Engineering the Speed River Bridge

Covered bridge engineers face the same basic questions that confront all frame designers. Can the proposed structure safely handle the expected loads? Will deflections be within acceptable limits? Can the members handle the resultant axial, shear and bending stresses? Are the connections adequate to deal with the forces and moments imposed? The goal of engineering and modelling is to create a sufficiently good approximation of the real thing to resolve these questions before the fact. There is a tension between the requirement to provide simple yes or no answers and the urge to recreate reality in the perfect model. Given world enough and time, we could perhaps build a computer model that would predict bridge behavior within extremely fine tolerances, but in the process we could also spend an entire bridge budget.

Bridge design loads include live loads—floor (pedestrian) load, snow load, wind load—plus dead load (Fig. 1). While the live loads are

Fig. 1. Design Load Summary

Live Loads	Unit Load psf	Area	Pounds
Floor	100[1]	1,364	136,400
Snow	32[2]	3,404	108,928
Wind			
Lower Lattice	9.3	1,848	17,186
Upper Lattice	6.2	1,932	11,978
Leeward Roof	-2.82 (suction)	2,138	-6,028
Windward Roof	3.95[3]	2,138	8,443
Dead Load			146,942

[1] Compare highway bridge design load of 55 psf.
[2] Snow load taken on horizontal projection.
[3] Roof wind load taken normal to roof surface.

significant, they are both variable and intermittent, and the likelihood of all three reaching their maximum at the same time is very small. Indeed, the Speed River Bridge will almost certainly never see a pedestrian load of 100 pounds per square foot. The real enemy of covered bridges is the steady, unvarying pull of their own weight.

Different loads are handled by different parts of the bridge. The lattice trusses are the primary mechanism for carrying vertical load (including a vertical wind load component). Horizontal load (wind) is dealt with by X-bracing at the level of the top chord, and by the floor system (intermediate lower chord, joists, double diagonal decking). Racking forces are resisted by bracing in the 15 major portals built into the length of the bridge on 12-ft. centers in midspan and 4-ft. centers over the piers. We analyzed these loads by designing a series of interrelated structural models, each of which addressed one or more aspects of the loading while at the same time providing sufficient information to forge the next link in the chain.

Snow, floor, and dead loads could be obtained through simple arithmetic, but the determination of wind load was not so straightforward. The walls of the bridge are partly open, allowing wind to pass through the upwind lattice and exert pressure on the downwind side. The question was how much wind would penetrate and how hard would it push on the far lattice. Sophisticated windload analysis or wind tunnel tests being out of the question, we made the worst-case assumption of undiminished wind force on the leeward wall. We did allow for the porosity of the lattice (two-thirds solid, one-third open), calling for 9.3 pounds per sq. ft. combined pressure and suction on the cedar-clad lower walls and 6.2 psf on the bare lattice above.

To determine the vertical windload component, we built a computer model of the bridge cross section and loaded it with wind (Fig. 2). The six vectors with "psf" labels are the wind loads applied to walls and roof (positive numbers signify net pressure, negative numbers net suction). The four "lbs" vectors at the base are the resultant reactions, that is, the forces necessary to counteract these loads in order to maintain equilibrium. Since reactions oppose loads, upward-pointing reaction vectors correspond to loads pushing downward, left-facing reaction arrows indicate loads to the right, and so on.

The leeward vertical reaction of 1,551 pounds represents the wind's down pressure on one bay (12 ft.) of bridge, translating to a unit load of around 130 lb/ft or 10.8 lb/in. The wind model also supplies a measure of side loading on the bridge (2x1,480 = 2,960 lbs/bay for a total of 35,500 lbs), as well as wind's overturning moment. Summing the individual bay windward vertical reactions of 1,403 gives a total overturning force of 16,800 lbs, hardly a match for the 147,000 pound bridge dead load.

Our principal engineering concern was with the strength and stiffness of the Town Lattice trusses that carry the bulk of the load. The preliminary truss design called for a 60° lattice, but because of the long span (120 ft. pier to pier) and heavy loading, the lattice angle was reduced to 45°. If you think of the truss as the structural analog of an I-beam in which the chords act as flanges and the lattice as web (Fig. 3), you can see that this beefs up the truss in two ways: it produces a denser lattice (more crossings) and also moves the intermediate chords away from the neutral axis and closer to the flanges (the extreme fiber) where they carry more load. (Yes, it also increases the dead load on the bridge.)

Our first candidate truss was made almost entirely of No. 1 Douglas fir 3x10, with a 45° lattice on 4-ft. centers, 132 ft.-long lower chords and 144-ft. upper chords. Town trusses are built up as a sandwich: first the outer chords (two laminae of 24-ft. 3x10) are laid down, then the first layer of lattice at 45°, another layer at 135°, and finally the laminated inner chords (3x10 plus 3x12), with 6x12 posts embedded on 12-ft. centers (Fig. 2). An effort must be made to stagger and space the chord joints so as to maintain three continuous laminae with maximum distance between breaks.

The multiplicity of parts and intricate geometry of large complex structures like covered bridges make them difficult or impossible to analyze by classical means, but the advent of the personal computer has brought with it powerful tools of structural analysis. Finite element analysis is a mathematical tool that harnesses the computer to perform the thousands of matrix algebra calculations necessary.

