5, 35, 137?) The number of tested specimens for a given grouping was selected so that the normal distribution curve would represent a 75 percent confidence level in the results. A higher confidence level was considered unnecessary in combination with the 5 percent exclusion value, and there was a practical limit as to the number of tests that could be funded.

Hence design of new structures and the corresponding sizing of elements would be based on a 5 percent exclusion value with a 75 percent confidence level.

But notice that the stress/strain response of timber is substantially affected by the rate of loading in the test machine. It can "absorb" relatively large sudden loads without permanent deforma-

Phil Pierce

Hamden Bridge, Delaware County, New York, 128-ft.-span Long truss built over Delaware River in 1859 and propped midspan by pier in 1940s. During rehabilitation in 2000, pier was removed and bottom chords replaced by glulam because of theoretical weakness of chord splices, even if in good condition, in original design.

tion, but it will gradually creep under long-term load. Hence, the FPL decided early on that design values would be adjusted to what is termed *normal* loading—a period of load duration equivalent to 10 years. Adjustments for load duration of other than 10 years would be necessary in a subsequent phase of design.

These testing protocols and results are described and contained in American Society for Testing Materials (ASTM) specifications, notably D2555 *Standard Practice for Establishing Clear Wood Strength Values* and D245 *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.* More advanced work includes a commonly-cited extensive series of "in-grade tests": D2915 *Standard Practice for Sampling and Data-Analysis for Structural Wood and Wood-Based Products* and D1990 *Standard Practice for Establishing Allowable Properties for Visually Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens.*

The most widely adopted and cited tabulations of reference design values are provided in the *National Design Specification for Wood Construction* (*NDS*), promulgated and published by the American Wood Council, most recently in the 2012 edition. The 1991 edition of the *NDS* provides a meaty explanation of these matters, which I have simplified while retaining the important steps.

Design methodology Like metal and concrete specifications, early timber design specifications were based on an allowable-stress methodology derived from a stress-strain curve and statistical analysis, with appropriate reduction by a factor of safety to produce an acceptable or "allowable" stress for comparison to predicted (calculated) actual stresses.

These published allowable stresses, already reduced by the factor of safety, are commonly referred to as *reference design values.* As an example, the value for allowable tension can be thought of as equivalent to 55 percent of yield stress of a steel element—a value readily recognized by bridge engineers more familiar with steel than timber.

When working with steel, we commonly think of a single value as "the" factor of safety for a given stress—the 55 percent of yield in tension equates to a value of 1.82 as the factor of safety. But it is important to note that in timber the factor of safety as developed via statistical process is nothing near consistent.

In general, the average factor of safety in timber is on the order of 2.5. But because of the variability of wood, the factor may be larger or smaller for a given element. The procedures for establishing reference values per ASTM D245 and D2555, cited earlier, indicate that for 99 out of 100 pieces, the factor will be greater than 1.25, and for 1 out of 100, the factor will exceed 5. Such indeterminacy is very different from design in steel.

(The safety factor in wood is no simple matter. For a thorough analytic explanation of its development, see Lyman W. Wood, "Factor of Safety in Design of Timber Structures," *Transactions of the American Society of Civil Engineers,* Vol. 125, No. 1, pp. 1033–45. For a practical understanding of safety factors in wood, see fpl.fs.fed.us/documnts/fpltn/fpltn-222.pdf for the Forest Products Laboratory's *Technical Note 222.*)

What about "Load and Resistance Factor Design" methodology, now in vogue? Should we not be talking about its format conversion factors and resistance factors to enable sizing of elements? Perhaps so, but LRFD methodology still relies on strength values from small, clear specimen tests, with the 5 percent exclusion value and 75 percent confidence level built in. (And some timber engineers believe that there are kinks to be worked out in calibrating values between the two methodologies.)

Predicted stresses While focused on discussing development of allowable stresses, we became separated from the other part of the work, the prediction of actual stresses in service. Now we have to determine forces and corresponding stresses for the various types of loading that can be applied to the structure, and in which combinations, with their corresponding probability of occurrence.

