CONTOOCOOK RAILROAD BRIDGE
(Hopkinton Railroad Bridge)
Spanning Contoocook River at former Boston & Maine Railroad
(originally Concord & Claremont Railroad)
Hopkinton
Merrimack County
New Hampshire

HAER NH-38
NH-38

PHOTOGRAPHS

WRITTEN HISTORICAL AND DESCRIPTIVE DATA

REDUCED COPIES OF MEASURED DRAWINGS

FIELD RECORDS

HISTORIC AMERICAN ENGINEERING RECORD
National Park Service
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1849 C Street NW
Washington, DC 20240-0001

HISTORIC AMERICAN ENGINEERING RECORD

CONTOOCOOK RAILROAD BRIDGE
(Hopkinton Railroad Bridge)

HAER No. NH-38

LOCATION:	Spanning Contoocook River at former Boston & Maine Railroad (originally Concord & Claremont Railroad), Hopkinton, Merrimack County, New Hampshire UTM: 19.279593.4789158N, Hopkinton, New Hampshire Quad.
STRUCTURAL TYPE:	Town lattice through truss covered bridge
DATE OF CONSTRUCTION:	1889
DESIGNER/ BUILDER:	Boston & Maine Railroad
PRESENT OWNER:	New Hampshire Division of Historical Resources
PREVIOUS USE:	Railroad Bridge, 1889-1962; Warehouse, 1962-1989
PRESENT USE:	Historic landmark and tourist attraction
SIGNIFICANCE:	The Contoocook Railroad Bridge is one of eight surviving covered bridge in the United States. It is an excellent example of the double-web Town lattice truss used by the Boston & Maine Railroad in the late nineteenth century.
RESEARCHERS:	Lola Bennett researched and wrote the historical report, August 2003. Dorottya Makay wrote the engineering report under the supervision of Justin M. Spivey, August 2003.
PROJECT INFORMATION:	The National Covered Bridges Recording Project is part of the Historic American Engineering Record (HAER), a long-range program to document historically significant engineering and industrial works in the United States. HAER is administered by the Historic American Buildings Survey/Historic American Engineering Record, a division of the National Park Service, U.S. Department of the Interior. The Federal Highway Administration funded the project.

RELATED DOCUMENTATION: See Contoocook Covered Bridge (HABS No. NH-21). HABS measured the bridge, which was a Long truss that carried the main road to Hopkinton, in 1935 prior to its demolition. The photograph shows the Contoocook Railroad Bridge in the background.

Chronology

1784 Ithiel Town born.

1788 New Hampshire becomes ninth state to enter the Union.

1791 Eliphalet Poor establishes mill near this site.

1820 Ithiel Town patents Town lattice truss.

1844 Ithiel Town dies.

1848 Concord & Claremont Railroad chartered.

1849 First railroad bridge built at this site.

1853 Contoocook Village Bridge erected.

1870 Railroad leased to Boston & Maine Railroad.

1872 Concord & Claremont Railroad completed.

1888 J.P. Snow becomes engineer for Boston & Maine Railroad.

1889 Contoocook Railroad Bridge built.

1911 J.P. Snow retires.

1936 Contoocook Railroad Bridge survives flood.

1938 Contoocook Railroad Bridge survives hurricane.

1955 Passenger service ends on the line.

1962 Freight service ends on the line.

1980 Contoocook Bridge listed on the National Register of Historic Places.

1989 State of New Hampshire takes ownership of Contoocook Railroad Bridge.

1994 Arnold Graton repairs floor and roof.

2000 Contoocook Riverway Association formed.

2003 Contoocook Railroad Bridge recorded by Historic American Engineering Record.

Introduction

Some of the earliest railroad bridges were timber structures because wood was abundant, cheap, and easy to work with. In 1830, Lewis Wernwag built the first wood railroad bridge in the United States for the Baltimore & Ohio Railroad over the Monocacy River in Maryland. Within a short time, wood bridges were commonplace on America's growing network of railroads.

Presumably hundreds of covered railroad bridges were built in the nineteenth century. In 1841, one English traveler noted, "The timber bridges of America are justly celebrated for their magnitude and strength. By their means the railways of America have spread widely and extended rapidly."[1] By the late nineteenth century, most railroad bridges were being built of iron or steel.[2] The Boston & Maine Railroad (B&M) was an exception – they continued to build timber bridges into the early twentieth century. The Contoocook Railroad Bridge is one of eight surviving covered wood railroad bridges in the country and an excellent example of the double-web Town lattice truss design used by the Boston & Maine Railroad in the late nineteenth and early twentieth centuries.[3]

Description

The Contoocook Railroad Bridge is a continuous two-span, double-web Town lattice truss covered wood bridge on a mortared stone pier and abutments. The total length of the bridge is 157'-3", with spans of 63'-4½" and 68'-9¾".[4] The truss is 20'-0" high from the top of the upper chord to the bottom of the lower chord and 21'-0" wide overall, with a width of 15'-8" between the trusses. Vertical clearance is 19'-4".

The spruce trusses are framed in the manner patented by Ithiel Town in 1820 and modified in 1835. The upper and lower chords are triple lines of 3x10" planks. There are secondary chords (three pairs of 3x10" planks) above the lower chord and below the upper chord, as patented in 1835. There is a third chord (three pairs of 3x10" planks) above the lower secondary chord. The chords sandwich offset double lattice webs, composed of 3x12" planks, three diamonds high. The webs are fastened at each lattice intersection with two 2" diameter treenails, at the upper chords with three 2" diameter treenails, and at the lower chords with four 2" diameter treenails. There are paired 6x12" timber posts at the ends of the trusses and two pairs of timber posts (four per truss) flanking the trusses at mid-span to carry the vertical forces to the pier and abutments.

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[1] Richard Sanders Allen, *Covered Bridges of the Northeast* (Brattleboro: Stephen Greene Press, 1957), p.94.
[2] Nevertheless, the process of replacing bridges was a gradual one. In 1889, railroad engineer Theodore Cooper stated that among the 1,600 miles of railroad bridges in existence, only 380 miles were iron. See Theodore Cooper, "American Railroad Bridges," *Transactions of the American Society of Civil Engineers* 21 (July 1889).
[3] Contoocook is the oldest of four surviving Town lattice covered railroad bridges built by the Boston & Maine Railroad (B&M). The others are: Wright's Bridge, Newport, NH (1905-6), Pier Bridge, Newport, NH (1906), and Fisher Bridge, Wolcott, VT (1908). According to covered bridge historian Joseph Conwill, the Fisher Bridge was on the Saint Johnsbury & Lake Champlain Railroad (later, Saint Johnsbury & Lamoille County), although the B&M may have been operating the line when the bridge was built.
[4] Span lengths taken from drawings, sheet 3.

The upper lateral system consists of 8x8" transverse tie beams seated on the upper chord at the lattice intersections, spaced approximately 5' apart. There are 6x6" lateral cross braces between every three struts and sway braces between the tie beams and upper secondary chords.

The floor system is composed of 10x15" floor beams suspended from the lower chord with 1½" diameter bolts which pass through a plate under each beam and up through a wooden block on top of the chord and are fastened with a nut and washer assembly.[5] The floor beams support two lines of stringers, 6x12" timbers laid flat. Originally, the railroad ties and tracks would have been fastened on top of the stringers, but these have been removed. In their place is a timber walkway composed of 5x7" timbers placed transversely every 12" along the middle of deck.

Variable width vertical board siding covers the exterior of the bridge to 2' below the upper chord. The sheathing is fastened to nailers on the outer faces of the lattice webs. Rafters, measuring approximately 2x6", frame between a 5x5" timber on the outer edges of the tie beams and a 4x6" ridge pole resting on blocks on top of the tie beams. The rafters support longitudinal sheathing, to which is fastened a very low-pitched metal roof. The roof has overhanging eaves and exposed rafter tails. The portals are straight with hipped openings, pilaster moldings and shelter panels. The pediments have an outward curve at the eaves where the rafters would otherwise be exposed.

History

In the late 1700s, Eliphalet Poor established a mill near this site.[6] By the early nineteenth century, Contoocook was a small industrial hamlet with numerous saw, grist and silk mills. There was a bridge at this location prior to 1790, when the town voted to repair it.[7] That bridge, known as Poor's Bridge, was rebuilt in 1794, and presumably numerous other times prior to 1853, when a covered bridge was built here to carry vehicular traffic.[8] The present stone-faced concrete arch highway bridge replaced that bridge, which stood adjacent to the Contoocook Railroad Bridge for nearly half a century, in 1936.

In 1848 the Concord & Claremont Railroad was chartered to build a 60-mile line through the Contoocook River Valley. The company erected covered bridges at several major river crossings, including Contoocook Village.

Most modern sources report 1849 as the date of construction for the present bridge; however, according to a report of the Concord & Claremont Railroad published in 1884, the bridge at Contoocook was a covered bridge "of the Child's plan" which "has been damaged some by ice and water, and sags in the center. It is very old, and should be carefully attended to and rebuilt

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[5] The hanging floor system is unusual among Town lattice trusses.
[6] *Today and Yesterday: An Illustrated Historical Account of the Town of Hopkinton, New Hampshire 1765-1965* (Hopkinton, New Hampshire, 1965), p.16.
[7] C.C. Lord, *Life and Times in Hopkinton, New Hampshire* (Concord: Republican Press Association, 1890), p.93.
[8] See Historic American Buildings Survey, Contoocook Covered Bridge, HABS No. NH-21.

soon as practical."[9] While no documentation has been found specifically mentioning the construction of the present bridge, in 1888 the Boston & Maine Railroad took over the Concord & Claremont Railroad and immediately set about upgrading various parts of the line. In 1889, the railroad built the present double-web Town lattice truss covered bridge. Although Boston & Maine Annual Reports 1891-1893 do not specifically mention this bridge, the 1914 I.C.C. Valuation Survey lists 1889 as the date of its construction.[10]

Historic photographs of the bridge in the collection of the New Hampshire Antiquarian Society in Hopkinton clearly show two very different covered bridges here at different points in time. The first has a steeply pitched roofline, tight eaves and arched portals; the second is the present bridge with its low-pitched roof and squared portal.[11] The present bridge very closely matches other timber truss covered bridges built by the Boston & Maine Railroad during the period from 1888 to 1911 when Jonathan Parker Snow was bridge engineer. Despite the lack of written documentation, there is no question that the present bridge dates to this time period.

The Contoocook Railroad Bridge was knocked off its footings twice, once during the flood of 1936 and once during the hurricane of 1938. Both times, the bridge survived intact and was hauled back up onto the abutments.

In 1989, the Town of Hopkinton transferred ownership of the covered bridge to the state for maintenance and preservation. Although the state does not have funds available for this purpose, they are able to accept donations from the National Society for the Preservation of Covered Bridges and oversee volunteer preservation efforts. In 2000, the Contoocook Riverway Association was formed to raise money for village improvements, including restoration of the depot and Contoocook Railroad Bridge.[12]

Design

Ithiel Town was born in Thompson, Connecticut in 1784 and died in New Haven in 1844. As a young man he learned carpentry and later studied architecture at Asher Benjamin's school in Boston. For most of his life, he practiced architecture, primarily as a partner in the New York City firm of Town & Davis. Town designed a number of noteworthy buildings, including Christ Church in Hartford (1825), the New York City Custom House (1837), the Yale College Library (1842), and the Virginia State Capitol at Richmond (1842). Although he is primarily recognized as an architect, Town also made a significant contribution to the field of engineering when, in 1820, he was granted a patent for a truss bridge. As he explained in his 1821 pamphlet, "A Description of Ithiel Town's Improvement in the Construction of Wood and Iron Bridges," this

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[9] Horace, Enoch and Warren Childs of Henniker, New Hampshire, were related to Col. Stephen H. Long of the neighboring town of West Hopkinton, who patented the Long truss in 1830. Beginning in the 1830s, the Childs brothers acted as agents for Long and built many Long truss covered bridges in the area. In 1846, Horace patented the Childs truss, and subsequently formed his own company and built railroad bridges throughout New England.
[10] Wayne Perry, "Northern Railroad Bridges," *Covered Bridge Topics* (Spring 1984): 6-9.
[11] See *Today and Yesterday*, p.xx.
[12] Contoocook Riverway Association, "Preserving A Hopkinton Tradition," publicity flyer, 2000.

new method of bridge construction was designed to be "the most simple, permanent, and economical, both in erecting and repairing."[13]

Town's design consisted of two layers of overlapping planks running perpendicular to each other, with each layer arranged at an angle to the chords, forming a lattice fastened together with wooden pins or treenails at each intersection. The most significant feature of this design was that it could be quickly erected and utilized sawn planks instead of heavy hewn timbers. In 1821, Town published a description of this design in *The American Journal of Science & Arts*.