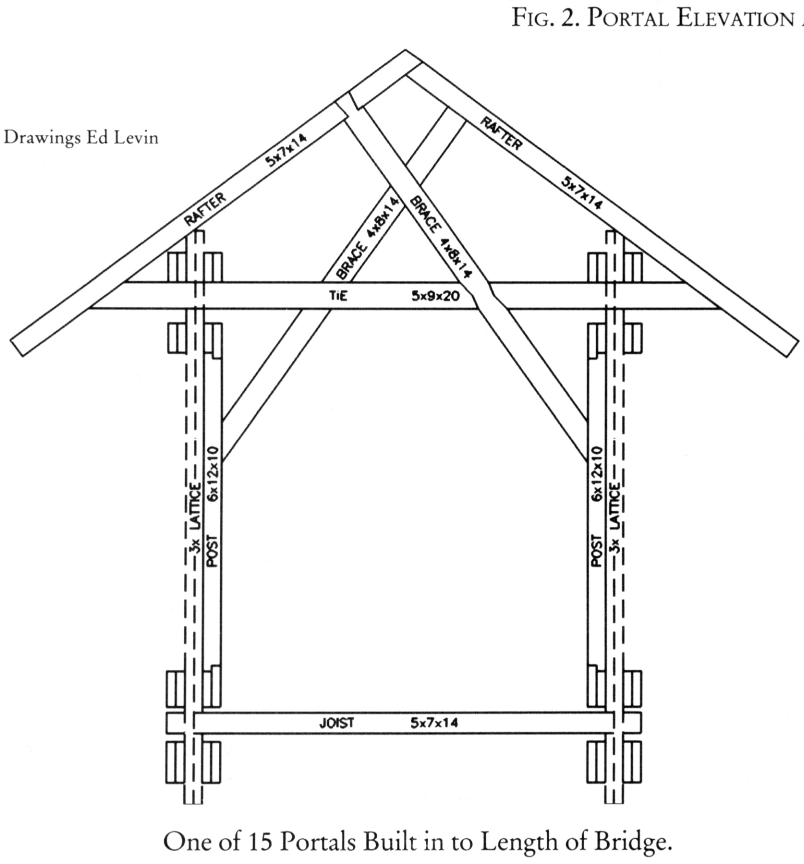
The first step in the process is *geometry definition*. *Nodes* or points are established in space and connected by *elements* (beams, plates, springs, solids) that are assigned cross-sectional and material *properties*, with the type and stiffness of joints governed by *end releases*. The model is located and stabilized by *restraints* in much the same way a foundation supports a building.

After the geometry is set, the stiffness matrix must be assembled. Getting this right on the first try is the equivalent of a hole-in-one. In a model of any size there are generally some unrestrained or unsupported points or elements that cause the matrix to blow up when some value is divided by zero. Complex models can take many minutes to run, and typically several attempts are required.

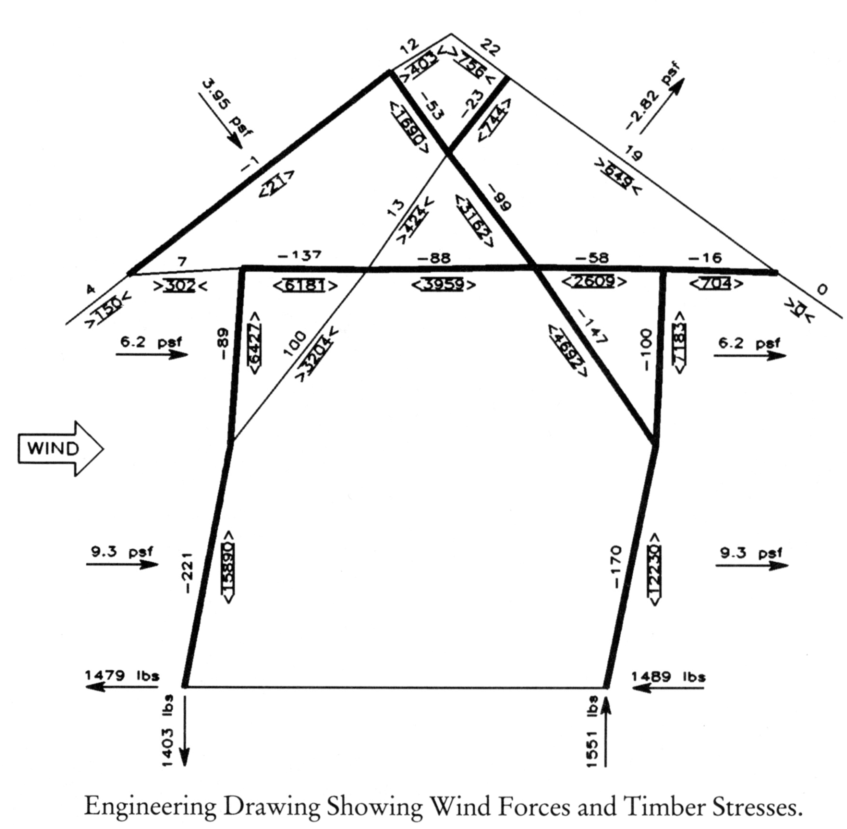
Once the matrix is built, you're over the hump. *Loads* are defined

Fig. 2. Portal Elevation and Typical Force-Stress Analysis.