The guidelines for which loads, and in which combinations they are to be applied to bridge structures in the United States, follow those published by the American Association of State Highway and Transportation Officials (AASHTO). In general, AASHTO has adopted the reference design stresses and adjustment factors of *NDS* with some minor tweaking.

Another diversion is necessary as part of predicting stresses. Recall that timber tends to accept short-term loads without damage but will creep with long-term loads. This phenomenon is accounted for in timber specifications via the load-duration factor, incorporated in stress evaluation according to the duration of the specific group of loads being considered. The factor for the individual group is associated with the shortest duration of load because if it were associated with the longest duration (dead load), the factor would always be the same. (Yes, this is weird and confusing—there is nothing really like it in steel or concrete analysis.) Load-duration groups could include, for example, the following:

Dead load only. AASHTO assumes dead load to be permanent throughout the life of the structure and assigns a load-duration factor, C_D, of 0.9 (this reduces the allowable to account for long-term creep).

Dead load + vehicular live load. AASHTO assigns a value equivalent to a total of 10 years of accumulated design loading over the life of the structure for a C_D of 1.00 (since the reference values are already given for an assumed load duration of 10 years).

Dead load + wind load. AASHTO assigns a value equivalent to a total of 10 minutes of full design wind force over the life of the structure with a corresponding C_D of 1.6—greater than one, recognizing the ability of wood to absorb relatively short bursts of loading.

Proceeding through the various combinations of loads with application of corresponding C_D, one arrives at the highest predicted stress to compare against the allowable selected from the tables with appropriate adjustments. (As an interesting aside, we might note that timber's ability to absorb large, quick loading eliminates the need for the "impact provision" multiplier of vehicular live loading required in steel or concrete bridge design.)

Member sizing Now that we have briefly reviewed the basis of wood design values as the statistically adjusted performance of small clear specimens of a given species, and explained some considerations in predicting stresses, we proceed to sizing of elements.

As we have seen, timber elements contain a variety of so-called defects—variations from clear straight grain—that reduce the capacity of the element from that implied by allowable stresses derived from small clear specimens. For example, a knot represents a major interruption to the flow of stress/strain along the path of an element. Reduction factors account for the effect of such deficiency. The slope of grain of an element is another key defect: if it's out of tolerance, it may warrant a reduction factor. Other defects include shakes and splits (forms of fiber separation), wane and other features of a natural material. More or relatively larger defects require greater reduction of allowable stress.

Reduction in capacity is made evident to designers by timber grading, with each grade—Select Structural, No. 1, No. 2 and the like—associated with a different set of allowable stresses. Identifying the grade via a "strength ratio" (the more the defects, the lower the value) is a multiplier of the results of small clear specimen tests.

So, we go to the *NDS* tables knowing the wood species we intend to use and select a structural grade that we intend to specify in our design. We then obtain the reference design values for bending, compression, shear, et al. Next, to size an element, we account for issues that can reduce the reference design value—e.g., moisture content or load-duration factor. We proceed to size the element accordingly so that the stresses are acceptable. But compared to what?

Restate the problem Comparisons of predicted stresses of extant wooden covered bridges against those of the *NDS*/AASHTO allowables routinely indicate overstress, i.e., lack of sufficient capacity of the bridge. In many cases, structure performance demonstrates more capacity than indicated by the standard allowables during evaluation. This leads to conflict over the need for element replacement or reinforcement.

What's wrong with our evaluation? Let's start with determination of predicted loads/stresses. It's important to recognize that dead load of covered bridges is much higher as a proportion to total load than is typical of modern steel or concrete bridges. The unit weight of timber elements varies per species, moisture content and preservative treatment. AASHTO specifies a density of 50 lbs. per cu. ft. (pcf) for design of new timber bridges, based on timber elements with high moisture content and creosote preservative. In-service unit weight of extant covered bridges is usually much less—often less than 30 pcf. If taken into account, this in-service weight would make a big difference to the calculation of reserve capacity for live loading. (Use of site-specific unit weights for extant covered bridges is accepted by AASHTO.)