> In this he specifically proposed spans of 120 to 160 ft. made of planks 10 to 11 in. wide by 3 to 3½ in. thick set at 45° forming relatively open lattices (only double or triple intersection), with a span:depth ratio of 10:1. The lattices were to have three or four 1½ in. diameter treenails through each intersection. To stabilise [sic] the lattices Town proposed single stringers on either side at top and bottom for spans up to 130 ft, but for longer spans he proposed adding others.[14]

The lattice design actually functioned as a series of overlapping triangles so that the load in any one triangle affected distribution of stress in all other triangles. Because the webs were fastened at every intersection, no triangle could function independently, and, as bridge historian Richard Sanders Allen points out, "Therein lay the great strength of the Town truss. It was a real invention, not resembling any design advanced for wooden spans in the thousands of years before its time that bridges had been built."[15] Because it did not rely on European precedents, the Town lattice is considered "the first truly American design" for a bridge truss.[16]

Town took out a second patent in 1835, adding a second lattice web. While single lattice trusses could safely carry railroad loadings to 80', they tended to warp under heavy loads; by doubling the lattice web, the truss became more rigid and resistant to warping. The secondary chords were used in both railroad and highway bridges to help stiffen the truss.[17]

Builder

On most railroads, metal trusses gradually superseded timber trusses in the late nineteenth century. The Boston & Maine Railroad, however, continued to maintain and build timber bridges into the early twentieth century. This was largely due to the efforts of Jonathan Parker

[13] Ithiel Town, "A Description of Ithiel Town's Improvement in the Construction of Wood and Iron Bridges: Intended as a General System of Bridge-Building" (New Haven: S. Converse, 1821), p.4.
[14] J.G. James, "The Evolution of Wooden Bridge Trusses to 1850," *Institute of Wood Science Journal* 9 (June and December 1982).
[15] Richard Sanders Allen, *Covered Bridges of the Northeast* (Brattleboro, Vermont: Stephen Greene Press, 1957), p.15.
[16] Raymond E. Wilson, "Twenty Different Ways to Build a Covered Bridge," *Technology Review*, Massachusetts Institute of Technology (May 1971).
[17] Although numerous examples of the secondary chords exist among the extant population of Town lattice trusses, the four surviving Boston & Maine covered railroad bridges are the only extant examples of the double-web Town lattice type.

Snow (1848-1933), an advocate of timber bridges, who served as an engineer for the Boston & Maine from 1888 to 1911.[18]

Early in his railroad work, Snow became convinced that wooden truss bridges should be maintained in service as long as possible, instead of being replaced with iron trusses. In 1895, nearly 70 percent of the bridges on the Boston & Maine Railroad were wood. It was accepted that they might have a shorter service life, but they could be easily reinforced if necessary, and they gave ample evidence of distress long before failure. An iron single-track bridge of 120' span cost about $5,300, but a spruce lattice truss was only about $3,500. Snow advocated the double-web Town lattice truss, pointing out that while single lattice trusses tended to warp under heavy loads, the double lattice stayed rigid because it functioned like a box girder.[19] In 1900, there were an estimated one hundred Town lattice truss covered bridges on Boston & Maine lines.[20] Although no longer in service, four of these covered bridges survive: Contoocook (1889), Wright's (1906), Pier (1907) and Fisher (1908).[21]

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[18] See Wright's Bridge, HAER No. NH-35.
[19] Jonathan Parker Snow, "Wooden Bridge Construction on the Boston and Maine Railroad," *Journal of the Association of Engineering Societies* 15 (July 1895): 31-43.
[20] W. Edward White, *Covered Bridges of New Hampshire* (Littleton, New Hampshire: Courier Printing Co., 1942), p. 53.
[21] See Wright's Bridge, HAER No. NH-35.

APPENDIX A: Surviving Covered Railroad Bridges in the United States

29-07-07	Contoocook Bridge	Merrimack County, NH	1889	157' Town lattice truss	B&M RR
29-07-09	Sulphite Bridge (see HAER No. NH-36)	Merrimack County, NH	1896	180' Pratt deck truss	B&M RR
45-01-05	Shoreham Bridge (see HAER No. VT-32)	Addison County, VT	1897	109' Howe truss	Rutland RR
29-05-14	Clark's Bridge (see HAER No. NH-39)	Grafton County, NH	1904	116' Howe truss	M&WR RR
29-10-03	Wright's Bridge (see HAER No. NH-35)	Sullivan County, NH	1906	124' Town lattice truss	B&M RR
29-10-04	Pier Bridge	Sullivan County, NH	1907	217' Town lattice truss	B&M RR
45-08-16	Fisher Bridge	Lamoille County, VT	1908	98' Town lattice truss	SJ&LC RR
47-38-01	Harpole Bridge (see HAER No. WA-133)	Whitman County, WA	1922	163' boxed Howe truss	Great Northern RR

APPENDIX B: Engineering Report

OBJECTIVES OF THE STUDY

This engineering report examines the Contoocook Railroad Bridge, built in 1889 and the oldest of the four surviving double-web Town lattice bridges. Double-web Town lattice trusses represent the most developed version of these structures. With two lattice webs on each side, they were strong and stiff enough to carry railway live loads up to as much as Cooper's E50 loading specification. Development of the lattice truss, first patented by Ithiel Town in 1820, was initially more empirical than engineered. They became well-proportioned structures by the last half of the nineteenth century. Town lattice trusses, even double-web ones, were characterized by simple construction technology, relatively low material cost, and ease of erection using common wood construction techniques.

The principal objective of this study was to quantify the static behavior of double-web Town lattice trusses under both dead and live load conditions, and to investigate certain three-dimensional behaviors, including rotational and linear deformational stiffness of the various treenail joints, all to provide guidance for sustainable use, maintenance, and rehabilitation of this, and similar, structures.

To quantify the static behavior of the Contoocook Railroad Bridge, the following tasks were undertaken:

- identification of the timber species by the USDA's Forest Products Laboratory;
- identification of the geometry and load conditions—based on HAER team research;
- identification of the treenailed joints' rotational and linear deformational stiffness – based on HAER team research and local testing;[22]
- calculation of joint stiffness through modeling and the use of existing former laboratory test results carried out by others;[23]
- two- and three-dimensional finite element analyses of the structure under dead and live loads.

DOUBLE-WEB TOWN LATTICE TRUSSES AS RAILROAD BRIDGES

Historic Context

Railroads required stiff, high-capacity bridges to withstand the large, dynamic live loads imposed by moving trains. Town's first patent of simple diamond lattice webs with a recommended span-to-height ratio of 10:1 and a 45-degree angle for the lattice members would not have been stiff

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[22] Prof. Benjamin Shafer and Rachel Sangree of Johns Hopkins University made up the field load testing team.

[23] Robert L. Brungaber and Leonard Morse-Fortier, *Wooden Peg Tests – Their Behavior and Capacities as Used in Town Lattice Trusses*, Vermont Department of Transportation, McFarland-Johnson consultant, Philip C. Pierce, project manager, tests performed at Massachusetts Institute of Technology, 1995.

enough for railroad service.[24] Timber was considered a safe building material, since it gave evidence of distress long before failing. In regions with abundant timber supply, it was chosen for use over iron up to the beginning of the twentieth century. New England had numerous saw mills that could furnish lumber for Town lattice trusses, using wood from the area's ample spruce forests, a specie that was ideal for truss structures. At the same time, iron production was limited in the region. These economic considerations had to be traded off against engineering challenges, too, especially as locomotive and train weights increased.[25] Town's double-web lattice, for which he received a patent in 1835, provided the needed stiffness while retaining the lattice truss's ease of construction. Together, these made wood a popular choice for New England bridges.

Although more technically advanced trusses were available to bridge designers by the time the Contoocook Railroad Bridge was built, and some critics disdained the Town truss. J. P. Snow, engineer of the Boston & Maine Railroad and builder of this bridge, felt otherwise:

> this style of bridge seems never have been developed to much extent outside of New England, and it is frequently referred to as peculiarly unscientific and wasteful of timber. It is however, the best of the purely wooden bridges, and its present survival here and its economy over all other types disproves its wastefulness.[26]

The most common and successful method of increasing a Town lattice truss's capacity and stiffness was to build them with an initial upward bow, or camber. J. G. James believed that railroads could not tolerate camber, but Snow countered that designs with 1" of camber for each 25' of length presented no problems.[27]

In the same region, David Hazelton designed and built a number of double-web Town lattice railroad bridges, such as Warren Bridge, using triple lower and upper chords (Figure 2). Snow surely appreciated Hazelton's activity, though he was a bridgewright instead of a trained engineer, but he questioned the concept, noting that, "… the tertiary chord has but little theoretical value, and judging by the amount that the joints are pulled they assist but little in carrying the chord strain."[28]

In comparison to the variety of improved double-web Town lattice structures, the Contoocook Railroad Bridge represents the "patent" well. Its analysis can provide a clear understanding of both the strengths and weaknesses of Town's design and evaluate the necessity and ingenuity of

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[24] James, *The Evolution of Wooden Bridge Trusses to 1850.*
[25] "In 1895, a single-track bridge of 120-foot span cost about $ 5,300 in iron, … but only $ 3,500 for a spruce lattice," see Historic American Engineering Record (HAER), "Wright's Bridge," HAER No. NH-35. "During the 1880s when Cooper first introduced his loading system, bridges were usually designed for loadings no greater than Cooper's E20. By 1894 Cooper was recommending the use of his E40 loading as a standard…," see William D. Middleton, *Landmarks on the Iron Road* (Bloomington, IN: Indiana University Press, 1999), 9.
[26] Snow, "Wooden Bridge Construction," 31-43.
[27] James, *The Evolution of Wooden Bridge Trusses to 1850*, 175; Snow, "Wooden Bridge Construction," 31-43, Also J. P. Snow, "A Recent All-wood Truss Railroad Bridge," *The Engineering Record.* 60, no. 17 (October 23, 1909): 456-457.
[28] Snow, "Wooden Bridge Construction," 36.

the "enhancements" by Hazelton and Snow. A comparison between Figures 1, 2, and 3 reveal some of these changes.

