ENGINEERING STUDY

For the

PISCATAQUOG TRAILWAY PROJECT—PHASE IV
MANCHESTER, NH

Federal Project No. Part of 04-33te
NHDOT Project No. 14412

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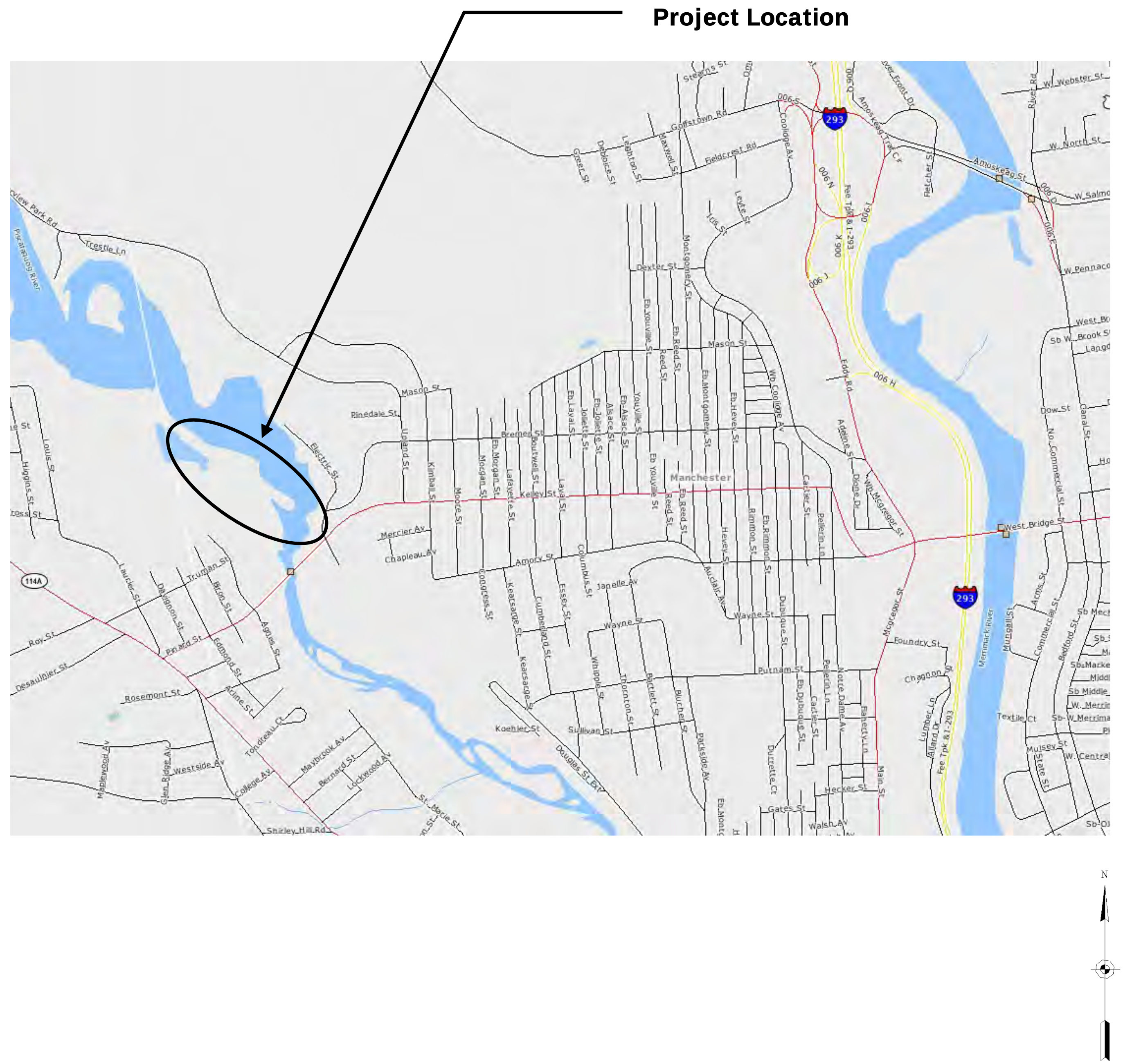
TABLE OF CONTENTS

APPENDICES

A. Existing Trestle Bridge Plan and Member Condition Notes
B. Environmental/Historical Resource Agency Coordination
C. Environmental Documentation
D. Project Photos
E. Plans of Proposed Improvements
F. Project Renderings
G. Engineer's Estimate of Probable Construction Costs
H. Dover Covered Bridge Plans
I. Glossary

LOCATION MAP



Piscataquog Trailway Project - Phase IV **Manchester, NH**	Hoyle, Tanner & Associates, Inc.	**File Name:** Piscat Eng Stdy.DOC
		DATE: 05/11 — **Page 1**

1 PROJECT DESCRIPTION

In accordance with the agreement between the City of Manchester and Hoyle, Tanner & Associates, Inc., this engineering study has been prepared to investigate the upgrade of an abandoned rail bed for pedestrian and bicycle use. It also includes rehabilitation and replacement options for a timber trestle bridge located within the project area. This report is administered, and the majority of funding provided through, the NHDOT Municipally Managed Transportation Enhancement Program.

The goals for this project include upgrade of the existing rail bed for pedestrian and bicycle use with some paved and some unpaved sections and timber guardrail or fencing where appropriate. Improvements to the trail intersection with Electric Street and improved connectivity with an adjoining trail for pedestrians are also desired. The goals for the Piscataquog River crossing include a safe, low maintenance structure that can accommodate pedestrians and light maintenance vehicles. It is also desired to reduce waterway restrictions at the bridge as there is a history of blockages at this location.

The Engineering Study report was compiled from existing City and New Hampshire Division of Historic Resources (NHDHR) information and data collected and photographs taken during site visits. The intent of this report is to evaluate existing conditions and project goals, and to recommend a solution which best accomplishes the project goals. The project limits considered for this report are approximately 1,800'

To aid the readers of this study and improve clarity, north has been assumed to be upstream from the bridge. This results in a trail orientation of east to west towards Goffstown. It is also important to note that several terms are used interchangeably throughout the study. The existing railroad bed is referred to as a trail, trailbed, shared use path and rail bed depending upon the context of the section.

2 EXISTING CONDITIONS

South Trail Section at Trestle Bridge

2.1 Trailbed

Portions of the existing trailbed for this project are currently owned by the State of New Hampshire, Town of Goffstown and the Piscataquog River Apartment Limited Partnership (PRALP). A 30' wide trail easement to benefit "Rails to Trails" is shown on the recorded plan for Goffstown and Piscataquog portions however, there is no record of it being completed. The Town of Goffstown owns from the Town line easterly for approximately 450'. The PRALP property then extends for approximately 1025' to the existing bridge and the State of NH owns the bridge and property to Electric Street, which is approximately 330' along the railroad.