Drawings Ed Levin



One of 15 Portals Built in to Length of Bridge.



Engineering Drawing Showing Wind Forces and Timber Stresses.

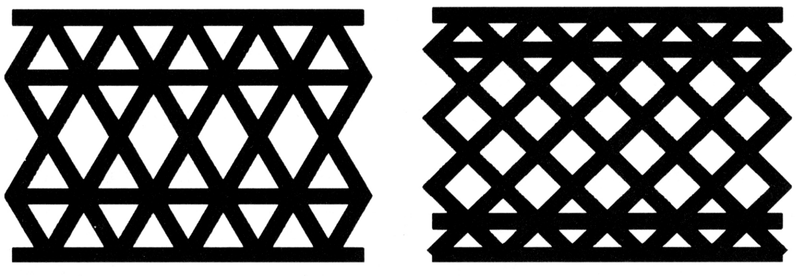


FIG. 3. LATTICE COMPARISON, 60-DEGREE VS. 45-DEGREE.

and applied to the model to determine *displacements*, and in the final step, the computer works backwards from effect to cause, using these calculated deflections to determine *forces*, *stresses* and *reactions*. As the name implies, the elements are finite—the model will not dispense information about just any part of the structure, but doles it out per-node, per-element.

Before pegging, a Town Lattice truss is nothing more than an artfully arranged lumber pile. Once pegged, there are no joints as timber framers recognize them, the only connections between members and layers being bridge pegs acting in shear. True, once the sides of the Speed River Bridge were raised, there was plenty of mortise and tenon work in the portals and roof trusses. But the span stands or falls on the strength of its 2,000 1¾-in. pegs.

In finite element analysis, the default condition—the assumed condition unless otherwise specified—for a simple joint between beams is a rigid, welded connection. Give it a "moment release" and you have a pinned connection, a fair approximation of the standard mortise-and-tenon joint, which doesn't develop much bending moment in its allowable deflection range. Give it an "axial release" and you have an unpinned connection. Modelling a Town Lattice presents a greater challenge to the engineer. Beams lap over one another with as many as four superimposed nodes at each crossing point, each one representing the interface between two layers of timbers: outer chord to outer lattice, outer lattice to inner lattice, inner lattice to inner chord, inner lattice and inner chord to post.

To demystify the action of the lattice, go out to your shop, lay two scraps of 2x4 over each other at right angles and drive a single 10d nail near one corner of the lap. Now you have an *axial* or *translation spring*: the sticks can still rotate on the nail, but they can't slide relative to one another. Add another nail to the lap in the opposite corner from the first and you've not only doubled the shear strength of the joint, you've also added a *torsion* or *rotation spring* to the system: now the joint can develop torque and resist rotation of the pieces relative to one another. Substitute 3x10 and 1¾-in. pegs for 2x4 and nails and multiply the setup several hundredfold and you've got yourself a Town Lattice truss.

The pegs themselves are modelled with *springs*. Every single shear connection is represented by two axial springs (X & Y) and a torsion (Z) spring. The axial or translation springs restrict horizontal (X) or vertical (Y) movement in the plane of the lattice, the torsion or rotation spring maintains the crossing angle between paired members in the truss. In modelling the bridge pegs, the trick lies in figuring out how much stiffness to give the springs. In a mortise-and-tenon joint, about which we have some information,* withdrawal of the tenon is resisted by pegs acting in double shear and also by friction in tight fitting joints, and rotation is restrained by these two components plus direct contact between the members at tenon shoulders and ends. In the lattice we have a quite different connection, where lattice crossings are pinned with two 1¾-in. pegs averaging 7 in. apart, and lattice-chord intersections receive three pegs on slightly closer centers (Fig. 4). We ran a series of test loads on the lattice truss model with various assumed peg stiffness values, then indulged in further informed hand-waving, picking spring stiffnesses that seemed the best match for the data and our expectations.

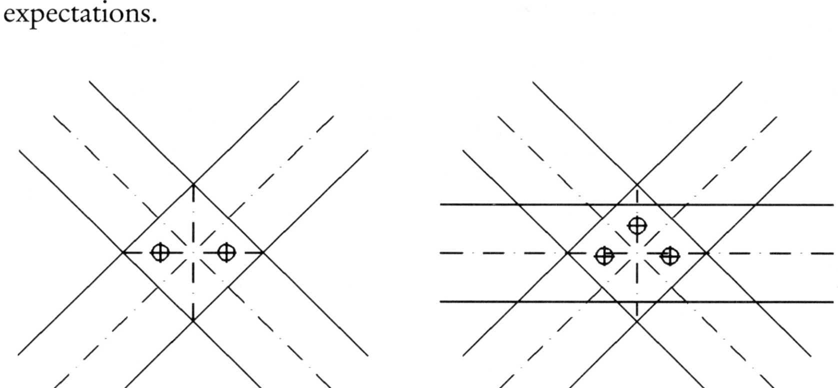


FIG. 4. TYPICAL PEG LAYOUTS, LATTICE ONLY AND LATTICE-CHORD.