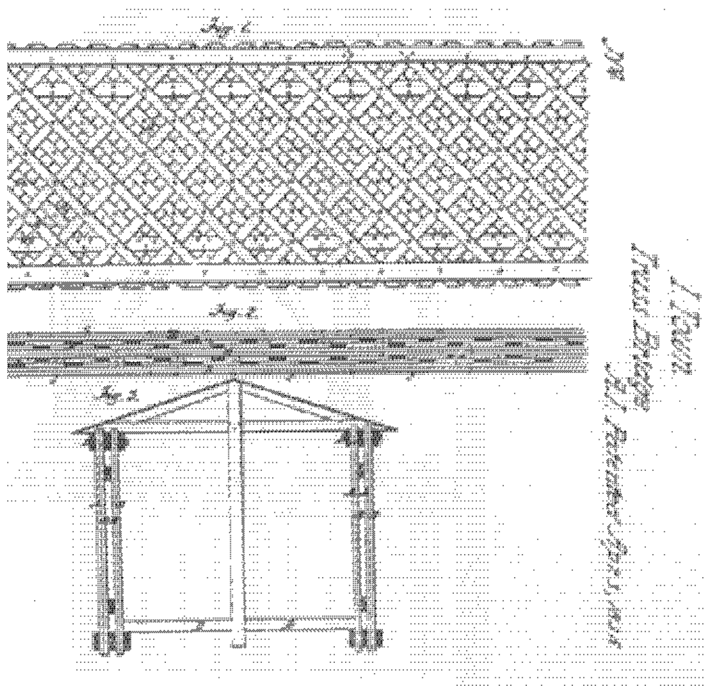

Figure 1. Town's 1835 patent

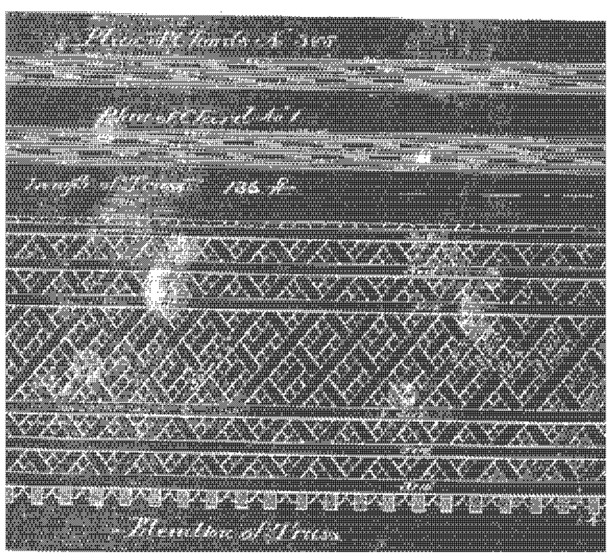

Figure 2. Hazelton's design for the Warren Bridge

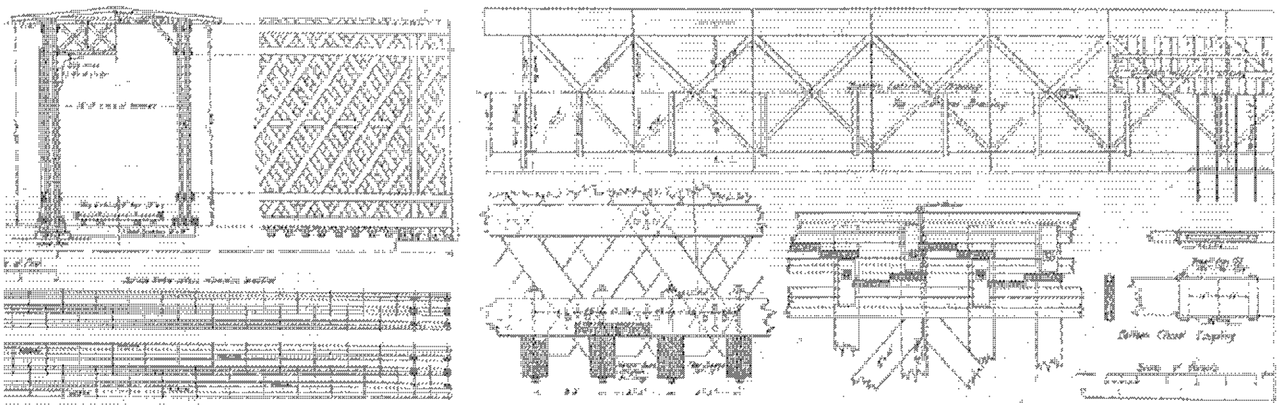

Figure 3. Snow's standard design

Classical Simplified Analysis of Town Lattice Structures

Due to their numerous interconnections, Town lattice trusses are statically indeterminate structures. As such, a thorough and accurate analysis of their behavior requires complex techniques that include individual-member-segment deformations in the stress calculations, or that deconstruct the truss into a series of Warren trusses. In the era before computers, most manual calculations used a simplified method known as equivalent beam analyses. In this method, the bending moment was broken into axial tension and compression couples with shear taken as axial forces by web members.

Historic Load Conditions

While a number of methods have been used over the years for determining the design loads for bridges, by far the most popular for railroad bridges has been the rating system developed by Theodore Cooper about five years before the Contoocook Bridge's construction. It was based on the driving axle weights of 2-8-0 steam locomotives, the most common type of the time. Designers reapportioned the axle weights of other locomotive wheel arrangements to an equivalent 2-8-0 to determine the appropriate Cooper rating. Snow provided an example of this for one 1895 bridge, where, "25,000 lb on each of three axles [and a] 44 ft wheelbase for engine and tender" corresponded to Cooper's E10 rating, as shown in Figure 4.[29] In Cooper's scheme, the "E" indicates a locomotive, or "engine," and the number specifies the weight on each driving axle in thousands of pounds. Note that the specification has two locomotives pulling the train.

[29] "Talking about a 111½' span bridge 17½' deep apart of centers also giving the total weight of 100.000 board feet and 6000 lb iron," in Snow, "Wooden Bridge Construction," 31-43.

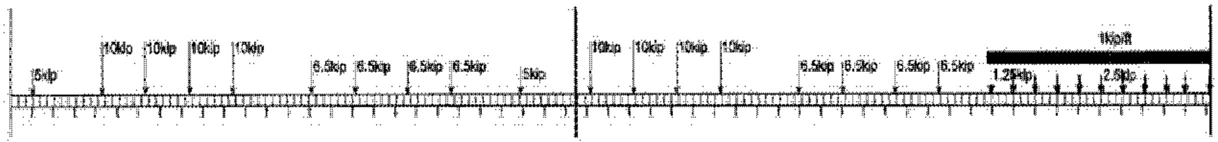

Figure 4. Cooper's E10 load distribution[30]

Historic Allowable Stresses

While material properties and performance capabilities were only partially understood in the 1880s and 1890s, experience and what little theory there was gave designers useful information they could use with a reasonable degree of confidence. The most important parameter was the material's allowable stress under different types of loads. Table 1 lists the values Snow used for one bridge he designed and built about 1909.

Table 1. Typical allowable stresses for eastern spruce[31]

Allowable tension on net section	Allowable compression on gross section	Maximum flexure stress in floor beams	Maximum shear	Crushing pressure under washers	Maximum shear on oak treenails
1000 (800) psi	700 (650) psi	1200 psi	100 (80) psi	360 psi	500 psi

Though they began as empirical designs, double-web Town lattice trusses for railroad service constructions became well-designed structures using the most advanced structural analysis methods of the time. Taking a holistic approach to their design, builders were able to detail all the sub-structures and joints needed to construct a reliable bridge.

[30] Middleton, *Landmarks on the Iron Road*, 1999.
[31] Values of the table are from Snow, "A Recent All-wood Truss Railroad Bridge," 456-457. Values in parentheses represent spruce design values given in Snow, "Wooden Bridge Construction," 31-43.

THE CONTOOCOOK RAILROAD BRIDGE

Geometry of the Bridge

The Contoocook Railroad Bridge is a double-web Town lattice truss railroad bridge, continuous over a central pier to form two spans. The central pier is skewed to match the stream flow, but it still creates approximately equal spans of 71' in length. The two trusses are similar, but not symmetrical about the pier. No lattice members are present for approximately the width of two panels at the southeast and northwest corners of the bridge.

The total length of the bridge is 157'-3". The height between the axis of the lower and upper chords is 19'-5¾".

The overall depth-of-truss / span ratio is 1/3.56. The overall behavior of the truss is, therefore, more influenced by shear than bending. Even the single span ratio of 1/7.32 is quite high. This is due, at least in part, to the greater height of railcars over road vehicles.

The overall length of the floor beams is 21', and the free space between inner lattice truss faces is 15'-3^{3}/8".

Trusses placed in mirror are generally considered to have good lateral stability. This was actually shown by the box-girder behavior of the Contoocook Bridge during floods and hurricanes that affected the bridge twice during its lifespan.[32]

Sub-structures

Both the floor system and track details are according to the common design of the time.[33] Floor beams are suspended under the primary lower chord of the truss with 1¼ to 1½-inch iron rods with washers and timber blocks so that the floor beams are alternately loading the inner and the outer trusses. The 10" x 15" floor beams are spaced between 2'-5" and 2'-7" on centers.

The lower and upper laterals are modified Howe trusses, also according to standard design practice of the time, but the field inspection revealed a couple of unusual items. The lower diagonals are not continuous, but stopped and nailed to stringers, and the upper lateral braces have no iron ties, but rather timber ties at each lattice joint.

The roof structure consists of principal and secondary rafters. Transverse bracing contributing to lateral stability is achieved through knee braces. The roof is covered with sheet metal at the

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[32] C. Philip Pierce, *Covered Bridge Manual*, draft, 14-22. In a box girder, the entire structure, including the roof floor, and trusses, essentially form a stiff box and that acts as a unit to carry the vertical and horizontal loads.

[33] The Plan of Standard Boston & Maine Railroad lattice truss supplied by J. P. Snow within his article in the *Journal of the Association of the Engineering Societies*, 1895, presents the same floor system solution, as it can be also found at the other two double Town lattice webs survived in the Sugar river-valley: Pier Bridge and Wright's Bridge Newport, NH.

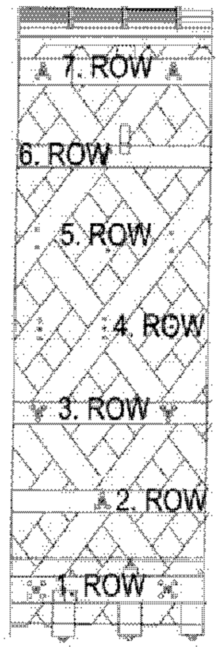
present time, although it presumably was originally shingled, as was the common practice at the time.

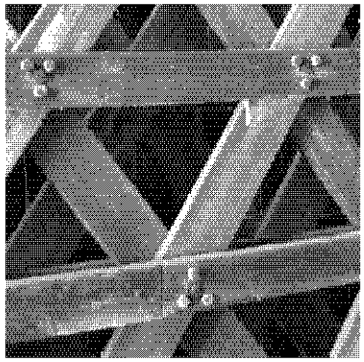
Characteristic Features

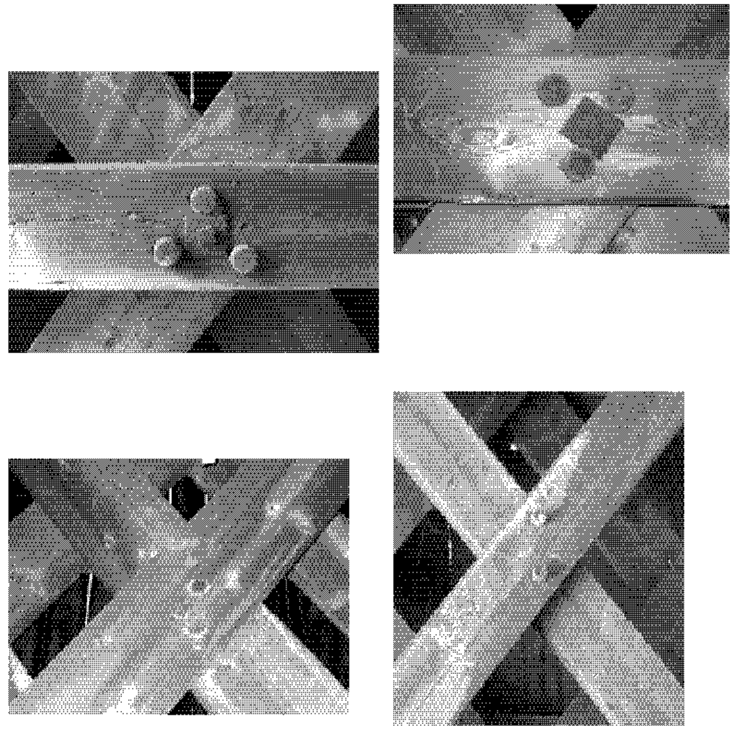
Comparing the Contoocook Bridge to other surviving double-web Town lattice structures, as well as to standard designs published in the specialized literature, reveals a number of features that do not seem to be standard ones for the type, but appear to be details chosen by Snow to better suit this bridge to its site and function as a railroad bridge. These include:

- three lower chords instead of the patented and general two. Although it has only two sets of upper chords instead of Hazelton's version of three lower and three upper chords.[34]
- seven lines of intersections. Snow's published design includes nine rows of joint lines.
- a 2'-6" floor-beam spacing, combined with 4'-10¾" dimension. In Snow's published design, the inter-axis displacement of floor beams is 2'-3", with a lattice panel dimension of 4'-6".
- A variety of treenail group patterns. Figure 5 shows the various patterns employed.

[34] Snow also mentioned bridges built recently (before 1895) with three sets of chord, as well as those built by Hazelton, "... although they were built without engineering advice, they bear analysis well, with the possible exception on the bottom chords.", Snow, "Wooden Bridge Construction," 36, 39.







Joints with 3/8" bolt at the middle:

4 treenails in a square – lower primary chord

3 treenails – triangle with base at bottom – secondary lower chord and primary upper chord

3 treenails – triangle with base on top – secondary top and tertiary lower chords

2 treenails – 4th row of intersections

Joints without bolt:

2 treenails – 5th line of lattice intersections

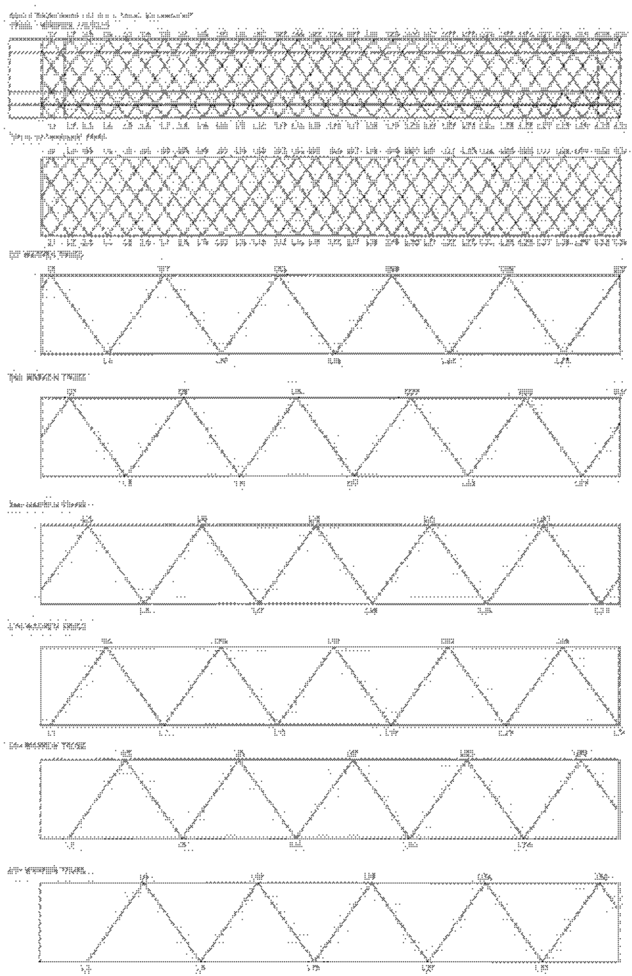
Figure 5. Different treenail-joint patterns used in the Contoocook Railroad Bridge

Structural Determinacy and Simplified (Warren) Analysis

Town lattice trusses are statically indeterminate structures. For a single inner truss, the degree of indeterminacy (N) can be determined as follows:

$$N = m + r - 2j = 527 + 4 - (2 \times 227) = 77$$

where m = number of members, r = number of support reactions, and j = number of joints.