The railroad bed and line was constructed and completed in the project area in 1850 and was last operated as an active rail line by the Boston and Maine Railroad. The rail line was officially abandoned in 1981, after over 130 years of service. The railroad carried an enormous amount of freight over these years therefore the potential for chemicals and/or contaminants in the existing fill materials is unknown. Disturbance on the trailbed will be minimized due to this potential (see Section 4). Based on the information available on the Natural Resources Conservation Service (NRCS) web site, the existing fills under the railbed are granular well drained soils mapped as Hinckley and Windsor loamy sand, which are considered to be a Hydrological Soil Group (HSG) A soil types.

The existing trailbed is mostly a sandy loam surface which is undeveloped, with some brush and small trees encroaching on the top section of the bed and steep vegetated sideslopes. The sideslopes range from relatively flat to 1:1 slopes with signs of significant previous and ongoing erosion in the steeper areas of the eastern side of the bridge adjacent to Electric Street. The existing erosion appears to be due to the lack of vegetation, sandy soils and foot traffic up and down the steep slope to and from the parking lot to the north.

Existing utilities are present in the railbed and include overhead power lines on the east side of the bridge and a sewer line from Goffstown on the west side of the bridge with a municipal metering station on Piscataquog River Apartment Limited Partnership property.

A gravel foot path is located approximately 400' west of the trestle bridge. It leads in a southerly direction and will require a proper trail intersection to be designed.

2.2 Bridge No. 1.89 (404) [Trestle Bridge]

The existing timber trestle bridge (trestle) was reportedly built in 1941 for the Boston and Maine Railroad according to a single record plan of the trestle (see Appendix A). Prior to the trestle, there have been two covered bridges built at the site (ref. www.lostbridges.org). The first was a Town Lattice with arches built in 1875 that was in place until 1915 when it was replaced with a double Town Lattice. The double Town Lattice Bridge was lost to fire on September 1, 1941. Notches for the arch bearings of the 1875 bridge are still prominent on the stone abutments.

1875 Bridge Arch Bearings

On October 28, 2010 an inspection team from Hoyle, Tanner visited the trestle. Inspection access was provided by Wright Construction Company, Inc. using raft-mounted staging. The purpose of this inspection was to obtain field measurements and photography and to determine the overall condition of visible components of the trestle. Our inspection was limited to the visible portions of the trestle above the water level which was approximately 16' below the east abutment beam seat elevation. A summary plan of the bridge is included in Appendix A including nomenclature used in this study, typical member sizes, and field notes. All members discussed below are pressure treated wood, except where otherwise noted.

Trestle Superstructure

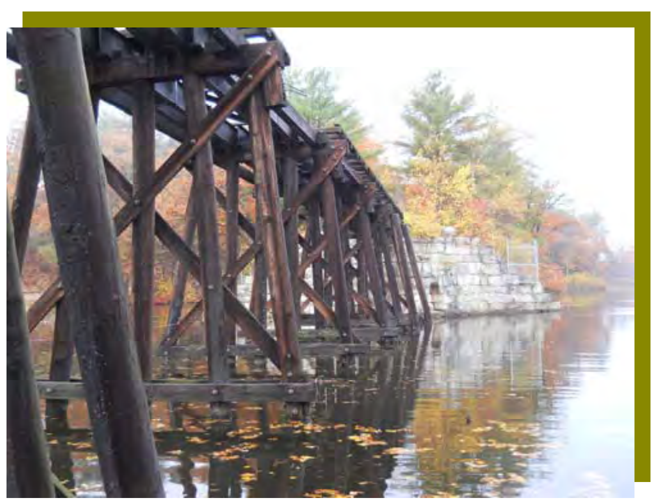
Typical Trestle Superstructure

The superstructure consists of timber cross ties bearing on 4 – 20” deep steel beams and 7¾”x11½” timber beams that span longitudinally from bent to bent. The steel beams are braced extensively with steel angles that span both transversely from beam to beam and diagonally between the top flange of the center two steel beams. At each end of the cross ties, there is a timber curb. No railroad track is currently in place on the bridge.

The condition of the trestle superstructure varies from poor to good. The cross ties are in fair to poor condition with many missing near the center spans of the trestle and rot throughout many ties. The longitudinal timber beams that flank the steel beams are in fair to good condition with the ends of several beams exhibiting twisting and rot at the ends. The steel beams are generally in good condition with little rusting or deterioration noted. The steel beams have an unknown coating that is largely intact on all surfaces except for the top flange.

Bents

The trestle bents each consist of five main piles, two of which are battered transversely to the bridge, and one upstream pile that acts as a bumper. The piles vary in diameter with an average diameter of 13” at near the waterline. The piles are capped with a 11-¾”x13-½” pile cap that appears to be connected to the pile with a ¾” diameter pin, centered on the pile. The piles and cap are connected with 2-¾”x9-½” diagonal bracing that is generally through-bolted to each pile and cap. No underwater inspections of the bents were completed as part of this study.



Timber Bents Looking North

10" of Rot in 13-½" Deep Cap Beam at Bent 5

The condition of bents 1-5 is generally fair to good with some components in poor condition, while bent 6 (westernmost bent) is in poor condition with extensive rotting of the piles. While there are portions of each bent in poor condition, the bents are in surprisingly good condition after 70 years of service. Portions of the diagonal brace just below the waterline is broken or split at all six bents and is likely due to impact damage from ice or debris. The upstream bumper pile is missing above the waterline at bents 2-4. The cap beams for bents 3-6 all exhibit extensive rot starting from the top surface. In some locations this rot extends nearly through the entire depth of the cap beam.

Substructure

The substructure consists of large cut stones generally in a running bond pattern that serves as the trestle abutments and wingwalls. Portions of the substructure include chinking and pointing, however it is not consistent throughout. Chain link fence was added by the City of Manchester at each abutment in 2002. The fence posts are either ground mounted behind the abutment or side mounted to the face of the wingwalls.

Face of West Abutment and Southwest Wingwall

The 16' of abutments and wingwalls visible above the waterline are in good condition with no indications of settlement, bulging or movement noted. The pointing and chinking is missing in some locations and vegetation is growing in many of the stone joints. A small tree has begun to grow at the northeast wingwall.

The remaining east approach to the trestle largely consists of sand which is extensively eroded. The current approach elevation at the east abutment is approximately 5' lower than the top of the cross ties on the trestle. The erosion is especially noticeable at the southeast wingwall of the bridge.

3 DESIGN CRITERIA

3.1 Trailbed

DESIGN GUIDELINES:	1) 1999 AASHTO Guide for the Development of Bicycle Facilities. 2) New Hampshire Statewide Bicycle and Pedestrian Plan. 3) 2004 AASHTO Guide for the Planning, Design and Operation of Pedestrian Facilities.
DESIGN SPECIFICATIONS:	1) NHDOT Standard Specifications for Road and Bridge Construction, 2010.
REGULATIONS:	1) Americans with Disabilities Act Accessibility Guidelines (ADAAG). 2) Uniform Federal Accessibility Standards.