We finally fixed the axial spring modulus at 500,000 pounds per in. per peg per single shear interface, such that lattice crossings were valued at 1 million lb/in. and chord joints at 1.5 million. We used a rotation spring constant of 3 million inch-pounds per radian (there are 2 Pi radians in one complete revolution) at all joints, after discovering that tailoring the value to each of the many combinations of peg numbers and intervals made no significant difference to our results.

The portal posts, locked into the lattice truss by long pegs at post-lattice crossings, lapped into the chords with a stub tenon (Fig. 2). Because of its short length (2 in.), this joint was not pegged and was modelled with a compression-only spring, with the result that axial forces on the ends of the posts show up as peg rather than beam output. Our model bridge was supported by model steel beams cantilevered out over the piers to pick up the adjacent nodes 4 ft. either side.

WHILE the bridge has a marked camber, the truss model is laid out on a straight line (Fig. 5). The purpose of bridge camber is aesthetic rather than structural, and as we saw in Guelph, it does create wonderful sightlines both outside and inside the bridge. Camber also acts as a kind of litmus paper for bridge health (by the time all the camber has flattened out, it's time for repairs), but it has no effect on structural performance unless the ends of the curve are restrained horizontally, and so is ignored in the model.

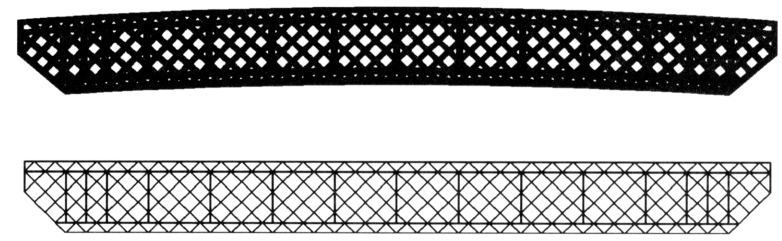


FIG. 5. CAMBERED (ACTUAL) AND STRAIGHT (MODEL) TRUSS RENDERING.

The lattice truss model (Fig. 6 overleaf) represents only half the length of the bridge. Since we were dealing with a symmetrical structure loaded symmetrically around its midpoint, we were able to model half the truss by cutting it at midspan and restraining horizontal translation and rotation at nodes in the plane of symmetry. This was vital since even the half-model strained the capacity of the software and computer (to say nothing of the patience of the operator).

The model built—chords, lattice and posts laid out, pegs installed, bed timber attached and supported, and stiffness matrix assembled—it was finally time to add some load. The computer can account directly for the dead load of the truss (lattice, chords and posts) if it knows the density of the material and size of the members, but additional dead load (floor joists, flooring, siding, roof trusses, roofing) and live load must be calculated and entered by the operator as unit loads. In addition to these distributed loads, there was to be a single concentrated load on the end of the truss. The covered bridge spanned the 120 ft. from pier to pier, and was to be approached on both ends by open ramps reaching 48 ft. from pier to abutment. Initial plans called for hanging these ramps from the upper truss chords, adding a 28,000-pound point load to the first run of the model.

At this point, it was once again time to play the worst-case assumption game. Ontario building code allows for two different load modification factors: a load combination factor to allow for reduced probability of simultaneous maximums of live loads in combination, plus a load duration factor to account for the ephemeral nature of live loads. While the two are applied in slightly different ways—the combination factor (CF) reduces load effects while the duration factor (DF) increases allowable stresses—they produce the same result. In Guelph, the two worst cases, one with wind, one without, were compared and the winner was the windless case. It is an indication of the twists and turns in our road that we went out of our way to figure the vertical windload on the truss only to have it drop out of the calculations at this next step in the process.

With the model built and loaded, it was finally ready to serve its intended purpose as an interactive design tool: a run of the model would indicate weak spots in the structure, changes would be entered and the model run again, and so on. But before this could happen, the first chore was to translate the output into a useful form. The need for graphic output was apparent, since each run of the truss model produced sufficient data to fill hundreds of pages. A quick exploration of the tabulated stresses revealed that the bridge channelled its load through true truss action—via axial loading—and the timbers exhibited minimal shear and bending stress. (Maximums were 36 psi shear, 571 psi bending in the bottom chord over the pier.) Thus only two types of drawings were required: beam output showing axial forces and stresses, and peg (spring) output showing shear forces.

The last run of the truss model is depicted in Figs. 6 and 7 (overleaf). The beam diagram shows the deflected shape of the half-truss under load with deflections magnified 50-fold. Compression members are represented by thick lines, tension members by thin ones. The numbers below the lines are axial *forces* in pounds, those above the lines give the corresponding stresses in psi. While a single beam drawing can tell the whole story, it takes four to render all the peg loads. (Outer chord to outer lattice, outer lattice to inner lattice, inner lattice to inner chord, inner lattice and inner chord to post.) In the sample shown (Fig. 7), the left-hand portion of the drawing shows output for each joint as separate X and Y forces and Z moments, reading top to bottom. In the right-hand portion the three components are resolved into a single vector.