One single lattice is 77 times indeterminate. As there are connections between the two lattice webs as well, a double-web Town lattice truss would be more than twice as indeterminate as a simple lattice truss.

Theoretically, the Warren analysis makes these bridges computable by historic, hand calculations. Presumably, this, or an equivalent-beam analysis, was done for this bridge.

Figure 6. Deconstruction of a Town lattice truss into several Warren trusses to simplify the analysis

Present Condition

Examination by the Forest Products Laboratory confirmed that the bridge was built using locally cut eastern spruce, a desirable specie for Town lattice truss construction. It has held up well. The bridge, being oversized from engineering point of view, is in very good condition. There is no sign of deflection at mid-span. The high degree of stiffness originally built into this bridge is no doubt a major reason for this good overall condition. There are some minor problems that do not have a significant influence on the bridge's overall behavior. They can be grouped into two categories: problems related to the original design and construction, and problems related to later decay and repairs.

Problems Related to the Original Design and Construction

These problems stem from the butt joints use to connect the chord members. Some of the planks are fixed with 3/8-inch bolts, but they are not adequate in size or number to carry the chord's tension loads. They may have been useful during assembly of the truss, presumably done while the timber was green (workability being better), to prevent independent lateral bending of the planks.[35] Figure 7 shows one of these partially bolted splices. The structure contains symmetrically bolted (most frequent), half bolted, and simple butt joints with no bolts.

A number of splices are placed in such a way that they interfere with the lattice-chord joints, making the joint weaker. Although the splices have been surveyed and these interferences noted, they were not introduced into the 3D model except as reduced member cross-sections.

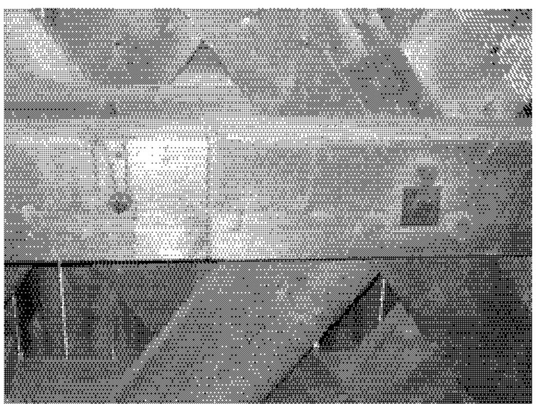


Figure 7. Partially bolted splice

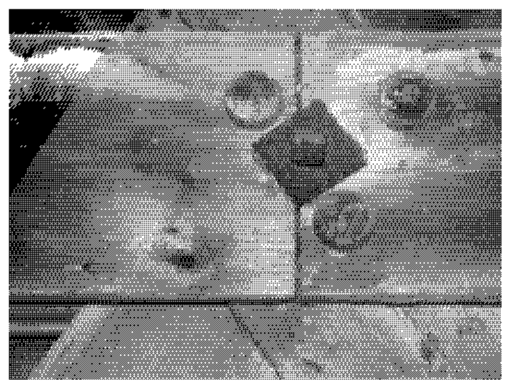


Figure 8. Splice interfering with a chord-lattice joint

Problems Related to Later Decay and Repairs

A number of lattice webs present longitudinal (shear or shrinkage) cracks that were reinforced by the addition of iron clamps to hold the two parts firmly together (Figure 8). On the level of the secondary lower chord, there is a similar, but probably older, reinforcement intervention. Here there are clamps formed both by timber and iron elements to reinforce a cracked chord member (Figure 9). This intervention might be of the same age as the bridge.

[35] Snow's report actually says the same about the ¾" bolts for lattice joints but takes into consideration 800 pounds in transmission of tension forces, see Snow, "Wooden Bridge Construction," 31-43.

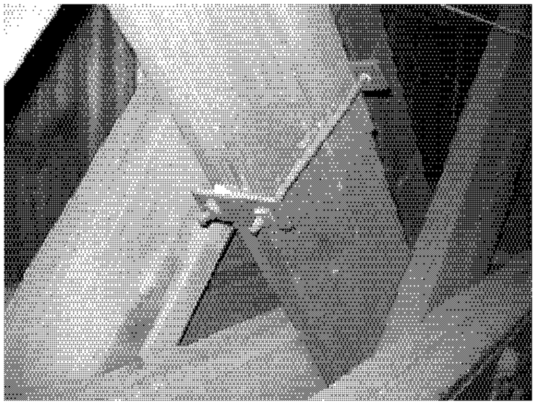


Figure 9. Lattice reinforcement using an iron clamp

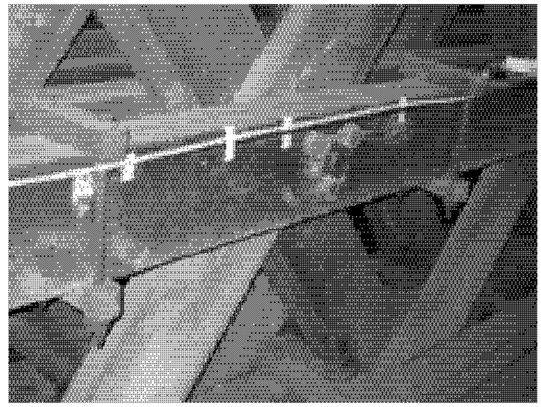


Figure 10. Secondary lower chord reinforcement with clamps

The northern truss has evidence of more intervention. There are several of the iron clamps just described, but major decay is marked by the following interventions as well:

- extra hanger beams of 12' x 11¾" square were inserted at both abutments (floor beams 56, 57, and 58 were hung at the western side to the extra hanger beam, being supported by the abutment and beam 56). The same system was repeated at east side for floor beams 1, 2, and 3.
- the same hanger system was used at the pier for floor beams 27, 28, and 29, which were suspended from the hanger beam that is supported by beams 26 and 30.

Structural Analysis of the Contoocook Railroad Bridge

The overall beam behavior of Town lattice bridges generally has been exhaustively detailed in the engineering report addendum to HAER's report on the Brown Bridge (see HAER No. VT-28), therefore this analysis focused on other questions about the Town lattice truss–especially double-web lattice structures. The primary investigative process involved mathematical modeling using SAP2000 linear finite element analysis software, supplemented by Axis VM7 software.

The questions to be resolved, along with descriptions of the investigative model variants, are as follows:

1. How does the existence of splices influence the overall behavior of the structure (stiffness, mid- span deflection under dead load)?

   Two-dimensional models created to identify the differences with the different applicable combinations of rigid (infinite stiff joints) and pinned (rotationally flexible) joints in the primary lower and upper chords and lattice intersections as follows:

Var 1. 2D inner lattice, stiff joints, full section of chords.
Var 2. 2D inner lattice, stiff joints, reduced section of chords (50% for chords made up of two elements, and 33% where chord is made up of three elements).
Var 3. 2D inner lattice, pinned joints at lattice ends, reduced section of chords.
Var 4. 2D inner lattice, pinned joints at lattice ends, reduced section of chords, single span (No central pier, but the same load conditions).

2. Would it have been better, i.e., more economical, to develop a proper joint for tension transmission instead of adding the tertiary lower chord?

   Var 5. 2D inner lattice, pinned joints at lattice ends, reduced section of chords, tertiary chord missing (to identify the efficiency of introducing the tertiary lower chord versus tension joints at the splices).

3. Would a single-span bridge have been preferable to the two-span bridge actually built?

   Some of the historic evidence mentions the Contoocook Railroad Bridge as a 157' single-span structure, so the differences between the actual double-span bridge and an equivalent single-span version have been compared.

4. Were the designer's choices for web member angle, number of intersections, and truss proportions the optimum ones?

5. How much did doubling the lattice truss affect its overall stiffness?

   This involved the construction of the following two-dimensional and three-dimensional models:

   Var 6. 2D both lattice webs, pinned joints at lattice ends, reduced section of chords.
   Var 7. 3D inner lattice webs, mathematically modeled joint stiffness, both total and reduced sections of chords.
   Var 8. 3D both lattice webs, mathematically modeled joint stiffness, both total and reduced sections of chords.

Historic structures, even if they are engineered or semi-engineered ones, are not (or at least not perfectly) regular, so it was necessary to decide whether to model the truss as ideal, with parallel lines and perfect joints, or as a deformed structure with all of its irregularities.

Since the differences/deformations are small enough to be irrelevant to the overall behavior from a structural point of view, the models were constructed as ideal structures to simplify the calculations. To introduce real joint stiffness, a 3D model of one double-web lattice truss was created.

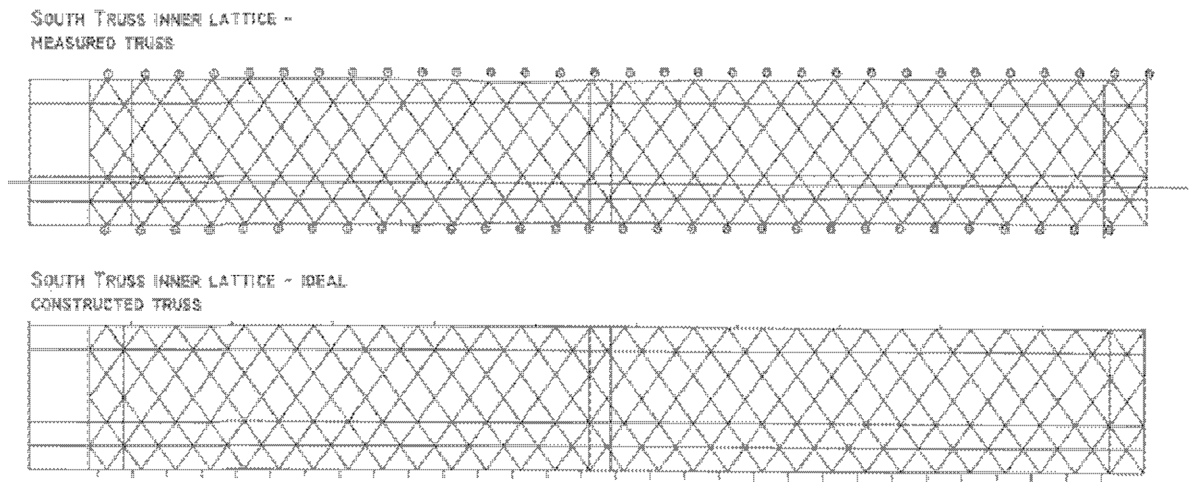


Figure 11. Comparative model of the real (measured) and ideal structure, as used for present report

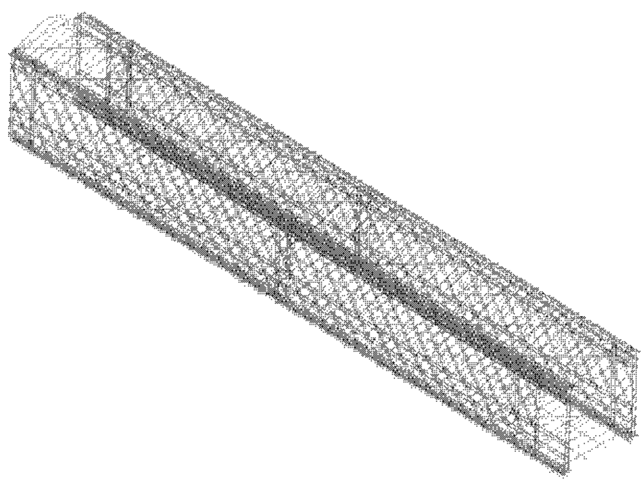


Figure 12. 3D model of the bridge

The bridge was built using eastern spruce, which has the mechanical properties listed in Table 1.