3.2 Bridge No. 1.89 (404) [Trestle Bridge]

DESIGN LOADING:	H10 (10 tons)
DESIGN SPECIFICATIONS: (Rehabilitation)	1) AASHTO Standard Specifications for Highway Bridges 17th Edition, 2002. 2) NHDOT Standard Specifications for Road and Bridge Construction, 2010.
DESIGN GUIDELINES: (Rehabilitation)	1) Guidelines for Historic Bridge Rehabilitation and Replacement, November, 2008.
DESIGN SPECIFICATIONS: (New Bridge)	1) AASHTO LRFD Guide Specifications 5th Edition with 2010 Interims. 2) NHDOT Standard Specifications for Road and Bridge Construction, 2010.
DESIGN GUIDELINES: (New Bridge)	1) AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges, 2nd Edition.
DESIGN MANUALS: (Rehabilitation & New Bridge)	1) NHDOT Bridge Design Manual, 2000

3.3 Accessibility

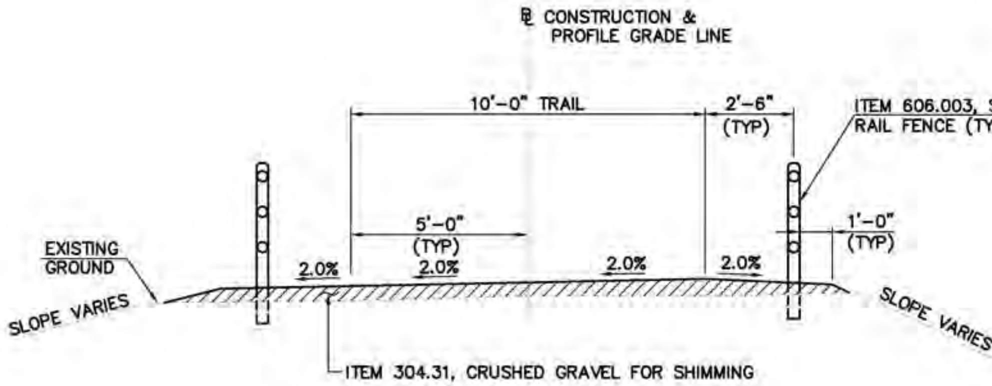
Due to the inclusion of federal funding for the proposed project, the finished project must comply with Federal Highway Administration (FHWA) accessibility requirements. The FHWA reviews projects for compliance with the Americans with Disabilities Act of 1990 (ADA) and Section 504 of the Rehabilitation Act of 1973. The FHWA, in February 2000, provided technical guidance for disability compliance for transportation projects.

A September 12, 2006 memo from Frederick D. Isler, Associate Administrator for Civil Rights and King W. Gee, Associate Administrator for Infrastructure further clarified the FHWA's requirements for accessibility on transportation project where the FHWA provides funding. The memo states that "shared use paths and pedestrian trails that function as trails should meet the accessibility guidelines proposed in the Access Board's *Regulatory Negotiation Committee on Accessibility for Outdoor Developed Areas Final Report* found at www.access-board.gov/outdoor/outdoor-rec-rpt.htm". This document includes the following requirements:

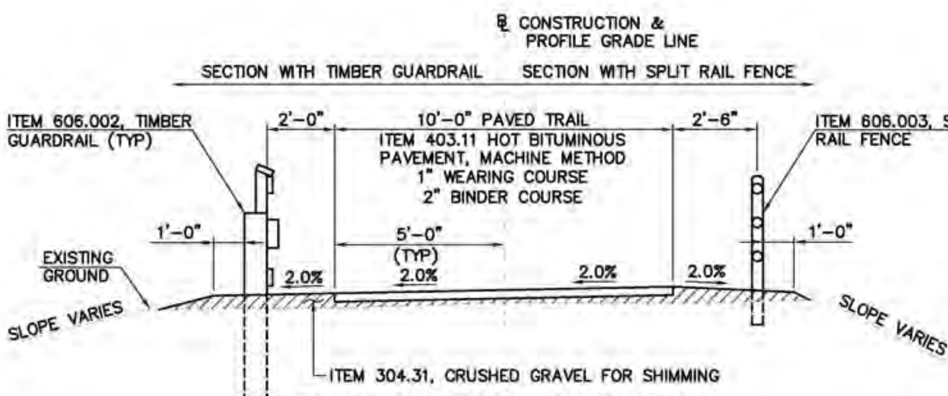
- Trail surface to be firm and stable.
- Clear width of 10' minimum.
- Openings in trail surfaces to be of a size that does not permit the passage of a ½" diameter sphere.
- Protruding objects comply with ADAAG 4.4.1.
- 80 inches minimum clear head room.
- Maximum trail cross slope of 1:20.
- Handrails are not required on trails.
- 4 provisions for slope
    - 1:20 or less for any distance.
    - 1:12 maximum for 200' with resting intervals of 200' or less.
    - 1:10 maximum for 30' with resting intervals no greater than 30'.
    - 1:8 maximum for 10' with resting intervals no greater than 30'.

4 PROPOSED TRAILBED IMPROVEMENTS

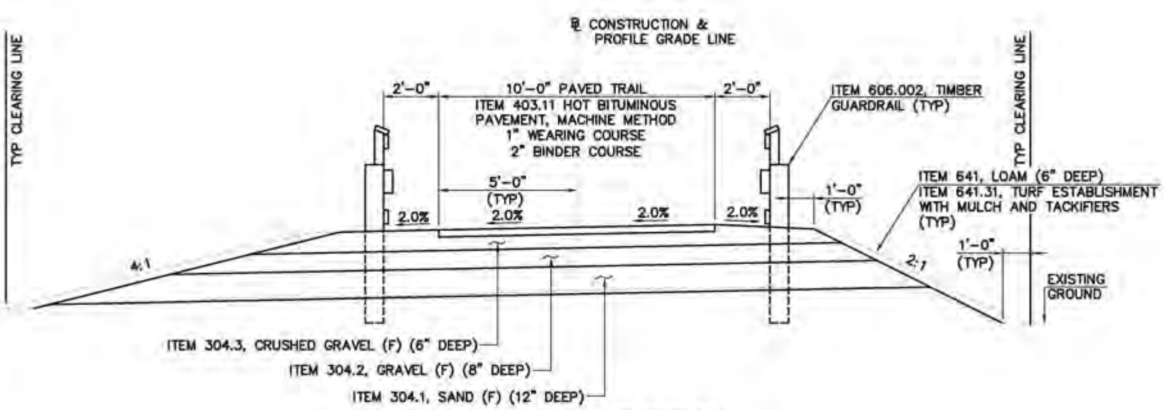
The proposed trail will be constructed over the existing railroad bed which will be modified and improved for pedestrians, bicyclists and occasional maintenance vehicular traffic. All unauthorized vehicle traffic will be deterred at Electric Street through the use of a removable and lockable bollard. The western side of the trail will be more of a challenge to deter vehicles and all terrain vehicles from access. The existing metering station is accessed through an adjacent apartment complex by a paved road that terminates at the trail and will ultimately lead vehicles to the trail itself. Once trail construction is complete, limited pedestrian access will be allowed at this location, but will not be encouraged. No parking facilities or improvements are anticipated in this area as part of this project. Appropriate signage in this location to deter motorized vehicle usage on the trail and periodic enforcement will likely be required, at least during and upon completion of the construction.
The trail will be constructed of a crushed gravel surface from the Goffstown town line to approximately station 6+00 where the finish surface will transition to pavement and continue as pavement to the bridge deck. The cross slope of the trail will be 2% in a northerly direction towards the river from the trail beginning at the town line to the bridge deck. The existing side slopes will remain and vegetation will be removed as necessary to provide the required clearances. A split 3-rail fence will be installed on both sides of the trail adjacent to the proposed information kiosk from station 11+00 to 13+50. A timber guardrail system will be installed on both sides of the trail from station 13+50 to the bridge deck. The proposed kiosk that houses general information and maps will be located at the newly reconstructed intersection of the gravel foot path and trail.