To understand this last step, head back out to the shop and rescue that 2x4 lattice sample from the firewood pile. Stand the "X" erect on the bench so that the two nails make a horizontal line. Hold the right side steady and push down on the left. If the center of rotation is the

*Brungraber, Robert L., *Traditional Joinery: A Modern Analysis*, Unpublished PhD Thesis, 1985, pp. 49-64 & 105-116.

midpoint between the nails, then it is easy to see that the nail on the left is feeling a downward push while its mate is being forced upward. So if you apply a bending moment to a two-peg joint, the pegs feel opposing forces at right angles to the line between them.

Remembering that lattice pegs are aligned horizontally on 7-in. centers, you can translate a 2,100 in-lb rotational moment into two vertical forces of +300 and -300 pounds (300 lbs x 3.5 in. x 2 = 2,100 in-lb). While in practice these paired forces add to the vertical shear on one peg and reduce it on the other, in the peg output we made another conservative assumption, to match the sign of both forces to that of the joint's main Y-component and sum the three to obtain a net increase in Y-force. The total peg shear per joint was then calculated by taking a vector sum of the X and augmented Y components. Because of their primarily horizontal alignment and closer peg spacing, chord (3-peg) joints were treated the same way as lattice (2-peg) joints, the only difference being that the per-peg shear force is one-half of the total in the lattice and one-third in the chords.

The first run of the model keyed us in to the limiting factors that would drive the design. As mentioned earlier shear and bending stresses were not at issue. Neither, as it turned out was deflection. The maximum midspan sag we saw was under 2 in., or one part in 720, safely ignored. The principal areas of concern turned out to be peg shear forces in the lower chords above the bed timber and tension in the bottom chord at midspan.

The Canadian Standards Association manual, *Engineering Design in Wood*, lists the following design values for axial stress in Douglas fir (figures in psi):

	Tension	Compression
No. 1	652	1,261
Select Structural	1,000	1,406

Maximum compression and tension loads in the lattice are found at midspan in the top and bottom chords, respectively. Spreading the top chord load of 98,696 pounds over four 3x10s yields a compressive stress well within acceptable limits (98,696 lbs ÷ 120 in^2 = 822 psi). Because of the laminated chord structure, tension at butt joints in the bottom chord is only carried across three of the four laminae, raising tensile stress above allowable values (100,348 lbs ÷ 90 in^2 = 1,115 psi). The tensile force of 100,348 lbs is the resultant for select structural 3x12. The actual load from the initial run with No. 1 3x10 bottom chords was higher. Specifying select structural 3x12 for the bottom chord lowered this figure to 929 psi (100,348 ÷ 108), but there was still a problem: the chord cross-section acting in tension is further reduced by the pegs that fasten it. With two out of three pegs in each joint lined up with the grain (Fig. 4), net chord section in tension was reduced to 78.5 in^2 (108 in^2 - 31.5 in^2). In the end, we upgraded to 3x14 select structural, increasing the section and raising the load slightly, but bringing axial stress down very close to the acceptable range (126 in^2 - 31.5 in^2 = 94.5 in^2; 101,180 lbs ÷ 94.5 psi = 1,071 psi). Switching to 2-peg joints or reducing peg size in the bottom chord at midspan would have tweaked this figure the rest of the way into safe country, but ample precedent in the form of surviving Town Lattice bridges indicated that further precautions were unnecessary. The final chord configuration from top to bottom was 3x10 No. 1 fir upper chords, 3x12 select intermediate lower chord, 3x14 select bottom chord.

Unlike the forces in the chords, web axial forces are higher near the ends of the bridge and drop as you move to the center. This stress reduction was taken into account by graduating lattice size, starting with 3x12 over the piers and switching to 3x11 and finally 3x10 in the middle of the bridge (indicated by line types in Fig. 7). Since peg shear values follow the same pattern (decreasing towards midspan), we considered reducing peg sizes as well (down to 1½ and 1¼ in.), but as a practical matter this would have been a nightmare.

There is no official specification of design values for wooden pegs, so our work in this area involved a modest amount of hand waving. We did have one piece of evidence to go on: Milton Graton, of Ashland, N.H., the dean of modern covered bridge builders, had once tested some bridge pegs in a hydraulic press. His 2-in. white oak pegs failed in shear under a load of 24,000 pounds. Reducing this number proportionally for 1¾-in. pegs yielded 18,375 lbs, and applying a safety factor of 5 indicated a working peg shear of 3,675 lbs, a figure that concurred with tension test data gathered by Ben Brungraber. We were further reassured by Jan Lewandoski's report that his experience of dozens of Town Lattice bridges had shown peg failure to be almost nil.

In our attempt to reduce excessive peg loads, the hanging ramp was the first thing to go. The massive point load it imposed was channelled down the web, adding to already high shear forces over the bed timber. The ramp was extended to rest on the pier, but even without this load, a further reduction in peg shear was called for. The bulk of the peg load over the supports came from web compression members, with the vertical component predominating. This load was handled by cutting the lattice tails flush with the bottom chord so as to deliver their thrust directly to the steel bed timber. (To protect the lattice tails from condensate, the steel was covered with a ½-in. layer of structural plastic.) The residual horizontal thrust component could be handled by the pegs.