Table 1. Mechanical properties of eastern spruce

	kip in	lb ft
Mass per unit volume	$4.40E^{-08}$	0.9133
Weight per unit volume	$1.70E^{-05}$	29.3976
Modulus of elasticity	1340.277	$1.93E^{+08}$
Poisson ratio	0.372	0.372

Table 2 lists the section characteristics for both full and reduced cross sections of the chords.

Table 2. Section characteristics for chords

Var 1. Full section						Var 2. Reduced sections					
No.	Name of section	Width	Depth	Area	I	No.	Name of section	Width	Depth	Area	I
		in	in	in^2	in^4			in	in	in^2	in^4
1	Chord total: main chord lower + upper	11	11.75	129.25	1487.048	1	Chord total: main chord lower + upper	8.25	11.75	96.9375	1115.286
2	Second. chords total: secondary chords lower + upper	8.25	9.75	80.4375	637.2158	2	Second. chords total: secondary chords lower + upper	5.5	9.75	53.625	424.8105
3	Lattice	2.75	11.75	32.3125	371.762	3	Lattice	2.75	11.75	32.3125	371.762
4	Stud (post)	5.5	9.75	53.625	424.8105	4	Stud (post)	5.5	9.75	53.625	424.8105

Load conditions

The load conditions for the two-dimensional and three-dimensional models were determined for the following cases:

> Load case 1. Dead load is self-generated for the structural elements, the weight of other elements are added.
> Load case 2. Ideal mid-span concentrated load (result of equivalent beam analysis) of 124 kip.
> Load case 3. Cooper's E20, representing the double of the values given in Figure 5.
> Load case 4. Reduced concentrated mid-span force (result of equivalent beam analysis) of 74 kip.
> Combination of dead load + Cooper's E20 live load (Table 3).

Since the main focus of this investigation was the identification of the element behavior versus static methods, no wind load or snow load conditions were taken into consideration.

Total weight of wooden members and steel rails	241.62 kip
Total dead load (including 10% for misc. components)	265.78 kip = 1.69 kip / ft

Table 3. Cooper's E20 live load

No.	Name of layer	E10 / axis	n	E20 / axis	n	E40 / axis	kip
		kip		kip		kip	suspension point
1	Concentrated load / engine axis	5	2	10	4	40	5
2	Concentrated load / engine axis	10	2	20	4	40	10
3	Concentrated load / engine axis	6.5	2	13	4	26	6.5
4	distributed linear load on each beam	2.5	2	5	4	10	2.5

Equivalent Beam Method

The equivalent beam method is a manual method of calculating the overall stiffness of a structure by assuming it to be an equivalent homogeneous beam. These calculations were done to identify possible live loading for on-site testing and to serve as a comparison basis for other methods.

1. All chords considered continuous and having a full section.
2. Chord areas reduced 50 percent (doubled elements) and 33 percent (tripled elements).

Table 4. Equivalent inertia and load needed to achieve 1/8" deflection at mid-span of 70' single- span equivalent beam

No.	Neutral axes	Equivalent inertia (*I*)	Uniform load (plf)	Concentrated load (kip)
1.	y = 110.12 in	9154322.54 in^4	19.71	124.178
2.	y = 109.67 in	5456137.48 in^4	11.75	74.012

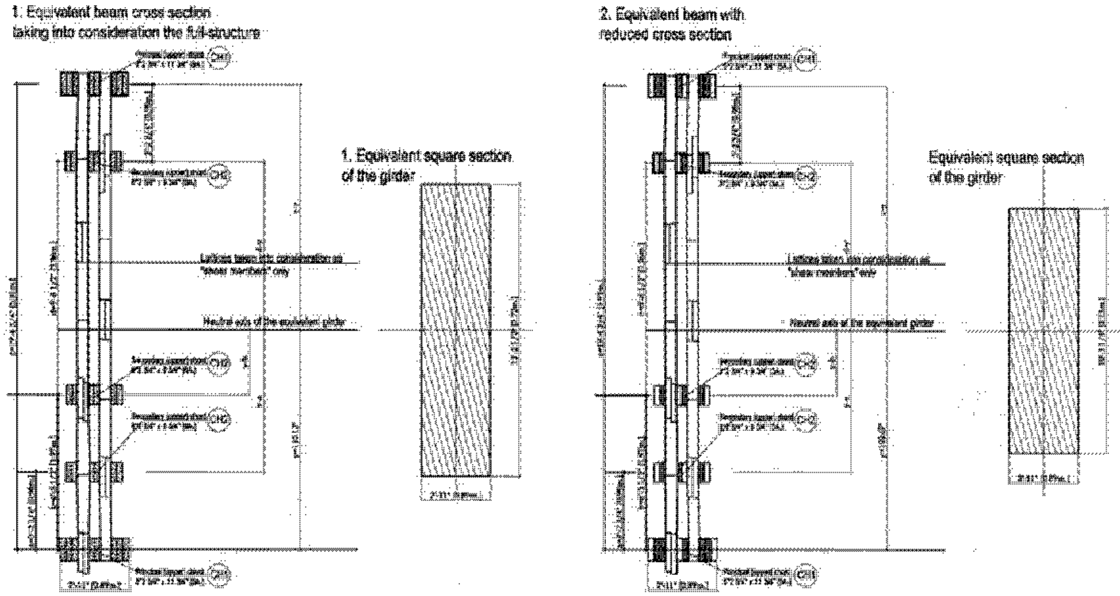


Figure 13. Equivalent beam sections – inertia calculation

These results show that the stiffness of condition 2 (spliced chords), measured in concentrated force applied at mid-span to achieve 1/8" deflection, is 40.4 percent less than that of condition 1 (continuous chords). Even with the condition 2, a 74-kip load would be required to achieve a 1/8" (the minimum measurable) deflection. This led the team to cancel the idea of load testing the bridge to measure overall deflection.[36]

As some of the historic evidences mention Contoocook Railroad Bridge with a single 157' span, the merits of this hypothesis were considered throughout this report. Even with such a span, the stiffness (measured in middle span deflection) of the double-web Contoocook Bridge is three times greater than that of a single-web Town lattice bridge commonly used in roadway service.[37]

Comparison of Two-Dimension Finite Element Models Under Dead and Live Load Conditions

For the inner lattice analysis (Var 1, Var 2, Var 3, and Var 4), a comparison of the trusses' behaviors was made on the basis of analyzing the following characteristics:

– deflection at mid-span under various load conditions.

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[36] See Historic American Engineering Record (HAER), Taftsville Bridge," HAER No. VT-30. On this bridge, field load testing was carried with 11.8kip, appreciatively six time less than the one needed at Contoocook;
[37] Historic American Engineering Record (HAER), "Bath-Haverhill Bridge," HAER No. NH-33, 2003.

- maximum axial force (tension and compression) in the characteristic members (L-1 = principal lower chord; L-2 = secondary lower chord; L-3 = tertiary lower chord; U-1 = principal upper chord; U-2 = secondary upper chord; D = lattice diagonal).
- maximum bending moment in the same elements.

The results of the finite element analysis are summarized in Tables 4 through 8.

Table 4. Mid-span deflection comparison data

No	Value to compare	Load case	Var 1. – Stiff joint + full chord	Var 2. – Stiff joint + reduced chord	Var 3. – Pin joints + reduced chord	Var 4. – Pin joints + reduced chord single span
1.1	Mid-span deflection (in)	dead load	-0.071271	-0.072544	-0.073227	-0.960914
1.2		concentrated load at mid span	-0.537686	-0.585876	-0.592842	-2.968619
1.3		Live load Cooper's E20	-0.140883	-0.152804	-0.154254	-2.556176
1.4		reduced concentrated	-0.320877	-0.349636	-0.353793	-1.771595
1.5		Combination dead load + Cooper's E20	-0.212154	-0.225348	-0.227481	-3.51709

These data are the values of mid-span deflection. A comparison to Var 2, which was considered to be the base (control) value, indicated the following:

- The introduction of pinned joints at lattice ends reduces stiffness only 1 percent.
- The increase of stiffness, measured in mid-span deflection, by having continuous chords (Var 1) is less than 10 percent, about one-fifth of the 50 percent estimated with the equivalent beam analysis.
- The deflection increased 4 to 15 times when the span was doubled, with less increase for a concentrated load (influenced by span on the 3rd), and the greatest increase for distributed loads (depending on span at the 4th).

Table 5. Change of stiffness

Load case	Stiffness increase % Var 1/2.	Stiffness increase % Var 3/2.	Deflection increase % Var 3/4.	Deflection/span ratio (71') Var 3	Deflection/span ratio (144') Var 4
Dead load	1.74	0.93	1212.24	1/11500	1/1750
Concentrated load at mid-span	8.13	1.18	400.74	1/1430	1/560
Live load Cooper's E20	7.73	0.94	1557.12	1/5500	1/657
Reduced concentrated load at mid-span	8.13	1.17	400.74	1/2400	1/948
Combination dead load + Cooper's E20	5.80	0.94	1446.10	1/3730	1/470

The deflection-to-span ratios are very low, but the inner joints were modeled as stiff joints, which increased the calculated stiffness.

Table 6. Axial forces in chord members

No	Value to compare	Load case	Stiff joint + full chord	Stiff joint + reduced chord	Pin joints + reduced chord	Pin joints + reduced chord single span	Change 1/2	Change 2/3	Change 4/3
2.1	Maximum tension (kip)	Dead load	8.416	8.468	8.33	–	0.61	-1.66	
2.2	PRIMARY UPPER CHORD	Concentrated load at mid-span	43.108	46.94	46.517	–	8.16	-0.91	
2.3	member id 465	Live load Cooper's E20	17.768	19.431	19.236	–	8.56	-1.01	
2.4		Reduced concentrated load at mid-span	25.726	28.012	27.76	–	8.16	-0.91	
2.5		Combination dead load + Cooper's E20	26.184	27.899	27.566	–	6.15	-1.21	
3.1	Maximum tension (kip)	Dead load	4.727	4.21	4.207	28.85	-12.28	-0.07	585.76
3.2	PRIMARY LOWER CHORD	Concentrated load at mid-span	39.02	37.346	34.903	99.929	-4.48	-7.00	186.30
3.3	member id 257/472	Live load Cooper's E20	11.548	10.973	11.026	76.184	-5.24	0.48	590.95
3.4		Reduced concentrated load at mid-span	23.286	22.287	20.829	59.635	-4.48	-7.00	186.31
3.5		Combination dead load + Cooper's E20	16.275	15.183	15.234	105.034	-7.19	0.33	589.47
4.1	Maximum tension (kip)	Dead load	1.322	1.147	1.167	8.769	-15.26	1.71	651.41
4.2	SECONDARY LOWER CHORD	Concentrated load at mid-span	-12.165	-11.604	-11.752	-3.742	-4.83	1.26	-68.16
4.3	member id 236/444	Live load Cooper's E20	4.397	4.216	4.317	22.38	-4.29	2.34	418.42

4.4	MS	Reduced concentrated load at mid-span	-7.26	-6.925	-7.013	-2.233	-4.84	1.25	-68.16
4.5		Combination dead load + Cooper's E20	5.719	5.363	5.484	31.084	-6.64	2.21	466.81
5.1	Maximum tension (kip)	Dead load	2.681	2.224	1.929	–	-20.55	-15.29	
5.2	SECONDARY LOWER CHORD	Concentrated load at mid-span	6.691	5.451	4.311	–	-22.75	-26.44	
5.3	member id 473	Live load Cooper's E20	6.656	6.003	5.276	–	-10.88	-13.78	
5.4	MP	Reduced concentrated load at mid-span	3.993	3.253	2.573	–	-22.75	-26.43	
5.5		Combination dead load + Cooper's E20	9.337	8.227	7.205	–	-13.49	-14.18	
6.1	Maximum tension (kip)	Dead load	3.556	2.952	2.974	1.651	-20.46	0.74	-44.49
6.2	THIIRD LOWER CHORD	Concentrated load at mid-span	12.089	10.2	10.262	-16.973	-18.52	0.60	-265.40
6.3	member id 475	Live load Cooper's E20	8.239	7.364	7.404	1.704	-11.88	0.54	-76.99
6.4	MP	Reduced concentrated load at mid-span	7.215	6.087	6.124	-10.129	-18.53	0.60	-265.40
6.5		Combination dead load + Cooper's E20	11.795	10.316	10.379	3.355	-14.34	0.61	-67.68
7.1	Maximum compression (kip)	Dead load	-8.086	-7.45	-7.475	-48.505	-8.54	0.33	548.90
7.2	PRIMARY UPPER CHORD	Concentrated load at mid-span	-54.639	-54.826	-55.038	-165.457	0.34	0.39	200.62
7.3	member id 225/465	Live load Cooper's E20	-24.419	-24.424	-24.497	-130.831	0.02	0.30	434.07
7.4	MS	Reduced concentrated load at mid-span	-32.607	-32.719	-32.845	-98.74	0.34	0.38	200.62
7.5		Combination dead load + Cooper's E20	-32.505	-31.874	-31.972	-179.335	-1.98	0.31	460.91
8.1	Maximum compression (kip)	Dead load	-3.314	-2.8	-2.833	-19.486	-18.36	1.16	587.82
8.2	SECONDARY UPPER CHORD	Concentrated load at mid-span	-24.014	-22.182	-22.477	-74.168	-8.26	1.31	229.97
8.3	member id 206/471	Live load Cooper's E20	-10.738	-9.815	-9.932	-53.382	-9.40	1.18	437.47
8.4	MS	Reduced concentrated load at mid-span	-14.331	-13.238	-13.414	-44.261	-8.26	1.31	229.96
8.5		Combination dead load + Cooper's E20	-14.053	-12.615	-12.765	-72.868	-11.40	1.18	470.84

Note: MS = mid-span area, MP = middle pier area.