TYPICAL TRAIL SECTION
STA. 0+00 – STA. 6+00



TYPICAL TRAIL SECTION
STA. 6+00 – STA. 14+50
STA. 17+25 – STA. 18+09



TYPICAL TRAIL SECTION
STA. 14+50.00 TO STA. 14+74.50
STA. 16+44.75 TO STA. 17+25.00

Proposed Timber Guardrail

Both approaches to the bridge deck on the west and east sides need to be filled with a combination of compacted sand and crushed gravel meeting NHDOT specifications to meet the required finish elevation of the bridge. The approaches will have timber guardrail installed to provide a barrier to the adjacent steep side slopes.

Proposed 3-Rail Fence

The east approach to the bridge deck will be a paved finish surface to Electric Street. The 2% cross slope of the trail will be in a southerly direction towards the river from the bridge deck to Electric Street. A Mechanically Stabilized Earth block wall on the north side of the trail of approximately 5 feet in height is required due to elevation of the new bridge deck, width of the proposed trail and the existing eroded steep slopes. Steeper slopes in the area of the retaining wall and existing bridge abutments will require the installation of timber guardrail to be set a minimum of 2' off the edge of pavement and the side slopes will be treated with 6" loam and seed with a coconut fiber reinforced erosion fabric to ensure a vegetated stable slope is established.

The painted brick delineation crossing at Electric Street will have double stainless steel detectable warning devices installed for pedestrians as an added precaution to the proposed signage.

The total estimated cost of the trail work in 2012 dollars is approximately $150,010 (See Appendix G for Engineer's Estimate of Probable Construction Costs). This estimate of cost remains the same regardless of the bridge option selected.

5 ENVIRONMENTAL REVIEW AND DOCUMENTATION

The proposed project will require environmental review and compliance with applicable state and federal regulations. This section summarizes the environmental related effort completed to date as well as the anticipated permits/documentation that will be required for the project.

Hoyle, Tanner & Associates, Inc.

Due to the inclusion of Federal funds, the project must comply with the National Environmental Policy Act of 1970 (NEPA). NEPA requires that federal agencies consider the environmental impacts to proposed actions and reasonable alternatives to those actions. The project was presented to the NHDOT Monthly Natural Resource Agency Coordination Meeting on February 16, 2011 (see Appendix B for meeting minutes) to seek input from environmental resource agencies. Based on the feedback received at this meeting, it appears that the project will qualify for a Categorical Exclusion (CE) under NEPA. The CE document includes a review of key aspects of the project and if the project qualifies a detailed environmental analysis is not required and the project may proceed. The CE document is typically prepared during the preliminary plan phase of the project.

The project will have impacts within New Hampshire Department of Environmental Services (NHDES) jurisdictional wetlands and will therefore require two NHDES permits. A Standard Dredge and Fill permit will be required for wetland impacts. In addition, since the Piscataquog River is listed as a protected river under the NHDES Comprehensive Shoreland Protection Act (CSPA), a permit application must be submitted for improvements within 250' of the reference line, which is the visible mean high water line of the river. A presentation to the Piscataquog River Local Advisory Committee (PRLAC) of the proposed improvements will also be required. The primary goals of the CSPA and PRLAC are to protect and improve water quality of the river and to educate people about the uses and precautions needed adjacent to the river.

A review of the project for rare species and exemplary natural communities was conducted through the New Hampshire Natural Heritage Bureau (NHB) (see Appendix C). According to the NHB, "The species considered include those listed as Threatened or Endangered by either the state of New Hampshire or the federal government." The results of the review determined "…that, although there was a NHB record (e.g., rare wildlife, plant and/or natural community) present in the vicinity, we do not expect that it will be impacted by the proposed project."

6 HISTORICAL CONSIDERATIONS

In accordance with RSA 227-C:9 "Directive for Cooperation in Protection of Historical Resources", Hoyle, Tanner presented the project at the bi-monthly NHDOT Cultural Resources meeting on January 6, 2011. The meeting focused on Alternatives 1, 2 and 3 described in Sections 9.1 through 9.3 of this study as Alternative 4 was not being studied at that time. The consensus of the committee was that no additional documentation would be required for Alternative 1. If Alternative 2 or 3 were selected, and Individual Inventory Form would be required and there would be the potential for mitigation for Alternative 3.