The Achilles heel of Town Lattice bridges is their lack of stiffness in cross-section. The structure of Ithiel Town's truss (patented 1820) presents scant opportunity to brace against racking, making the framework prone to side sway, and a glance through many old lattice bridges reveals them to be leaning to the side. But suppose you embedded posts at regular intervals in the lattice and sprang braces from them? Joel McCarty, our bridge designer, developed the idea: what about passing braces rising all the way to the roof and offering multiple crossing points at post, tie, brace and rafter? This strategy promised to provide the best defense we had seen against parallelogram tendencies, and the eventual result (Fig. 2) should insure the continued rectangularity of the Speed River Covered Bridge.

To date we had developed analysis and design strategies for the separate components, but still had no comprehensive picture of how the whole bridge would function. The time had come to put portals and lattice trusses together in context, to see how the system would stand up to the combined effects of dead and live load acting in three dimensions (Figs. 8 and 9).

It was not practical or economical to build a full bridge model showing every member and peg in the lattice and every joist, board and nail in the floor, so the lattice and floor were represented in the 3D model by *plate* elements, which were assigned densities to account for dead load and equivalent stiffnesses to simulate their performance under load. In the case of the lattice, having already modelled the truss, we knew its maximum combined load deflection (1½ in.), and could simply adjust wall plate stiffness in the 3D model to duplicate this deflection.

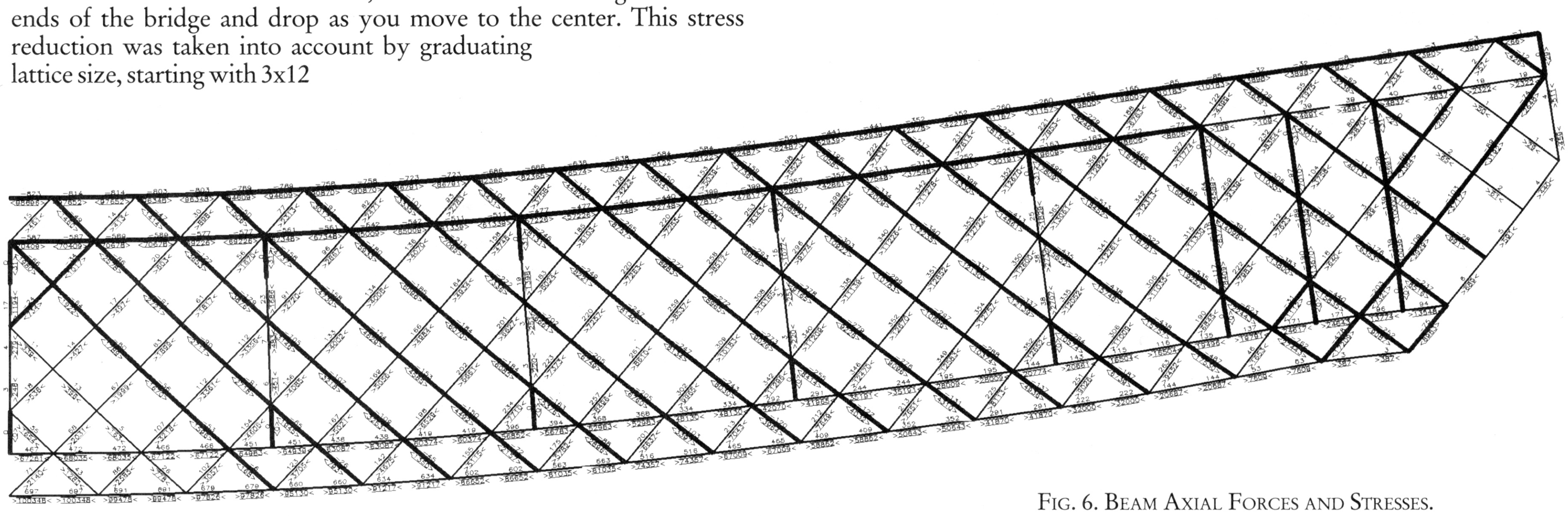
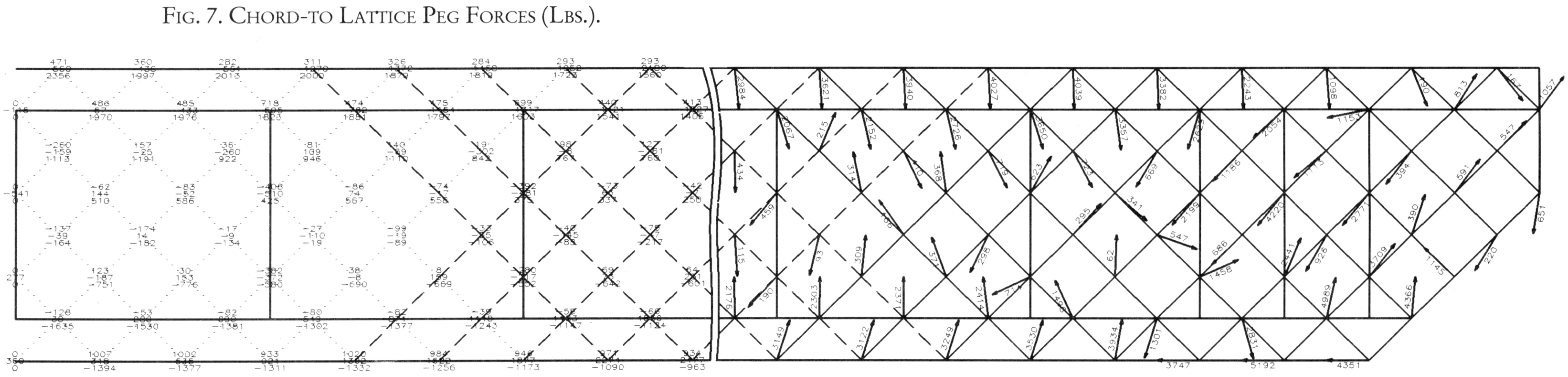


FIG. 6. BEAM AXIAL FORCES AND STRESSES.