Secondary upper chord tension values are not included in the comparative results table.

Table 7. Axial forces in lattice members

No	Value to compare	Load case	Stiff joint + full chord	Stiff joint + reduced chord	Pin joints + reduced chord	Pin joints + reduced chord single span	Change 1/2	Change 2/3	Change 4/3
9.1	Maximum tension (kip)	Dead load	4.457	4.096	4.346	7.442	-8.81	5.75	71.24
9.2	in lattice member	Concentrated load at mid-span	14.572	14.478	15.417	12.521	-0.65	6.09	-18.78
9.3	MP	Live load Cooper's E20	13.636	13.509	14.472	19.263	-0.94	6.65	33.11
9.4	515/902	Reduced concentrated load at mid-span	8.696	8.64	9.201	7.472	-0.65	6.10	-18.79
9.5		Combination dead load + Cooper's E20	18.093	17.605	18.819	26.705	-2.77	6.45	41.90
10.1	Maximum tension (kip)	Dead load	1.386	1.349	1.377	0.656	-2.74	2.03	-52.36
10.2	in lattice member	Concentrated load at mid-span	65.509	53.658	56.272	65.509	-22.09	4.65	16.41
10.3	MS	Live load Cooper's E20	4.358	4.471	4.434	5.92	2.53	-0.83	33.51
10.4	251/460	Reduced concentrated load at mid-span	31.692	32.022	33.582	39.094	1.03	4.65	16.41
10.5		Combination dead load + Cooper's E20	5.744	5.82	5.811	6.576	1.31	-0.15	13.16
11.1	Maximum compression (kip)	Dead load	-8.552	-8.132	-8.399	-13.027	-5.16	3.18	55.10
11.2	in lattice member	Concentrated load at mid-span	-30.744	-31.794	-32.807	-22.653	3.30	3.09	-30.95
11.3	MP	Live load Cooper's E20	-19.541	-20.166	-20.704	-30.108	3.10	2.60	45.42
11.4	459/924	Reduced concentrated load at mid-span	-18.347	-18.974	-19.578	-13.519	3.30	3.09	-30.95
11.5		Combination dead load + Cooper's E20	-28.115	-28.319	-29.124	-43.113	0.72	2.76	48.03
12.1	Maximum compression (kip)	Dead load	-0.267	-0.283	-0.318	0.031	5.65	11.01	-109.75
12.2	in lattice member	Concentrated load at mid-span	-19.833	-19.902	-23.142	-13.603	0.35	14.00	-41.22
12.3	MS	Live load Cooper's E20	1.594	1.477	1.592	2.662	-7.92	7.22	67.21
12.4	278/515	Reduced concentrated load at mid-span	-11.836	-11.877	-13.811	-8.118	0.35	14.00	-41.22
12.5		Combination dead load + Cooper's E20	1.328	1.194	1.274	2.693	-11.22	6.28	111.38

The first two models with only stiff joints contributed to bending moment concentrations on abutment studs and chords, but this was unrealistic, and as such was not considered further. Even versions with pinned joints at the lattice ends introduce a bending moment concentration at single support points. This should be different in reality, due to more support points between the abutment and bolster beam. "Bolster beam," as used here, represents a distributed line of support for the bridge that could not be modeled with the software used. Therefore, large negative values of support bending moments were not included in the table below.

Table 8. Bending moment comparison table

No	Value to compare	Load case	Stiff joint + full chord	Stiff joint + reduced chord	Pin joints + reduced chord	Pin joints + reduced chord single span	Change 1/2 %	Change 2/3 %	Change 4/3 %
13.1	Mid-span bending moment (kip-ft)	Dead load	0.735	0.5352	0.2963	1.6458	-37.26	-80.63	455.45
13.2	PRIMARY BOTTOM CHORD	Concentrated load at mid-span	3.348	2.973	1.8037	3.2495	-12.62	-64.83	80.16
13.3	MP	Live load Cooper's E20	5.471	5.3106	5.1784	4.4966	-3.02	-2.55	-13.17
13.4	440/58	Reduced concentrated load at mid-span	1.998	1.7742	1.0764	1.9392	-12.61	-64.83	80.16
13.5		Combination dead load + Cooper's E20	5.952	5.7547	5.4747	6.0987	-3.43	-5.11	11.40
14.1	Mid-span bending moment (kip-ft)	Dead load	0.275	0.2501	0.2709	0.5464	-10.08	7.68	101.70
14.2	PRIMARY BOTTOM CHORD	Concentrated load at mid-span	-6.120	-5.8892	-8.2746	-5.2308	-3.92	28.83	-36.78
14.3	MS	Live load Cooper's E20	1.046	0.9752	1.0879	4.9692	-7.24	10.36	356.77
14.4	285/496	Reduced concentrated load at mid-span	-3.652	-3.5145	-4.9381	-3.1216	-3.92	28.83	-36.79
14.5		Combination dead load + Cooper's E20	1.153	1.0508	1.1669	5.5157	-9.75	9.95	372.68
15.1	Mid-span bending moment (ki-pft)	Dead load	-1.198	-1.0185	-0.7315	-3.5714	-17.66	-39.23	388.23
15.2	PRIMARY TOP CHORD	Concentrated load at mid-span	-3.206	-2.7511	-1.7342	1.7429	-16.54	-58.64	-200.50
15.3	MP	Live load Cooper's E20	-1.611	-1.3742	-0.8636	2.6264	-17.22	-59.12	-404.12
15.4	463/47	Reduced concentrated load at mid-span	-1.913	-1.6418	-1.0349	1.0401	-16.54	-58.64	-200.50
15.5		Combination dead load + Cooper's E20	-2.809	-2.3927	-1.5951	-0.6382	-17.42	-50.00	-59.99
16.1	Mid-span bending moment (kip-ft)	Dead load	0.568	0.5368	0.5424	0.8052	-5.85	1.03	48.45
16.2	PRIMARY TOP CHORD	Concentrated load at mid-span	0.604	0.4273	0.6042	1.4801	-41.40	29.28	144.97
16.3	MS	Live load Cooper's E20	0.330	0.2862	0.3041	0.9564	-15.37	5.89	214.50
16.4	253/518	Reduced concentrated load at mid-span	0.361	0.255	0.2796	0.8833	-41.41	8.80	215.92
16.5		Combination dead load + Cooper's E20	0.898	0.823	0.8465	1.7574	-9.16	2.78	107.61
17.1	Minimum bending moment (kip-ft)	Dead load	-1.115	-0.7874	-0.7185	-0.6494	-41.66	-9.59	-9.62

17.2	SECONDARY BOTTOM CHORD	Concentrated load at mid-span	-3.895	-2.9782	-2.72	-1.3043	-30.79	-9.49	-52.05
17.3	MP	Live load Cooper's E20	-2.705	-2.0635	-1.8603	-1.8157	-31.09	-10.92	-2.40
17.4	500/95	Reduced concentrated load at mid-span	-2.325	-1.7773	-1.6232	-0.7784	-30.79	-9.49	-52.05
17.5		Combination dead load + Cooper's E20	-3.820	-2.8509	-2.5789	-2.465	-34.01	-10.55	-4.42
18.1	Minimum bending moment (kip-ft)	Dead load	-0.757	-0.5218	-0.5348	-0.8233	-45.11	2.43	53.95
18.2	THIRD BOTTOM CHORD	Concentrated load at mid-span	-2.789	-2.0824	-2.1318	-1.7162	-33.93	2.32	-19.50
18.3	MP	Live load Cooper's E20	-1.682	-1.2735	-1.2735	-2.3212	-32.06	0.00	82.27
18.4	474/97	Reduced concentrated load at mid-span	-1.664	-1.2427	-1.2722	0.9828	-33.93	2.32	-177.25
18.5		Combination dead load + Cooper's E20	-2.439	-1.7688	-1.8083	3.1061	-37.89	2.18	-271.77
19.1	Minimum bending moment (kip-ft)	Dead load	-0.564	-0.4112	-0.3831	-0.6837	-37.14	-7.33	78.47
19.2	SECONDARY TOP CHORD	Concentrated load at mid-span	-2.175	-1.72	-1.6026	-1.3157	-26.44	-7.33	-17.90
19.3	MP	Live load Cooper's E20	-1.080	-0.8578	-0.8008	-1.8159	-25.93	-7.12	126.76
19.4	498/93	Reduced concentrated load at mid-span	-1.298	-1.0264	-0.9564	-0.7852	-26.44	-7.32	-17.90
19.5		Combination dead load + Cooper's E20	-1.644	-1.269	-1.1839	-2.4996	-29.56	-7.19	111.13
20.1	Minimum bending moment (kip-ft)	dead load	-0.349	-0.3659	-0.3708	0.5519	4.76	1.32	-248.84
20.2	LATTICE MEMBERS	concentrated load at mid span	-1.245	-1.4036	-1.4219	0.9923	11.27	1.29	-169.79
20.3	MP	Live load Cooper's E20	-0.770	-0.8781	-0.8928	1.5534	12.37	1.65	-273.99
20.4	455/79	reduced concentrated	-0.743	-0.8376	-0.8485	0.5922	11.27	1.28	-169.79
20.5		Combination dead load + Cooper's E20	-1.118	-1.244	-1.2635	2.1054	10.13	1.54	-266.63

These data suggested the following observations and conclusions:

The maximum tension was in the upper primary chord, above the central pier. It was 1.98 times greater than the maximum value in the lower primary chord, under dead load condition. (The perfect beam behavior for uniformly distributed load produced a 1.77 ratio for bending moments).

Under the mid-span concentrated load, the ratio was 1.33 for the concentrated force (compared to 1.2 for the perfect beam behavior's bending moment ratio).

The axial force distribution data showed that the overall behavior was closer to a self-formed arch with tension tie than to a continuous girder. The same conclusion could also be derived by comparing maximum tension values in the lower primary chord to compression values in the upper primary chords. These were not equal, as equivalent beam theory would suggest.

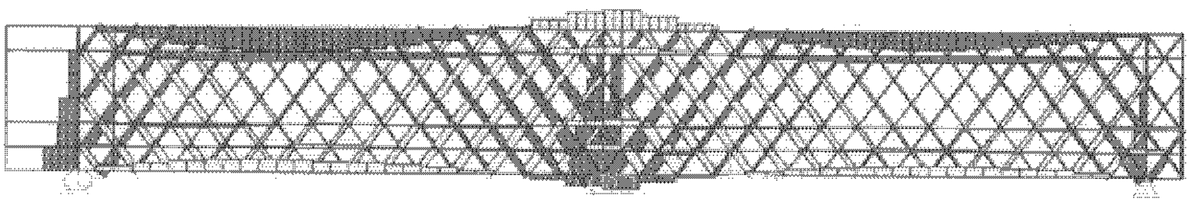

Figure 14. Axial force diagram at Combination 1, dead load + Cooper's E20

The maximum tension in the primary upper chord was more influenced by the lack of continuity in the lower chord than the overall stiffness; 8.65 percent in comparison to 7.73 percent versus control.

The same compression diagram that is specific to the overall behavior of the Town lattice truss (compression arch line) repeated locally for the secondary and tertiary lower chords. In the presence of a large concentrated force, the adjacent chord members became compressed.