7 UTILITIES

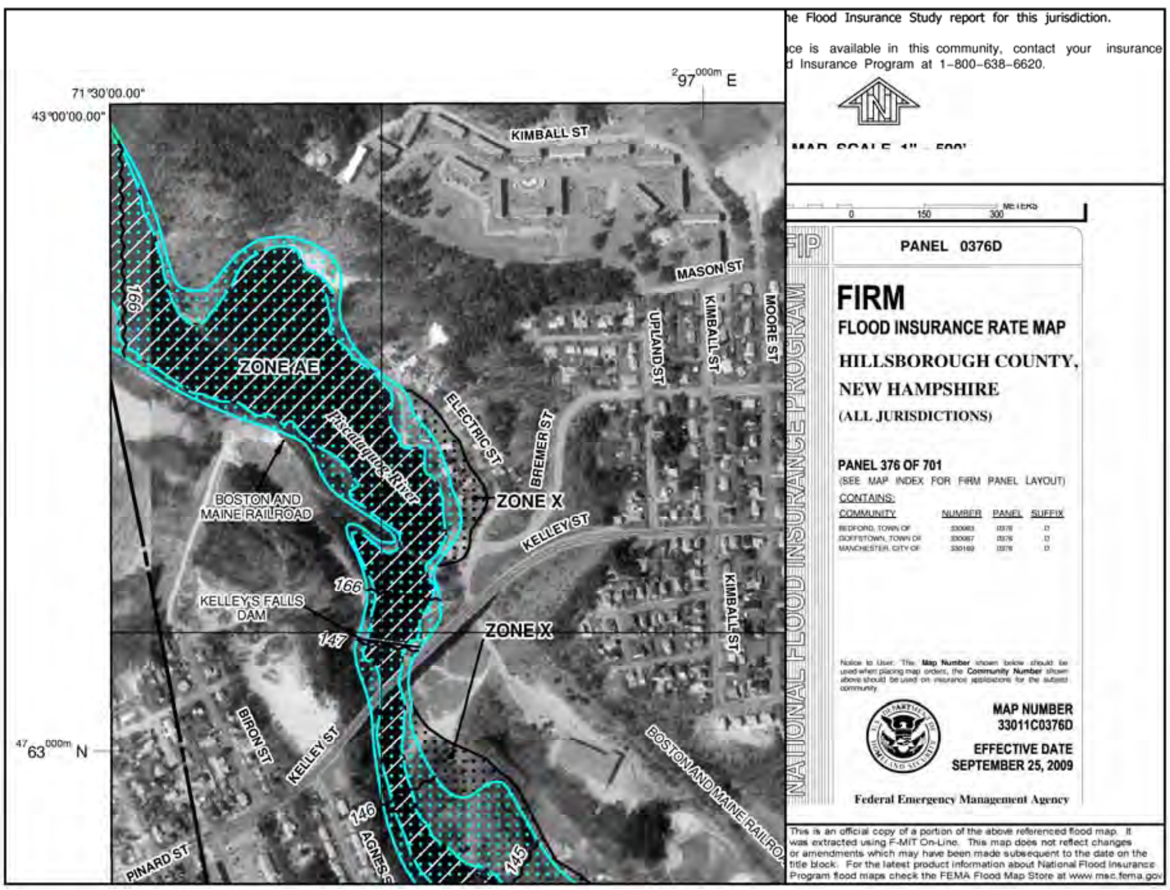
The only known utilities within the project limits are overhead utility lines leading to the dam headworks building at the east end of the project and water and sewer lines at the western 550' of the project. It is not anticipated that these utilities will be impacted as part of the project.

8 HYDRAULICS

The hydraulic characteristics of the Piscataquog River at the existing trestle crossing were reviewed with consideration to NHDOT bridge design requirements as well as the May 2009 New Hampshire Stream Crossing Guidelines (Stream Crossing Guidelines). Design flood elevations were determined utilizing FEMA Flood Insurance Rate Maps (FIRM).

The NHDOT requires the 50-year design flood flow (Q_{50}) and elevation to be determined for bridge projects while the Stream Crossing Guidelines require that the 100-year flood be ‘accommodated’. The Q_{50} design flood event has a 2% chance of being met or exceeded each year while the Q_{100} design flood has a 1% chance. The Stream Crossing Guidelines include further requirements for minimum bridge openings and bridge types and do not differentiate between bridge replacement and rehabilitation projects.

The FEMA FIRM provides a 100-year flood elevation of 166 upstream and downstream of the bridge, while the stone abutment bearing elevation is 170.43 and the west approach trail elevation is 173.16. Both of these elevations are over four feet above the 100-year flood elevation, therefore the existing trestle can accommodate this flood event. The FEMA FIRM also indicates that shoreland upstream of the trestle is flooded during the 100-year flood event, which is consistent with the known history of flooding in the area.



Per the Stream Crossing Guidelines, the width of stream crossing structures should provide for the adequate passage of water, sediment, and organic matter at all flow levels. In an attempt to standardize adequate stream crossings, an opening of 1.2 times the bankfull width is provided as a minimum requirement. In reviewing the Piscataquog River upstream of the bridge it is clear that existing trestle crossing does not meet this requirement, therefore an alternative design can be proposed if a specific rule stated in the guidelines is not practicable (Env-Wt 904.09). Practical is defined by Env-Wt 101.69 as "available and capable of being done after taking into consideration cost, existing technology, and logistics in light of overall project purposes." A written request sealed by a P.E. or environmental scientist must be submitted to NHDES that explains how the proposed alternative demonstrates that adhering strictly to the stream crossing guidelines is not practicable in this case. This request must also state how the proposed alternative meets specific design criteria specified in Env-Wt 904.05 to the maximum extent practicable and also satisfies all general design criteria specified in Env-Wt 904.01.

9 BRIDGE REPLACEMENT/REHABILITATION ALTERNATIVES

9.1 Alternative No. 1 – Rehabilitate Existing Trestle

This alternative includes largely retaining the timber bents and steel superstructure of the existing trestle. The remaining rail ties will be removed and a new concrete filled metal pan deck installed over the steel beams. A pedestrian and bicycle appropriate steel rail will be added and the abutment backwall modified to accommodate this new use.

This structure alternative has the following characteristics (see Appendix E, Figure No. 9):

- Modification of the existing backwalls for rehabilitated structure.
- Repointing of portions of the existing abutments.
- Replacement of bent No. 6 in its entirety.
- Select minor repairs to bent members.
- Replacement of damaged or missing bent lateral bracing.
- Replacement of cap beams on bents 3, 4 and 5.
- Removal of rail ties and installation of a concrete filled metal pan deck.
- Installation of a new steel railing.

The estimated bridge construction cost of Alternative No. 1 in 2012 dollars is approximately $307,785 (see Appendix G for the Engineer's Estimate of Probable Construction Costs). Adding the estimated trail construction cost of $150,010 and 15% for contingencies gives a total estimated project construction cost of $526,700.

9.2 Alternative No. 2 – Modify Existing Trestle

During moderate to high flood events in the past, debris has built up at the trestle crossing which reduces the available opening and increases the backwater upstream of the bridge. Concerns regarding the flooding implications have been expressed by the Town of Goffstown. City of Manchester forces have removed the debris in the past at considerable cost due to the location of the trestle and danger associated with removing debris during high flows. Alternative No. 2 includes a rehabilitation of the existing bridge and removal of two bents to improve flow through the bridge. The alternate also includes replacement of the steel beams in the two spans where bents are removed.

As part of the evaluation of Alternate No. 2, a review of ice loads on the modified trestle was conducted. In October of 2002, the U.S. Army Corps of Engineers published a manual titled "Engineering and Design Ice Engineering". This document discusses the characteristics and effects of ice loading, as well as standard methods for the calculation of ice forces on structures. Pertinent parts of the manual are summarized in this section of the study as they relate to Alternate No. 2.

Any structure where ice is a potential hazard should consider and be designed for forces generated by ice moving against it. Ice moves due to the influence of shear stresses that primarily result from interactions with the wind and water. A moving sheet or ice floe will transmit these forces to a structure in its path. The magnitude of the ice force that is transferred to the structure when a collision occurs will be controlled by the force required to cause failure in the ice. Ice can fail by crushing, splitting, bending, buckling, or a combination of these modes. For any given collision, the force that is transferred to the structure is limited by the lowest estimated force required to fail the ice in any of the possible failure modes. Ice crushing is one of the most common modes of failure and is assumed to be the primary mode of failure for ice that comes into contact with the trestle.

When ice comes into contact with a pier on a river, it imparts a horizontal force. The magnitude of this force depends on the initial mass and velocity of the ice. These two characteristics determine how much energy the ice has before impact with a structure and how much ice the structure will need to crush in order to resist this energy. During an ice collision with a structure, the ice can either come to rest or deflect away from the structure and continue downstream. Ice will come to rest during a head-on collision and ice will deflect off of the structure during an eccentric impact.