Now suppose we consider capacity, the allowable stress side of the comparison. The determination of capacity of an extant wooden bridge begins with timber species. A wood scientist can readily confirm species based on small samples (the size of a pencil). Then, what's the grade of the timbers? This is not so easy a question because not all surfaces of all timbers can be seen, but with limitations understood it can be tackled by a certified lumber grader to identify size and distribution of knots, slope of grain, etc. When examining elements in an extant structure, the best that can be done is to identify the highest grade that can be assigned to the element, based on what is visible (the unseen material could be better or worse).

Another limitation is that each element has its own defects and therefore possibly its own grade, but it would be impractical to assign different grades to each element. Finally, a structure with relatively more hidden surface is more difficult—a Town lattice truss with its high proportion of mating surfaces, for example, would be more difficult to evaluate than any other truss configuration. (Removing the bridge siding for this exercise is probably not going to be performed for practical or economic reasons.) It is then up to the engineer to choose an appropriate grade—a daunting decision that depends on confidence and circumstance.

Given species and grade for the given element, we now go to the *NDS* tabulation and find the reference design values. We identify all appropriate adjustment factors via *NDS* with AASHTO overrides as appropriate and come up with the allowable stress to compare to the predicted actual stress for the various group-loading combinations. Does the extant bridge have the live-load capacity we were looking for or expecting? Probably not. What we have done so far is the easy part.

Now what? Are there other factors related to loads or stresses that can be tweaked for covered bridges with hope of gaining the capacity that seems hidden? Well, recall that AASHTO specifies use of a load-duration factor C_D = 1.0 based on an assumed 10-year total duration of design vehicular loading. That seems suspect. A total of 10 years of vehicle load on a single element? Recall that this spec is to represent the accumulated total time. The passage of a vehicle over the bridge probably takes seconds. How might that accumulate to 10 years (315,360,000 seconds)? And the value is to reflect the accumulation of passages of the "design" vehicle, the heaviest plausible, not the accumulation of all vehicle passages: passages of other weights do not count. Something is odd here.

For extant covered bridges, it would seem reasonable to calculate a revised value of this load amplification factor based on actual, estimated or hypothetical traffic information, or at least more rational values than the arbitrary value of 10 years used by AASHTO.

Forest Products Laboratory Research Paper RP-487, "Statistical Considerations in Duration of Load Research," uses a certain equation to develop duration-of-load factors for various types of loads —e.g., two months for snow load C_D = 1.15; one day for wind load C_D = 1.33. (See fpl.fs.fed.us/documnts/fplrp/fplrp487.pdf.)

The formula for the duration of load factor is

$$108.4 \div (60X)^{0.04635} + 18.3$$

where X is the total number of minutes for which the given load has been applied over the life of the structure, and the numerical values are based on best-fit from research.

There are 5,256,000 minutes in 10 years. For the 10-year C_D values in the tables, the formula then yields

$$108.4 \div (60 \times 5{,}256{,}000)^{0.04635} + 18.3 = 62.1$$

(Those who may use this equation to verify the C_D for loads and combinations cited earlier will find the formula for "permanent" loads—that is, dead load—does not lead to 0.9. Apparently the value of 0.9 was selected around the time of World War II as modern timber specifications were being developed.)

Suppose our extant bridge was built in 1880 and has been crossed by heavy loads equivalent to our intended design vehicle on average 10 times per day since it was first built, with an average duration to the loaded element of one second during the passage of the vehicle. That yields a total loading duration of

$$133 \times 365 \times 10 \text{ passages} \times 1 \text{ second} \div 60 = 8091 \text{ minutes} = 5.62 \text{ days}$$

Less than six *days*, not 10 years as AASHTO would suggest. Using the 8091 minutes instead of 10 years, the formula yields a value of

$$108.4 \div (60 \times 8091 \text{ minutes})^{0.04635} + 18.3 = 77.4$$

compared to the normal loading value of 62.1, indicating a load-duration factor of $77.4 \div 62.1 = 1.24$, or 24 percent less live load than when using AASHTO's generic value of 1.00. Maybe we are unwilling to go this far and we instead assume twice as many occurrences or 20 per day. That leads to a value of 75.5 for the equation—or a C_D value of 1.21—still 21 percent less than the generic value.