FIG. 7. CHORD-TO LATTICE PEG FORCES (LBS.).



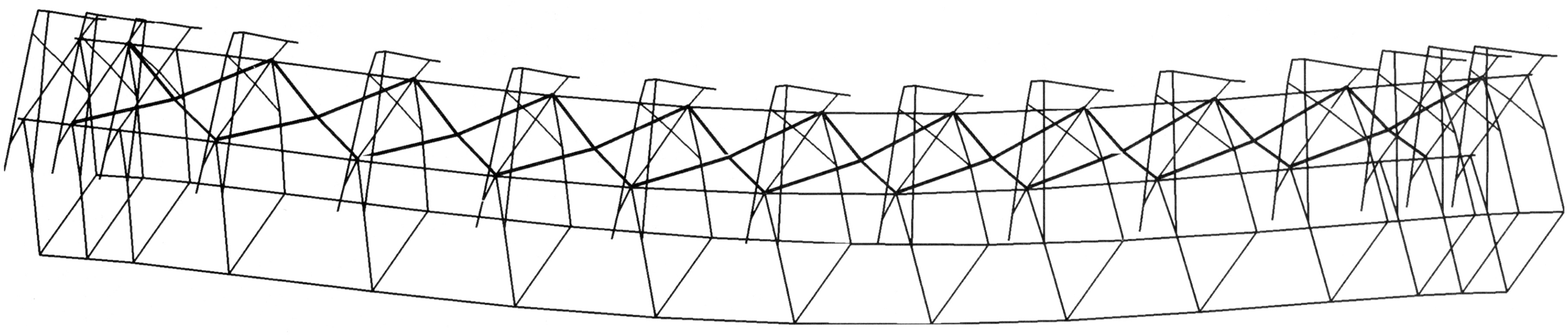


Fig. 8. Deflected Shape, Aerial Perspective View.

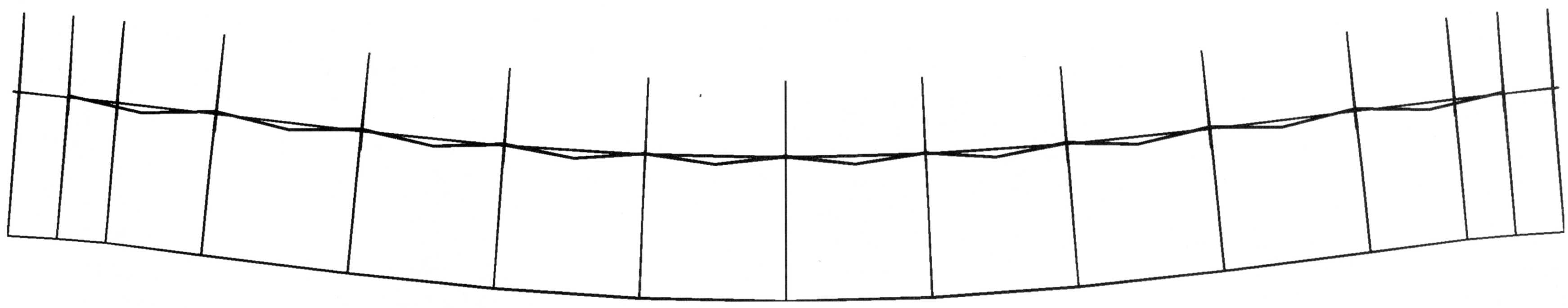
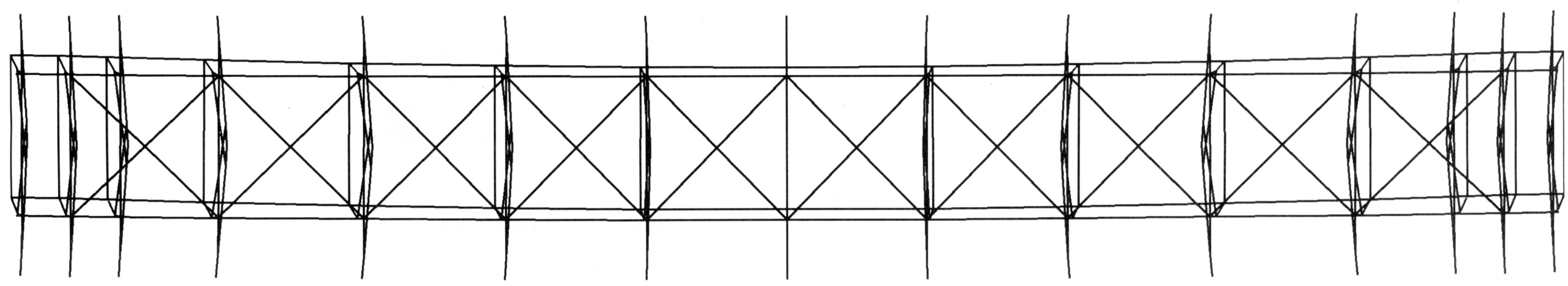


Fig. 9. Deflected Shapes, Elevation (Above) and Plan (Below).
X-Braces in Ceiling Plane Apparently Yield to Compression by Bending.