The maximum tension in secondary lower chord above the pier represented 46 to 48 percent of the maximum mid-span tension in the primary chord. The mid-span value of L-2 was only 28 to 38 percent of that of L-1. The efficiency of secondary lower chord above the pier was reduced significantly (11.82 - 20.31 percent) due to the lack of continuity in the lower chord elements. The introduction of pins at the lattice ends had no major influences, except on tension in the secondary lower chord at the pier, which was reduced 13.78 percent. The axial forces of Cooper's E20 did not perturb the overall beam behavior of the secondary lower chord.

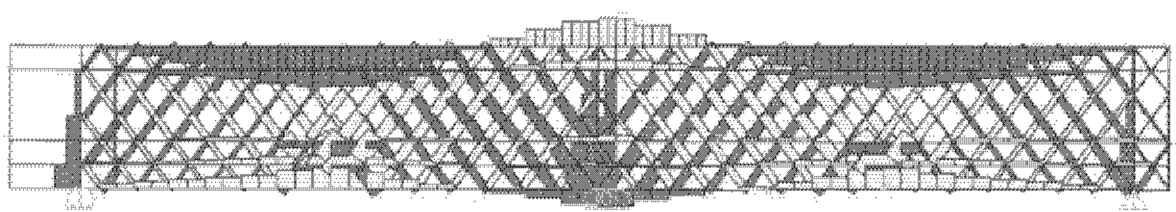

Figure 15. Axial force diagram at Load 2, mid-span concentrated force

The tertiary chord was strongly compressed for a large, local concentrated load in mid span region. For dead and Cooper's E20 live load, there was very little tension at mid span, only 0.072 kip. The tertiary lower chord was efficient above the pier, although its efficiency was reduced significantly (14.25 percent) due to the discontinuity of the chords. At quarter points the tertiary lower chord was under minor compression, both for dead and Cooper's E20 live load.

The maximum axial force in the primary upper chord was at mid span for live loads and the combination of dead plus E20 loads. The only load condition that caused more tension above the pier in the primary upper chord than the absolute value of the mid span compression was dead load alone.

As the data in Table 9 showed, lattice members became less efficient when the span was doubled and the middle support removed. In that case, the mid-span deflection increased 1,446 percent, the maximum tension in primary lower chord increased 589 percent, and the maximum compression in primary upper chord increased 460 percent. The tension in lattice members over the pier increased only 41 percent, while compression at the middle pier increased 109 percent.

Table 9. Increase of axial force for single-span versus two-span continuous truss

Load case	Single- span / double- span tension diagonal	Single-span / double-span compression diagonal
Dead load	2.34	2.45
Concentrated load at mid-span	1.40	1.32
Live load Cooper's E20	2.41	3.11
Reduced concentrated load at mid-span	1.40	1.32
Combination dead load + Cooper's E20	2.39	2.87

Doubling the span increased the tension in inclined diagonals at abutments between dead load and Cooper's E20 conditions by 2.34 to 2.41 times, and compression in declined diagonals at abutments between 2.45 and 3.11 times.

The efficiency of tension and compression lattice webs placed at mid-span (MS) versus those placed over the supports (abutments and middle pier) can be seen on the Table 10.

Table 10. Efficiency of mid-span lattice webs

Load case	Double-span tension in MP/MS lattice %	Single-span tension in MP/MS lattice %	Double-spans compression in MP/MS lattice %	Single-spans compression in MP/MS lattice %
Dead load	31.68	8.81	3.79	-0.24
Concentrated load at mid- span	365.00	523.19	70.54	60.05
Live load Cooper's E20	30.64	30.73	-7.69	-8.84
Reduced concentrated load at mid-span	364.98	523.21	70.54	60.05
Combination dead load + Cooper's E20	30.88	24.62	-4.37	-6.25

For shorter spans, tension members at mid-span were still approximately 30 percent efficient for dead load. Tension members remained similarly efficient for Cooper's E20 (30.64 - 30.73 percent).

Compression diagonals at mid-span were almost useless for dead loads, and locally could become tension members depending on the relative position of live loads. Even the secondary compression effect from an "out of scale" concentrated load was reduced by 70 percent (60 percent for the single span).

Lattice members were less influenced (3 percent) by reductions in chord members' sectional areas due to splices, except in the presence of a concentrated load, where it could increase by as much as 20 percent.

Table 11. Bending moment comparison

Load case	Bending moment above support kip-ft	ratio to mid span
Dead load	-2.58	3.51
Concentrated load at mid- span	-8.99	2.68
Live load Cooper's E20	-8.72	1.59
Reduced concentrated load at mid-span	-5.36	2.68
Combination dead load + Cooper's E20	-11.30	1.90

Negative bending moments for chords at abutments and the pier were not included in Table 11, as they represent several times greater value than the mid-span bending moment for the adjacent "span" between two lattice suspension points.

For the reduced chord section, the bending moment capacity is:

$$M_{cap} = S \times \sigma' = 10.827 \text{ kip-ft}$$

One interpretation of this is that no higher bending moment concentration can happen, thus causing first rotation and bending moment redistribution, rather than a collapse of extra loaded elements.[38]

Figures 16 and 17 show that bending moment is not a characteristic strain for mid-span secondary and tertiary chords or lattice members. Their maximum bending moment values range between 0.0511 and 0.0846 kip-ft.

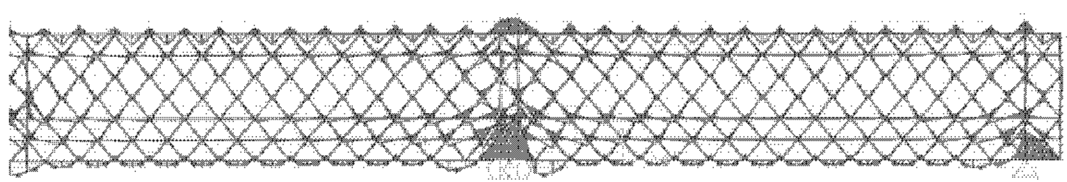

Figure 16. Bending moment diagram, Var 3, dead load

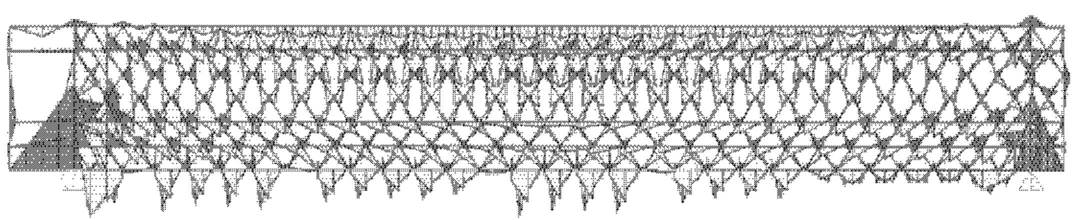

Figure 17. Bending moment diagram Var 4, dead load + Cooper's E20

[38] Bending moment capacity calculation considered a NDS max. allowable stress 775psi reduced with 205psi pressure resulting from compression of the same chord member.

Further variants, Var 5 and Var 5', were developed to model a continuous chord and no tertiary lower chord, as well as a reduced-section chord also missing the tertiary chord, to compare the efficiency of introducing tension joints to chords versus the efficiency of the tertiary chord. Table 12 presents these data.

Table 12. Comparison of efficiency of tertiary chord versus tension joints for chords

Deflection with reduced chord without tertiary chord (in)	Deflection with full chord without tertiary chord (in)	Deflection with reduced chord with tertiary chord (in)	Decrease through tension joints (%)	Decrease though tertiary chord (%)
0.072429	0.072429	0.0732	-2.33	1.10
0.620349	0.620349	0.5928	-8.24	-4.43
0.15531	0.15531	0.1543	-7.50	-0.68
0.370208	0.370208	0.3538	-8.24	-4.43
0.227739	0.227739	0.2275	-5.86	-0.11

Stiffness could be increased only 6 to 7.5 percent by introducing tension joints (according to two-dimensional modeling). From overall stiffness point of view, the tertiary chord had almost no effect, just 0.11 - 0.68 percent for live and combination loads.

Under large, "out of scale" concentrated loads, tertiary chords could have a measurable effect, but in reality, such large, concentrated forces would not be applied. Thus, J. P. Snow's opinion about the lack of efficiency of tertiary chords was confirmed.[39] They should have some overall effect on the lateral stability of the web, but the models used did not consider lateral loads.

The model without studs (posts) at the middle pier showed a reduction in overall stiffness (measured in deflection at mid-span) of 27 to 30 percent.

SUBSTRUCTURES

Suspended Floor System

The suspended floor system of the Contoocook Bridge was a common, historic design for double-web Town lattice bridges.[40] An analysis of the transmission of the maximum axle force from a floor beam to the primary lower chord was performed to determine:

- the bending and shear capacity of the floor beam.
- the perpendicular compression stress on the timber under the washers.
- the tension in the suspension rods.

[39] Snow, "Wooden Bridge Construction," 31-43.
[40] Snow, "Wooden Bridge Construction," 31-43.

Bending Capacity of the Floor Beams

All of these floor beams have the overall length of 21'. The worse static situation was with suspension points at the outer lattice webs, forming a 19' span, as shown below.

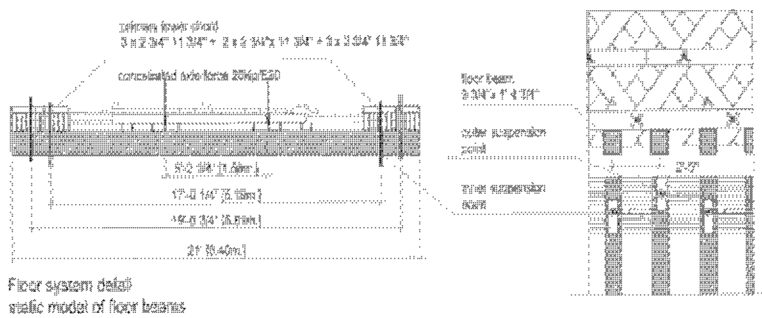

Figure 18. Floor beam detail

There were three load cases for the beam itself:

- uniform distributed load on line – self weight.
- double concentrated force from stringers, rail beam, and ties.
- Double-axle force as a live load

Flexure stress is the sum of flexure stresses due to the above mentioned load conditions:

$$\sigma = \sigma 1 + \sigma 2 + \sigma 3$$

Floor beams were not tested to identify the timber, but historic sources commonly cite yellow pine in this application, so yellow pine was assumed to be the material used here.[41] The allowable stresses are as follows:[42]

Species and commercial grade	Bending *Fb* (psi)	Tension II grain, *Ft* (psi)	Shear II grain, *Fv* (psi)	Compression ⊥ grain, $F_{c\perp}$ (psi)	Compression II grain, F_{cII} (psi)	Modulus of elasticity, *E*
Spruce, pine, & fir Select structural 5"x 5" & larger	858	507	122.5	310.25	604.5	1066000

Note: As it is historic nineteenth century timber, this was considered to be the best quality from actual NDS design values.

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41 Snow, "Wooden Bridge Construction," 31-43.

42 American Forest & Paper Association, American Wood Council, "Design Values for Wood Construction," *Supplement National Design Specification*, 2001, Table 4D, 47.

Table 13. Bending and stress in floor beams

No.	Version description	*P*	*w*	*l*	*x*	*b*	*c*	*R*	*Mmax*
		kip	plf	ft	ft	ft	ft	kip	pft
1	Bending moment from self weight - outer suspension		31.25	21.00	7.00	19.00	1.00	0.33	1394.53
1'	Bending moment from self weight - inner suspension		31.25	21.00	7.00	17.00	2.00	0.33	1066.41
2	Bending moment from dead load on stringer - outer suspension	0.3		19.00	7.00			0.27	1620.00
2'	Bending moment from dead load on stringer - outer suspension	0.3		17.00	7.00			0.27	1350.00

w = 31.25 pfl (dead load)

width in	depth in	A in^2	W in^3	$\sigma 1$ psi	$\sigma 2$	Fb psi	$\sigma 3$ psi
9.75	15.75	153.56	403.10	41.51404		858	
9.75	15.75	153.56	403.10	31.74603		858	768.2599
9.75	15.75	153.56	403.10		48.22606	858	
9.75	15.75	153.56	403.10		40.18838	858	786.0656

$\sigma 1$ = stress from uniform load along line (self weight)
$\sigma 2$ = stress from dead load through stringer
Fb = allowable stress from bending
$\sigma 3$ = allowable stress from bending from live load (axle)

The maximum half-axle force for the inner and outer suspended floor beams are:

P_1 = 3.66 kip (NDS value)
P_2 = 4.38 kip (NDS value)

Working with Snow's 1200 psi allowable stress the values are:

P_1^s = 5.13 kip (Snow's value)
P_2^s = 6.12 kip (Snow's value)

Note: these are the limit values that one single beam can carry.