In a head-on collision, the structure will completely stop the momentum of the ice. This is done by crushing the ice at the structure-ice interface. The Corps manual provides an equation that shows the relationship between the volume of crushed ice required to stop a solid piece of ice during a collision, the mass of the ice, and the velocity of the ice (Eq. 6-22):

$$\frac{Mv^2}{2} = p_e V$$

In the above equation, M is the mass of the ice, v is the velocity of the ice, V is the volume of the ice crushed (depth of the ice multiplied by the area of crushed ice), and p_e is the effective pressure or strength of the ice.

The term on the left side of the equation represents the kinetic energy of the ice before impact with a structure and the term on the right side of the equation represents the energy dissipated when the ice is crushed during a collision. Therefore, for a given effective pressure, ice will come to rest once enough of the ice volume has been crushed to equal the initial kinetic energy of the ice. The more massive an ice sheet or floe is and/or the faster it is moving, the more ice will need to be crushed by the structure during a collision bring it to a stop.

An eccentric impact will transfer some horizontal force to the structure and rotate the ice such that it retains part of its initial kinetic energy and continues downstream. The structure, in this case, would not need to bring the ice to a complete stop. The trestle bents are spaced approximately 22 feet apart. As a result, any dynamic ice floe that is less than 22 feet wide will likely only impact one pier in an eccentric impact and continue moving downstream. By reducing the number of piers, the horizontal ice force that results from this type of ice dynamic impact will not be increased.

Ice sheets that are wider than 22 feet, however, will currently come into contact with more than one pier during a structure-ice interaction. In this type of interaction, the contact area would be less with a reduced number of piers. As a result, the reduced number of piers will each have to crush more of the ice in order to bring the entire ice sheet to rest.

This is a concern for single, static sheets of ice that are frozen onto and stuck upstream of the structure. The momentum of the ice due to the shear forces imparted on it by both wind and water will continuously be transferred to and resisted by the reduced number of piers. Therefore, a rehabilitation option that includes bent removal would likely increase the static ice load on the remaining piers.

The increased ice force can be estimated, however, it requires a more advanced analysis in which the geometry, velocity, and physical properties of the ice would have to be approximated. It was determined that removing two bents on the Picataquog River Bridge would increase the static ice load on the remaining piers.

Based upon the anticipated increased ice loads on the remaining bents, it does not appear that this Alternative is viable without substantial modification to the remaining bents. The alternative was therefore not considered further as part of this study.

9.3 Alternative No. 3 – New Steel Truss Bridge

This alternative includes complete removal of the timber trestle (timber bents, steel beams, etc.) and replacement with a new, single span steel truss bridge. The truss bridge will be constructed from weathering steel tube members with a pressure treated wood deck. The railing will be 54 inches high with vertical pickets at four inches on center. This option has the advantage of removing six bents that can act as obstructions during flood events and provides a low maintenance, long lasting bridge structure.

This structure alternative has the following characteristics (see Appendix E, Figure No. 10):

- Modification of the existing backwalls for the new structure.
- Repointing of portions of the existing abutments.
- Removal of the existing trestle in its entirety.
- Installation of a new, single span steel metal truss pedestrian bridge.

The estimated bridge construction cost of Alternative No. 3 in 2012 dollars is approximately $403,700 (see Appendix G for the Engineer's Estimate of Probable Construction Costs). Adding the estimated trail construction cost of $150,010 and 15% for contingencies gives a total estimated project construction cost of $637,100.

9.4 Alternative No. 4 – Dover Covered Bridge

9.4.1 Background

During development of this Engineering Study it was noted that the City of Dover, NH has an approximately 153 foot long covered bridge available for re-use. On February 22, 2011 representatives of the City of Manchester and Hoyle, Tanner visited the site where the bridge is currently stored. Based upon this site visit, it was decided that the option of using this bridge at the Piscataquog River Crossing should be studied.

Dover Covered Bridge Elevation

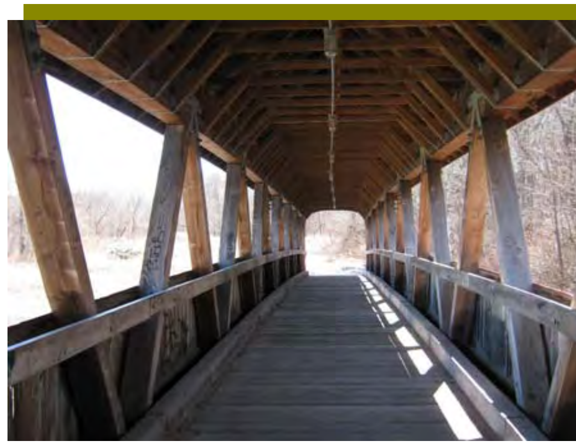
The Dover Covered Bridge (DCB or bridge) was constructed in 1996 and remained in service over the Cocheco River until 2010. The vehicular bridge adjacent to the DCB was replaced with the Tommy and Mary Makem Bridge, which accommodates both vehicular and pedestrian traffic making the DCB no longer needed in this location. The DCB is currently supported on concrete blocks in a storage area within sight of its previous location.

The DCB spans 151 feet 10 inches between bearings, 152 feet 11 inches end to end of bottom chord and is 10 feet 6 inches wide (8 foot 6 inches between inside railings). The bridge trusses consist of Douglas fir glulam and sawn members that are connected with bolted steel plates. The roof consists of pre-fabricated wood trusses that utilize metal plate connectors at the joints. There is extensive wood bracing in the upper portions of the bridge that is connected with metal connectors and steel 'x' bracing in the lower portions. The floor system consists of wood floor beams at each truss panel point with wood stringers spanning between.

The clear span between abutments at the Piscataquog River Trestle location is 151 feet 6 inches and the distance between backwalls is 169 feet 10 inches. If used in its existing configuration at the Piscataquog River crossing, the DCB would only have 8-½ inches of lower chord bearing at each end and the centerline of bearing would only be 2 inches from the face of the abutment, which is inadequate. We evaluated an option to lengthen the bridge by removing the end two lower chord sections from the existing splice to the bearing end, and replacing them with a longer section. The end two diagonal web members would also be replaced as the end diagonal would be shifted away from the center of the span, thereby increasing the center to center of bearing distance of the truss. This change does not greatly alter the truss forces and allows for the bridge to be supported properly at the abutments. As discussed in Section 9.4.3, the bridge was structurally evaluated for this new, longer configuration. We also evaluated a separate option where an additional section was added at each interior splice thus lengthening the bridge; however this option would increase the loadings in the truss web members beyond their design capacity which would result in increased member replacement and cost.