While this result may not represent a lot of savings, it's fair to explore the concept in a real-life evaluation. A younger covered bridge would probably have a larger C_D due to many fewer passages of vehicles, which indicates more capacity. This exploration assumes the same live-load force during each of the passages throughout the life of the bridge, whereas the 1800s did not necessarily have today's design loads. Certainly the weight of individual vehicles crossing the bridge varies with the vehicle.

This is a sticky issue. We are evaluating the effect of a specific weight of vehicle over a specific period of time and attempting to identify the total number of minutes of those passages. We could also be evaluating the results of a vehicle weighing less, but with more passages per day for comparison. There is no easy way to consolidate this topic into something truly black and white and widely accepted.

To complicate matters even more, since covered bridges can have snow on top of the roof while vehicles pass through the bridge, we have to consider a group-load combination of dead-plus-live-plus-snow at its own load-duration factor as well as its own probability-of-occurrence group-loading factor. Snow loads are not contained in AASHTO for modern design since we now use snow plows to push snow off uncovered bridges.

Are we lost yet? Hope not. Let's assume that our "refined" predicted actual stresses still don't properly cover real conditions.

How else to account for that extra strength? What about the extreme variability of that factor of safety noted above? Should we consider something else in our evaluation of a historic covered bridge? Is it the old-growth timber that we hear so much about, which must be stronger than modern timber? There is no doubt that old-growth timber was much more dense but, while density is an important aspect of the strength of timber, the reference design values provided in *NDS* have allowances for density built into specific grades based on empirical information. There are no readily available means of adjusting values of extant material to account for specific density.

Also, it's true that old-growth timber had many fewer knots (long story), but grading in the field is the limit of our means to evaluate a belief that old material is somehow stronger. We are trying to find extra strength that we can document in accordance with the standard of care of our times. We are not advocating that timber engineers go back to making our own specifications as did the 19th-century builders.

What about load testing? Strain gauges are commonly used to measure deflection or other movement of metal elements and sometimes concrete. Can we use strain gauges on timber? Hidden defects of larger bridge elements probably obviate strain measurements as indicative of actual stress. How do I know that I am measuring a legitimate "average" stress in an element, or even a realistic maximum stress? And what about the connections?

If we measure actual strains in an element and predict a capacity, can we say with any certainty that the joints have a similar or higher factor of safety? I think not. What we *can* do with strain measurements is to compare relative load sharing. For instance, the distribution of forces around a termination of a chord element of a Town lattice truss can be evaluated by strains with some degree of confidence.

What about deflection measurements? Flexural elements can be tested practically and with some confidence based on deflection, but not trusses, invariably the structural heart of a historic covered bridge. Deflections of timber trusses are extremely small, and the required accuracy of measurement makes reliance on the method suspect as well. For example, I have used various means to measure the deflection of a few historic bridges and found midspan deflections under a 15-ton vehicle load to be less than one-half inch. Such values are hard to replicate, and associating such small deflections with predictions of the capacity of the bridge is difficult.

What about our force analysis? I have not addressed the means and methods used to determine forces for this evaluation. Regardless of whether we use a simplified hand-analysis based on pinned-joint representation of truss behavior, or a computer program based on frame behavior (which should be very thoughtfully prepared), or some more advanced finite element representation, it's traditional that the analysis of the trusses be performed on a two-dimensional basis representing a single truss (without consideration of the deformation of the structure as a consequence of loading). Does truss analysis adequately account for the behavior of the structure as a whole?