It was a bit more problematic to analyze the diaphragm action of the diagonal decking chosen to stiffen the floor of the bridge. The layout of this system—double-diagonal rough 1-in. cedar flooring connected to joists and truss chords by a heavy nailing schedule—makes it a distant cousin of the lattice truss. A variety of educated guesses were proposed for determining diaphragm deflection before we finally adapted a method from the Council of Forest Industries of British Columbia *Plywood Construction Manual*. This formula contained three terms easily understood in terms of the I-beam analogy: one for chord strain, one for web shear (combined stiffness of floor joists and flooring), plus a factor for nail deformation under load. We papered several walls with the calculations before predicting that the floor of the bridge was one third as stiff as the lattice trusses.

The 16 different strands of top and bottom chords were modelled as four single units using the moment of inertia taken around their common centroid, and all remaining bridge timbers (posts, braces, ties, X-braces and rafters) were shown as beams with pinned joints.

Principal unanswered questions behind the 3D model were the determination of the bridge's racking resistance and horizontal deflection under wind load. How would our combinations of braces and diaphragms behave, how would they share the load?

In plan, lower chords were in tension, upper chords in compression as expected. In cross-section, the combined load put different parts of the braces into different states. Above the collar crossing, the braces went into compression, while below the collar they were stretched on the upwind side, compressed on the downwind. The two exceptions to this rule—upwind upper braces over the piers were in tension, lower braces in the three portals at midspan were all in compression—are easily explained by racking.

In still air, both parts of the braces feel a push as they naturally channel roof load down to the posts. Indeed this condition prevails at midspan even in the presence of a wind load. The tendency to rack is greatest at the ends of the bridge where the bottom chord is restrained laterally at the piers while the top is free to move. Here the wind distorts the end portals enough to put the upwind upper braces into tension. At midspan both upper and lower chords are free of restraint, so while overall deflection is greater, racking is much reduced and gravity rules. There is still some portal distortion since windloads are greater on the upper chords while the bridge is stiffest in the plane of the floor system.

Maximum horizontal deflection of the model was around ½ in. at midspan as compared to the vertical displacement of 1½ in. It's a testimony to the stiffness of the lattice trusses that while the combined gravity loads were 10 times the horizontal wind force, the vertical deflection was only three times the horizontal. As with the lattice truss analysis, shear and bending stresses in the portals were minimal. Axial forces worthy of note included a 3,300 pound tension load on upwind lower braces over the piers and tension in the ties and posts towards midspan. (It's a bit troubling to find posts in tension, until you look at the role they play the lattice truss.)

So far the exceptions only proved the rule, but there was still a surprise in store. Isolate the plane of the top chords, ties and X-braces and you're looking at a 144-ft.-long horizontal truss. Load it with wind and you expect to see a stress pattern similar to the Town truss, with alternately tensed and compressed diagonals. Why then were almost all the X-braces in our 3D model in compression? Take the gravity load off the model and the X-braces show a pleasing alternation, but when you run it with gravity and no wind, the unexpected pattern returns. In the combined loading, meanwhile, the massive gravity load simply overcomes the puny windload.

To solve the mystery, you need to step back and look at the model through a wide-angle lens. Our close focus on the portals and lattice trusses had masked the synergistic action of the bridge. What we were seeing was the entire structure reacting in bending like a giant square tube or beam. Looking at our old friend, the I-beam analogy, you can see that if the lattice trusses serve as the web, then the X-braces are the top or compression flange and the floor system the bottom (tension) flange. And sure enough we found tension dominating in the floor elements.

In the end the question remains: why *do* all this? Given the extensive empirical evidence of surviving covered bridges, is it really necessary to go to all this trouble? Do we really believe that finite element analysis models are true and accurate measures of bridge behavior? Yes and no. None of the designers of the Speed River Covered Bridge has any doubts that we could have built a fine bridge without benefit of computers, but show me a timber framer who isn't curious about how structures work and I'll declare him brain-dead. And construction in the modern world—especially in the public sector—is a game played by stringent rules requiring a degree of documentation most easily obtained through finite element analysis.

But the best reason for analysis isn't this bridge, it's the next one. It's a lot easier on the nerves to watch a model fail than a bridge. Run by run, model by model, bridge by bridge, we bring the simulated world closer and closer to the real one. When the commission comes in for that 200-ft. single-span highway bridge, we'll feel better knowing that the relationship between the two is a happy one. —Ed Levin

The author wishes to thank professional engineers Ben Brungraber (Keene, N.H.) and Dave Trudell (Markdale, Ontario) for their encouragement and assistance.