9.4.2 Bridge Condition

On April 7, 2011 representatives of Hoyle, Tanner performed a site visit to take field measurements and photographs and review the condition of the DCB. The field measurements were compared to plans and shop drawings obtained from the original designer HEB. The field measurements were found to be in good agreement with the available drawings of the bridge with only minor differences noted. A copy of the HEB plans which include member sizes have been included as Appendix H, therefore this information is generally not repeated below.



Inside of Dover Covered Bridge

The DCB is generally in good condition with some small areas in poor condition. The lower portions of the bridge have retained varying amounts of sand on top of the lower chord and decking. Especially noticeable inside the bridge is graffiti on the siding and truss members. The graffiti is extensive and extends throughout the length of the bridge. The following observations were also made regarding the condition of the bridge:

- The roof consists of wood shingles on plywood. The shingles are missing in several locations.
- The roof eave trim is painted white with the paint missing or peeling throughout the bridge.
- The bridge contains interior lighting utilizing simple fixtures and also contains small decorative lights along the eave line. The lights were not tested as part of our evaluation.
- The roof framing is in good condition with no distortion of damage noted.
- The bridge siding is ship lapped and in good condition with some boards missing at mid-span and at the truss bearings. The siding is painted red on the outside face with the paint fading or missing throughout the bridge.

Typical Stringer Connection

- The bridge trusses are in good condition with no member damage noted. The bridge trusses do exhibit some lateral sweep.
- The bridge deck is in good to fair condition with sand in the deck joints and wear and abrasion on the top surface.
- The floor beams and stringers are in good condition with water staining noted on the members. The stringer connections consist of pre-fabricated galvanized steel hangers nailed to the floor beams. These connections are

in fair to poor condition with localized rusting.
- The lower lateral 'x' bracing consist of steel rods that are in good condition. The rods do however make a banging noise as one travels across the bridge.

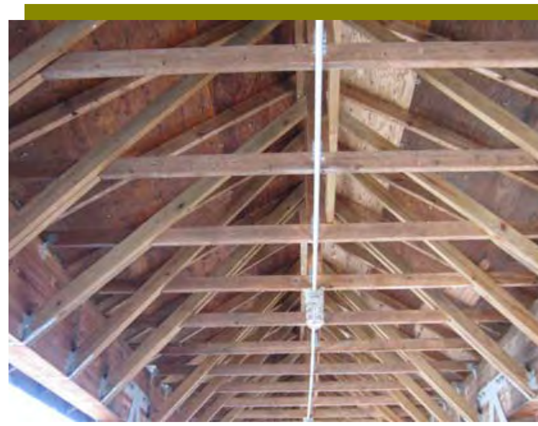
9.4.3 Analysis

The DCB was analyzed for dead, live and snow loads to determine if the lengthened bridge would be structurally adequate for a Piscataquog River Crossing. A discussion of this analysis follows.

9.4.3.1 Roof Framing

The existing roof framing consists wood shingles on ⅝" thick plywood supported by 1-½" by 3-½" rafters spaced at 24 inches on center. A fabrication tag was noted on several rafters which indicated that the trusses were fabricated using No. 2 spruce-pine-fir by Wood Structures, Inc. This species and grade were used in the analysis of the rafters.

The roof rafters were analyzed for dead load and snow load per the 2009 International Building Code (IBC) and referenced code documents. The snow load was determined using the US Army Corps of Engineers "Ground Snow Loads for New Hampshire" (ERDC/CRREL TR-02-06) and modified to a roof applied load following ASCE 7-10 (Minimum Design Loads of Buildings and Other Structures). The elevation corrected ground snow load for Manchester was 63 pounds per square foot (PSF) with a roof applied snow load of 34.6 PSF allowed by the IBC. This snow load was increased to 42 PSF in accordance with the City of Manchester Building Code Section 101.2.



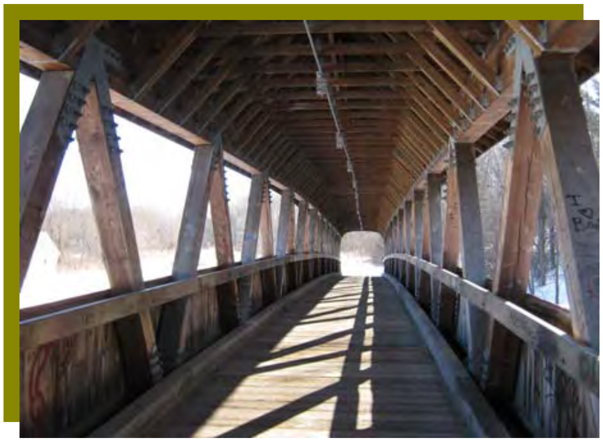
Roof Framing

The roof rafters were found to be adequate for dead and roof applied snow loads.

9.4.3.2 Trusses

The trusses were analyzed for dead, live and snow load with a bottom chord lengthened by 2 feet 3-½ inches on each end. This change moves the centerline of bearing to 1 foot 6 inches from the face of the abutment and provides 2 feet 9 inches of total chord bearing beyond the face of the abutments. This change will reduce the portal overhang at each end of the bridge with 1 foot of overhang maintained at each end. The bridge will bear on a new, 2 foot thick bearing pad to be placed on top of the existing abutments.

The AASHTO Guide Specifications for the Design of Pedestrian Bridges, National Design Specifications for Wood Construction, 2005 Edition (NDS) and ASCE 7-10 were used in our analysis for dead and live loads as well as member capacities. Snow loads were determined as discussed in Section 9.4.3.1. An analysis of the existing structure for wind loads was not completed as part of our review as the lateral bracing has performed satisfactorily for 14 years in a similar exposure situation to the proposed location in Manchester. Member allowable capacities were calculated using the 2005 NDS with the exception of the glulam members for which design values on the plans were used. Notably, these values did not correspond exactly to any standard glulam combination in the 1997 or 2005 NDS nor the 1993 AITC Glulam manual.

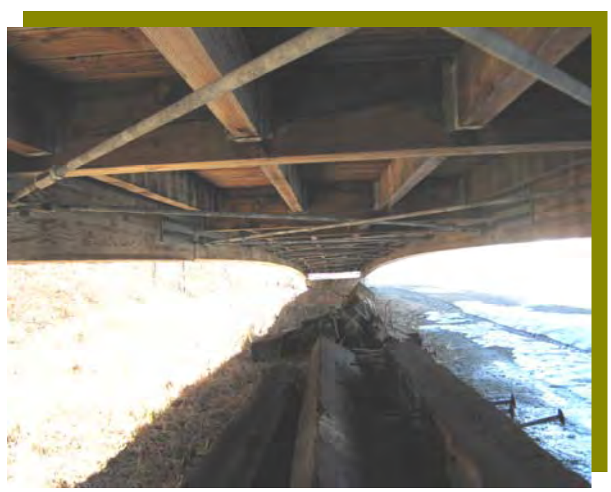


Bridge Trusses

Three vertical load combinations were analyzed for the lengthened bridge per ASCE 7-10. The bridge was reportedly designed for a 7,000 pound vehicle with unknown wheel configuration; however this loading was not evaluated as the intended use of the bridge will be pedestrian. The live load is reduced from 85 PSF used for the floor framing to 65 PSF as allowed by AASHTO for components that have larger tributary areas. The first load combination applies only dead load (the weight of the structure itself) and a 65 PSF uniform live load. The second load combination included full dead load and snow load while the third load combination combined full dead load with ¾ live and ¾ snow load. The results from these analyses were compared to the inventory stress values taken from the NDS code.