Should we expand the analysis into a full three-dimensional representation of the structure, complete with floor system, bottom lateral bracing system (if one exists), overhead bracing system, maybe even the rafters, roofing and siding? But positing some sort of box to account for observed supplemental bracing and strengthening is not a reliable structural representation for support of vehicular live loads, because clearly these structures move and shift forces among the various available load paths, especially at joints, in ways we cannot model, probably not even fathom.

Is it possible that the timber deck may represent a potential benefit as additional "bottom chord" material? This theory obviously demands confidence in the deck acting compositely with the truss, in which case a physical attachment between truss and floor is required to account for horizontal shear load sharing. This could be more readily evident in a Town lattice truss with closely spaced floor beams than in a queenpost truss with widely spaced floor beams.

Last chance—there must be something! Let's look yet again at that 5 percent exclusion value. It was selected for the purpose of sizing structures or elements, and it has proven to yield structures that stand up to loads quite well. But is it too conservative for evaluation of extant covered bridges?

If an existing element was one of those with a low initial capacity because of some defect or poor overall quality, there is a good chance that it has already failed and been replaced with one of much higher capacity than indicated by our 5 percent exclusion value. On the other hand, if we believe that the element is one with higher capacity, then how do we justify raising the bar (numerically increasing the exclusion value)?

One can develop a tabulation to compare the increased basic reference stress from an increase in exclusion. If we consider just this effect, and limit ourselves to Coastal Douglas fir as a species, we find that for a 20 percent exclusion (or at 86 percent of the mean), we gain about 23 percent in strength. At a 30 percent exclusion (or 91 percent of the mean), we gain about 32 percent. Similar findings can be shown for any other species, based on their test data.

So what are appropriate reasons to modify the exclusion value to account for within-species variability?

Should the value be at all a function of age?

Should it be a function of element location within the bridge? An element with a lower design stress level may have been subjected to many fewer instances of high overstress, and hence may be worthy of less caution, perhaps allowing a higher exclusion value. If the element is one with a higher design stress, it probably has had many more instances of even higher overstress, thereby dipping into that reserve capacity more frequently (and thereby being more prone to failure sooner than later), so we should be more careful in that situation, and a numerically *lower* exclusion limit probably would be appropriate.

Should the value be a function of bridge location? A bridge that has survived on a more heavily traveled road might have more inherent capacity than one on a lightly traveled road, thereby potentially justifying a higher exclusion rate, or it may be on the verge of its capacity, while a bridge on a lightly traveled road may not have seen many heavy loads and could have ample reserve (or little reserve).

Is there finally a reason to consider modifying the exclusion value to account for what's not included in the myriad of other modification factors? I am not advocating for a specific value but for thoughtful consideration of this factor when faced with the implied need to replace elements of historic covered bridges. It may be that accepting a higher exclusion rule would support retention of elements that appear to be serving well, regardless of statistical implications of weakness.

Where does this leave us? Well, not with an answer, but with food for thought and perhaps a hunger to continue this exercise. I remain convinced that the 5 percent exclusion value in the setting of allowable stresses is the most suspect element in our evaluation of a historic covered bridge's capacity.

Anything else? It remains vital that we always strive for sensible weight limitations on extant historic covered bridges—lower than eight tons whenever possible. Three tons is a common limitation when alternate routes are readily available. Covered bridges were not built for large modern vehicles and should not be expected to support them. Allowing heavier vehicles to use these precious structures only hastens their demise. We should not be looking for hidden capacity to support unnecessarily heavy loads. —Phillip Pierce

Phillip Pierce, PE (phil@philsbridges.com), is Senior Principal Engineer at CHA Consulting, Inc., Albany, New York. He has worked and consulted on over 100 historic covered bridges and was primary author of the Federal Highway Administration's Covered Bridge Manual *(2005). He wrote about the Bartonsville, Vermont, covered bridge in TF 107. This article appears in abbreviated form in the online publication* Wood Focus *(London).*