The first two members of the flexure stress are known, so the third component can be calculated when the sum itself is equal to the allowable stress. Two sets of calculations have been carried out, one using the NDS allowable stresses, and one, termed "Snow's value," using historic allowable stress values published by him.

The maximum half-axle force for the inner and outer suspended floor beams are:

P_1 = 3.66 kip (NDS value)
P_2 = 4.38 kip (NDS value)

P_1^s = 5.13 kip (Snow's value), full axle 11.25 kip
P_2^s = 6.12 kip (Snow's value)

Maximum Tension in Suspension Rods

Table 14 lists the maximum allowable tensions corresponding to a 20,000-psi allowable stress for the two rod sizes used in the bridge.

Table 14. Maximum tension in suspension rod

Name	**D** in	**A** in^2	Allowable stress psi	**Ft** kip
Suspension rod tension capacity	1.5	1.77	20000	35.325
Suspension rod tension capacity	1.25	1.23	20000	24.5313

P_{sr} = 24.53 kip, so even with a 50-percent reduction of working area, the allowable stress was a much larger value than the one given by flexure of the floor beams. (Snow used an allowable stress of 10,000 psi for wrought iron, reducing the capacity *Ft* to 12.26 kip.)

Maximum Compression Perpendicular to Grain

Table 15 shows the maximum compression perpendicular to the grain of the wood generated by the suspender rods through both circular and square washers.

Table 15. Maximum compression perpendicular to grain

Name	**D** in	**A** in^2	**Fcp** psi	**Fcp** psi	**Pcp** kip	**Pcp** kip	**Pcp^s** kip
Compression perpendicular on grain under circular washer	5.50	23.75	310.25	360.00	7.37	6.77	7.95
Compression perpendicular on grain under square washer	6.00	36.00	310.25	360.00	11.17	10.57	12.36

Pcp = 6.77 kip (NDS value) Pcp^s = 7.95 kip (Snow's value)

These analyses of substructure components and systems suggested that, for Cooper's E20 load capability (a maximum axle load of 20 kip), the floor system had to be stiff enough in the longitudinal direction to share loads between at least two adjacent beams.

Comparison of Three-Dimensional Finite Element Models

Three-dimensional models were constructed to allow a comparison with the two-dimensional model. The same finite element analysis software (SAP2000) was used for both models.

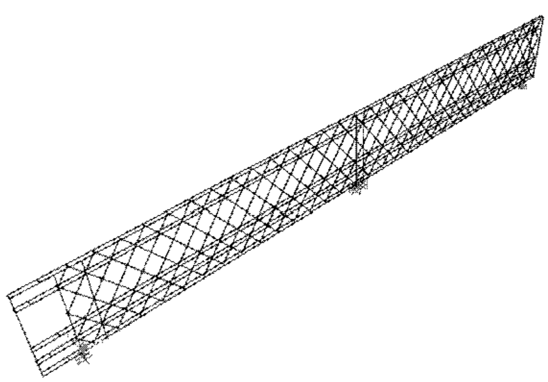


Figure 19. Axonometric view of the inner lattice truss, three-dimensional model

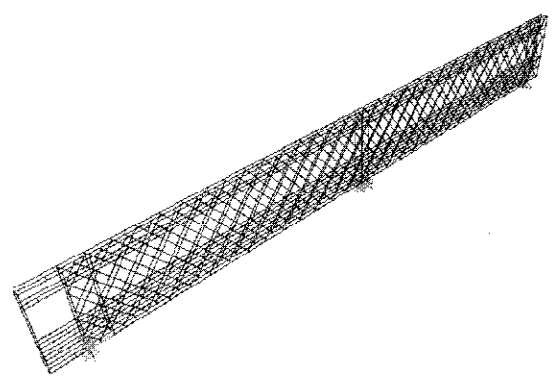


Figure 20. Axonometric view of the double lattice truss, three-dimensional model

Overall Stiffness of the Structure, Three- versus Two-dimensional Models

Referenced to the equivalent beam model, the overall behavior of the structure was similar for both models. The distribution of axial forces, shear forces and bending moments were similar as well, but the overall stiffness calculated differed remarkably. Table 16 shows deflections (inches at mid-span) calculated by the following models:

- 2D inner lattice truss
- 2D total (double truss)
- 3D inner lattice truss
- 3D integral – double lattice truss

Table 16. Deflections calculated by two- and three-dimensional models

No.	Load condition	2D Inner	2D Total	3D Inner	3D Integral
1	Dead load	-0.0727	-0.04835	-0.106515	-0.100171
2	Live load Cooper's E20	-0.20533	-0.10233	-0.325431	-0.26034
3	Reduced concentrated load at mid-span	-0.32219	-0.1831	-0.64902	-0.497939
4	Combination dead load + Cooper's E20	-0.27803	-0.15067	-0.431946	-0.360511

Using the results of the above table the following conclusions were suggested concerning the efficiency of doubling the truss:

> Under dead load condition, the double lattice webs deformed almost as much as the single ones. They were connected only through the middle planks of the chords and actually deformed almost independently.
>
> A two-dimensional model for a double lattice structure would significantly mislead interpretation of stiffness. As the data in Table 16 show, deflections calculated by the three-dimensional model are 2 to 2.5 times greater than the ones from a two-dimensional model, due to it being over-stiffened by the high number of infinite stiff joints that do not exist in the reality.

Reflecting on the equivalent beam analysis, which did not take into consideration any rotation or deformation that is possible (even in a 2D model with infinite stiff inner lattice joints) the equivalent beam deflection was 56 percent less than the two-dimensional model predicted. For a single Town lattice truss, the two-dimensional analysis calculated deflections 50 percent less than those generated by the three-dimensional model, so the actual stiffness was one-half of that suggested by the equivalent beam analysis. The differences were even larger (four times) for double lattice trusses

Characteristic Element Forces in the Three-dimensional Models

Table 17 synthesizes the member forces on a lower chord member, a compression lattice member, and a tension lattice member, all at the middle pier, comparing the single-truss, three-dimensional model (white background) to the double-truss, three-dimensional model (shaded background).

Table 17. Principal forces in typical elements

(a) Tension lattice member

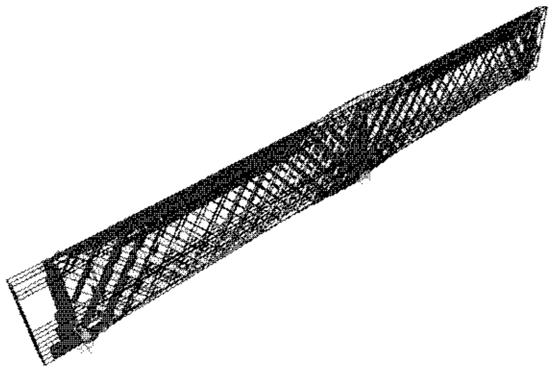
Frame	Load	P	P	V2	V2	V3	V3	T	T	M2	M2	M3	M3
		kip	kip	kip	kip	kip	kip	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft
906	Dead	2.573	2.18	0.002373	-0.006314	0.007135	-0.003121	0.0404	0.0236	0.0081	0.0023	0.0114	0.0347
906	E20	7.043	4.376	-0.077	-0.062	0.017	0.007062	0.0887	-0.045	0.0178	-0.082	0.157	0.0939
906	Concentrated	5.94	2.695	-0.004821	-0.012	0.035	-0.002634	0.1378	0.0256	0.015	-0.016	-0.0079	0.0015
906	Combination	9.616	6.556	-0.075	-0.068	0.024	0.003941	0.1291	-0.0214	0.0259	-0.08	0.1684	0.1287

(b) Compression lattice member

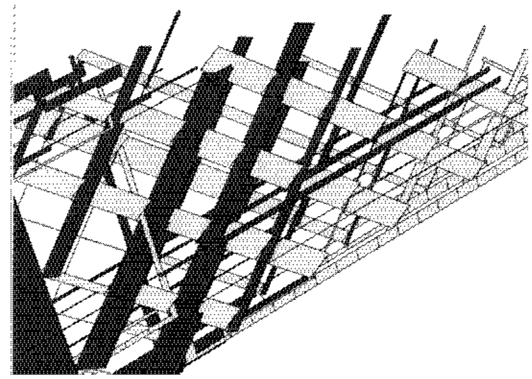
Frame	Load	P	P	V2	V2	V3	V3	T	T	M2	M2	M3	M3
		kip	kip	kip	kip	kip	kip	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft
1058	Dead	-7.04	-6.582	-0.135	-0.119	0.057	0.053	0.0572	0.0457	0.2234	0.199	-0.3403	-0.3085
1058	E20	-17.963	-10.874	-0.341	-0.21	0.151	0.106	0.1235	0.0423	0.5732	0.3444	-0.8994	-0.566
1058	Concentrated	-16.983	-9.101	-0.258	-0.127	0.144	0.072	0.1447	0.0702	0.5576	0.272	-0.7215	-0.3734
1058	Combination	-25.003	-17.457	-0.476	-0.329	0.208	0.16	0.1807	0.088	0.7966	0.5434	-1.2397	-0.8746

(c) Primary lower chord member

Frame	Load	P	P	V2	V2	V3	V3	T	T	M2	M2	M3	M3
		kip	kip	kip	kip	kip	kip	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft	kip-ft
1033	Dead	-3.204	-3.351	1.301	1.256	-0.124	-0.103	0.142	-0.0122	0.4705	0.307	-3.8986	-3.7492
1033	E20	-11.488	-7.186	4.613	4.043	-0.335	0.037	-0.538	-2.0919	1.2702	-0.957	-13.5046	-11.4431
1033	Concentrated	-12.651	-8.841	2.008	1.185	-0.333	-0.143	0.5079	-0.7606	1.41	0.445	-7.808	-4.7106
1033	Combination	-14.693	-10.537	5.914	5.299	-0.459	-0.066	-0.396	-2.1041	1.7406	-0.65	-17.4032	-15.1923



(a) General distribution



(b) Detailed distribution

Figure 21. Axial force distribution

These results suggested the following conclusions:

Axial forces were reduced 50 to 60 percent for live load in all analyzed members by introducing the second lattice. With dead load more equally and directly loading the members, the reduction in that case was limited to 5 - 18 percent.

Vertical shear forces (V2) were not shared much between lattice webs as axial force distribution. They were reduced only 10 percent for chord members (where they were considerable forces), but for lattice members shear is negligible.

Horizontal shear forces (V3) can be negligible in complete trusses, but they had measurable values for single-plank lattice trusses.

Though torsion became important when live loads were applied to floor beams hung from the inner lattice only, in reality there was already a load distribution on the floor beams that would reduce torsions significantly;

Bending around the weak axis became measurable in the three-dimensional models, but it was significantly reduced when the second lattice web was introduced;

Dead load resulted in axial force concentration in the middle chord members, as all floor beams were suspended from the middle chord planks.

CONCLUSIONS – STRENGTHS AND WEAKNESSES OF DOUBLE-WEB TOWN LATTICE TRUSS

Structural finite element analyses, using two- and three-dimensional models, revealed that the two trusses, though connected to each other, interacted less than expected, due to the limited rotational and shear stiffness of the treenail groups. The overall stiffness of the double-web Town lattice truss was only 15 to 20 percent greater than the single-web truss. In terms of mid-span deflection, the trusses acted almost independently under dead load, with stiffness only 6 percent for the double-web structure. The great stiffness of Town lattice trusses is achieved by the high number of chord-lattice and lattice-lattice joints that can transmit all of the characteristic member forces through treenail-group rotation and shear. Overall stiffness is as much due to the finite stiffness of the structural joints as to the displacement of main elements such as primary and secondary chords.

Equivalent girder theory could be as misleading, especially for short-span, extremely stiff structures like the Contoocook Bridge, as a two-dimensional model based on infinitely stiff joints. Though the two-dimensional models identified characteristic member forces well, they over-estimated overall stiffness.

Chords in Town lattice trusses consist of several planks butted together. These splices reduced the overall stiffness, but the results from the various techniques used varied widely, from about 7 – 8 percent in the two-dimensional model, to 25 percent in the three-dimensional model, and as much as 50 percent by the equivalent beam method.

The three-dimensional model, as well as experimental studies, revealed that the two trusses in the double-lattice truss were working almost independently under dead load alone. The transmission of dead loads was carried out both through the rotational, but primarily the shear, capacity of the joints, and the also have a similar, determinant role under live loads. Direct rotations and translations applied on inner chord members were not measurably transmitted to middle and outer chord members.

Double-web Town lattice trusses serving railroads were well designed from an engineering point of view. Their maximum live loads were transmitted through all structural members and sub-structures involved, with basically the same safety factors throughout.

Computer software and hardware facilities have developed rapidly. Good software and hardware resources are now available to help professionals analyze and understand these structures, although the definition of correct input data can still be challenging.

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