The truss web and chord members as well as the lower chord splices and end diagonal plates were evaluated for the lengthened bridge with the previously discussed load combinations. Our analysis indicates that the lengthened bridge is capable of supporting the previously discussed load combinations.

9.4.3.3 Floor Framing



Floor Framing

The floor beams and stringers are pressure treated Douglas Fir members with a grade of No. 1. They were analyzed for dead load and a live load 85 PSF in accordance with the AASHTO Guide Specifications for the Design of Pedestrian Bridges. Due to the smaller tributary area, the 85 PSF live cannot be reduced as is the case with the trusses.

Both the floor beams and stringers were determined to be adequate for these loads.

The connections between the floor beam and trusses are custom galvanized steel plates while the stringer to floor beam connections are pre-fabricated connections. These connections were not evaluated as part of our analysis; however the shop drawings provide design loads for these connections so presumably they are structurally adequate.

9.4.4 Estimate of Probable Construction Costs

An estimate of Probable Construction Costs was prepared for relocation of the bridge from Dover, NH to Manchester, NH as well as an estimate for repairs and recommended improvements to the bridge. These estimates were calculated utilizing our recent covered bridge project bid prices, recent municipal bridge aid project bid results and Means Guides. The relocation cost for the bridge was also peer reviewed by Wright Construction Company, Inc, a noted covered bridge contractor.

The relocation method assumed in our cost estimate includes the following steps:

<u>Disassembly</u>

1. Cut the roof in the locations of the two existing top chord splices and temporarily disconnect upper bracing and electrical conduit in these locations.
2. Temporarily support one third of the roof structure with a crane and remove the top chord through bolts at each top panel point in this third of the bridge.
3. Remove one third of the roof and top chord as a section and repeat for the remaining two sections.
4. Remove the bolts at each bottom panel point and remove the truss web diagonals.
5. Remove the interior railing, rub rail and siding at the two bottom chord splice locations.
6. Remove one section of stringers, decking and lateral 'x' bracing adjacent to each splice location.
7. Temporarily support the bottom chord and unbolt at each splice location leaving three sections of lower chord and framing.
8. Transport the sections of the bridge to Manchester. It is assumed that seven trips will be necessary - 3 for the roof sections, 3 for the lower chord and framing and a final trip for the web members and other removed pieces.

<u>Reassembly:</u>

1. Complete necessary abutment modifications, generally consisting of new concrete bearing pads and backwalls.
2. Remove railroad trestle bents 1, 5 and 6 with bents 2, 3 and 4 to remain.
3. Replace the end sections of the lower chord with longer sections and replace the end lower panel point connection plate and web members. Reassemble the truss into 3 complete sections on the approach to the Piscataquog River Crossing.
4. Complete to the greatest extent possible, the recommended repairs discussed below prior to installing the bridge over the Piscataquog. The south and middle section of the covered bridge will be assembled on south approach to the crossing and the north section on the north approach.
5. Utilizing a crane at the south abutment, pick the center section of the bridge and put in place over the river. Provide temporary blocking as required above the remaining bents.
6. Utilizing a crane at the south abutment, pick the south section of the bridge and put in place over the river. Reconnect the two sections at the chord splices.
7. Utilizing a crane at the north abutment, pick the north section of the bridge and put in place over the river. Reconnect the two sections at the chord splices.
8. Complete remaining work to the covered bridge and remove the remaining three bents.

The total estimated cost of the disassembly of the bridge, transportation to Manchester and reassembly over the Piscataquog River is $105,000, which was peer reviewed by Wright Construction Company, Inc. In addition to this cost, the bridge will require modification of the lower chord and end diagonal members which is estimated at $55,000. These costs do not include the cost of the recommended repairs to the bridge discussed below, abutment modifications or removal of the existing trestle. The relocation cost can be compared to a cost of $125,000, which was prepared by T. Buck Construction, Inc. for the Town of Kennebunkport, ME as they considered purchasing the bridge and relocating it to their Town.

As discussed in Section 9.4.2, the DCB is a 15 year old structure that will require some repair and maintenance work due to the condition of some of its members. The removal of graffiti from the bridge is recommended, but not included below as it is assumed that City of Manchester forces will complete this work. A listing of these recommended improvements and their estimated cost is included below.

Improvement	Estimated Cost
1. Remove the existing shingle roof and replace with a standing seam metal roof.	$30,000
2. Remove holiday lighting, repaint eave trim and siding.	$5,000
3. Clean and apply a coal tar epoxy coating to all stringer metal connections which are rusting.	$2,500
4. Install a foam covering at metal 'x' bracing to prevent banging noise.	$1,500
Total Estimated Cost of Recommended Improvements	$39,000

The total estimated bridge construction cost of Alternative No. 4 in 2012 dollars is approximately $332,700. Adding the estimated trail construction cost of $150,010 and 15% for contingencies gives a total estimated project construction cost of $555,450.

10 CONCLUSIONS AND RECOMMENDATIONS

An Engineering Study of upgrading approximately 1,800’ of rail bed and four options for a river crossing at an existing timber trestle was completed. Initial reviews of the project were completed by natural and historic resource agencies.

The proposed trail will be constructed over the existing rail bed utilizing paved and gravel surfaces. The trail portion also includes timber railing as appropriate, an at-grade crossing at Electric Street and wayfaring improvements at an intersecting trail. The estimated construction of the trail portion of the project in 2012 dollars is approximately $150,010.

The trailbed within the project limits crosses the Piscataquog River where a former railroad timber trestle exists. The following bridge alternatives were studied for this crossing:

- Alternative No. 1 – Rehabilitate Existing Trestle
- Alternative No. 2 – Modify Existing Trestle
- Alternative No. 3 – New Steel Truss Bridge
- Alternative No. 4 – Dover Covered Bridge

Alternative No. 2 which included removal of two piers for improved debris flow under the bridge was eliminated from further consideration due to the trestle's structural inadequacy for ice loads in a modified configuration.

Alternative No. 4 was eliminated from consideration after the availability of the bridge became uncertain due to political issues.

The remaining alternatives (1 & 3) were then compared to determine which alternative best met the City's project goals. The main advantage to Alternative No. 1 is a lower initial construction costs, however its long term service life is anticipated to be much less than Alternative No. 3. Alternative No. 1 does not improve the ongoing build-up of debris at the crossing which can exacerbate upstream flooding and is costly to remove. Since Alternative No. 3 as a single span would greatly improve flow through the crossing and it would have a much longer service life, Alternative No. 3 is the recommend bridge option.

The total estimated construction cost in 2012 dollars for the trail portion and Alternative No. 3 including contingency is $637,100.