REHABILITATION STUDY REPORT

# I-93 CORRIDOR IMPROVEMENT PROJECT

## BOW-CONCORD - STATE PROJECT NO. 13742-A

## REHABILITATION STUDY REPORT
## EXIT 14 - BRIDGE NO. 163/106


PREPARED BY:

McFarland Johnson

53 REGIONAL DRIVE
CONCORD, NH 03301
(603) 225-2978


PREPARED FOR:

New Hampshire DOT

JANUARY 2009

# TABLE OF CONTENTS

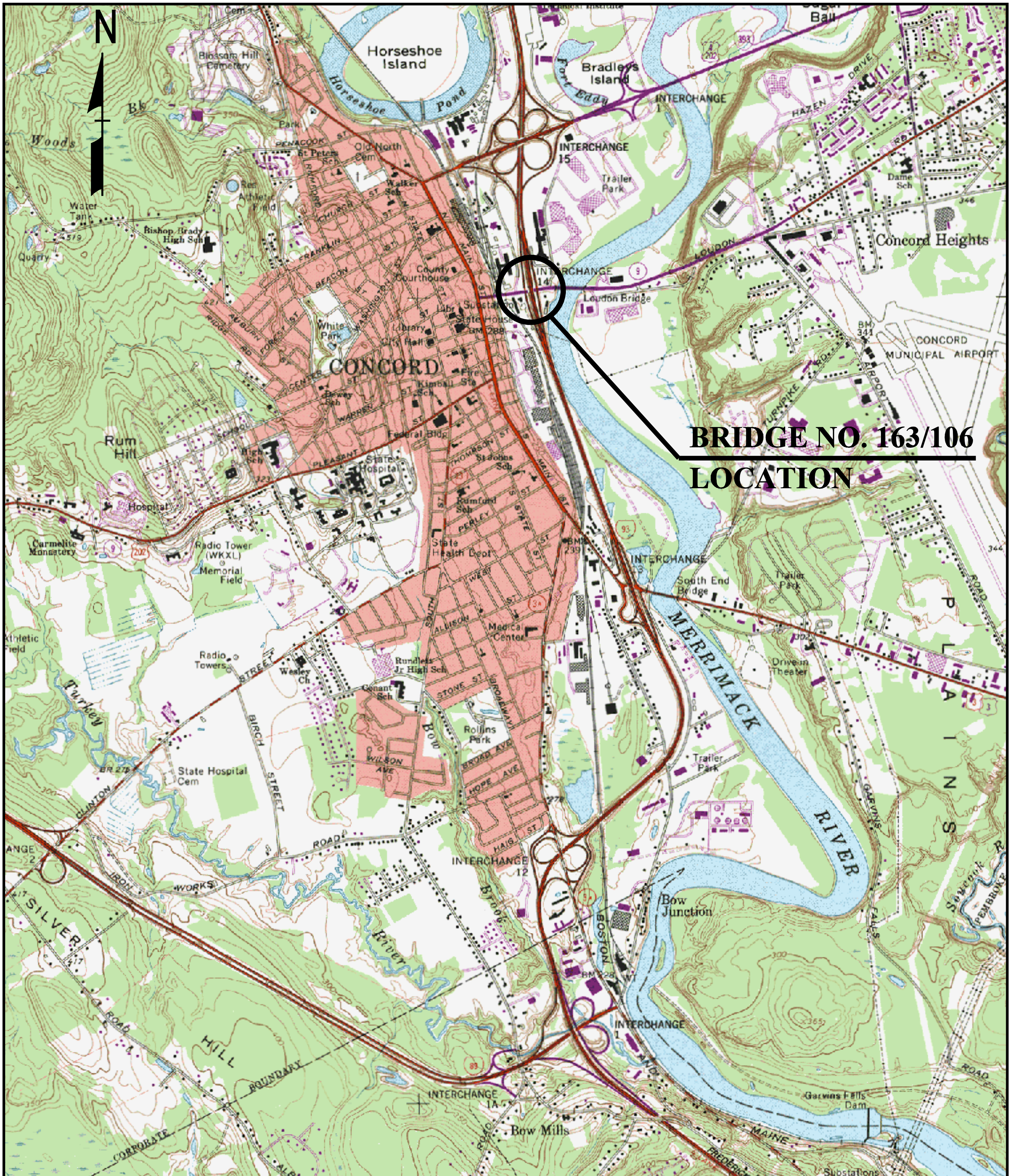
## LIST OF APPENDICES



## LIST OF FIGURES



## LIST OF DRAWINGS




2500 0 2500 5000

EXIT 14 BRIDGE REHABILITATION STUDY

PROJECT LOCATION PLAN

New Hampshire DOT

**McFarland Johnson**

53 REGIONAL DRIVE, CONCORD, NH 03301-5022
PH: 603-225-2978 FAX: 603-225-0095

FIGURE NO. 1

## EXECUTIVE SUMMARY

The purpose of this Rehabilitation Study Report is to evaluate the rehabilitation needs of Bridge No. 163/106, which carries I-93 (F.E. Everett Turnpike) northbound and southbound over NH Route 9 (Loudon Road) at Exit 14. A key component of the study is to evaluate traffic control alternatives to minimize traffic impacts along Interstate 93 and Loudon Road.

The bridge was originally constructed in 1966, with minor bridge approach rail and deck joint rehabilitations completed in 1970 and 1977 respectively. The bridge is currently on the State's Red List and is considered structurally deficient due to the deteriorated condition of the concrete deck slab. The deck slab is from the original construction, and at 42 years of age, is significantly beyond its life expectancy for a bridge of that era. The bridge has a federal sufficiency rating of 59.7% out of a possible score of 100%.

The study evaluates three bridge deck types for the proposed rehabilitation. Deck types considered include: conventional cast-in-place concrete, full-depth precast deck panels, and precast Exodermic™ deck panels. In addition, two prefabricated bridge superstructure options were evaluated for use. The first utilizes the Inverset™ type system with conventional crane construction, and the second utilizes near-site bridge prefabrication and Self-Propelled Modular Transporters (SPMTs) to replace the entire bridge superstructure.

The following three traffic control alternatives are also evaluated as part of this bridge rehabilitation study:

Alternative A: 2-Phase Construction with 3-Lanes using Night or Weekend Lane Closures
Alternative B: 3-Phase Construction with 4-Lanes using Temporary Deck Widening
Alternative C: 2-Phase Construction with 4-Lanes using Temporary Bridge and Detour Roadway

This Rehabilitation Study discusses the advantages and disadvantages of the deck replacement options as well as the traffic control alternatives. A summary of the proposed rehabilitation is provided below in Table 1.

**Table 1 – Summary of Proposed Rehabilitation**

| CRITERIA | DESCRIPTION |
|---|---|
| Recommended Alternative: | Alternative A1:Full-depth Precast Panels |
| Estimated Construction Cost: | $2,310,000 |
| Traffic Impacts: | Lane reductions, reduced speed limit, and oversize vehicle restrictions during both weekend work periods on I-93. Loudon Road closed at bridge during both weekend closures. |
| Utility Impacts: | None |
| Construction Duration: | 3 months |

## PROJECT DESCRIPTION

State Project No. 13742-C is the first of three programmed bridge rehabilitation contracts included under the larger Interstate 93 Bow-Concord Corridor Improvement Project. The primary purpose of Project No. 13742-C is to rehabilitate Bridge No. 163/106, which carries I-93 (F.E. Everett Turnpike) northbound and southbound over NH Route 9 (Loudon Road) at Exit 14. See Figure 1 – Project Location Plan.

The second programmed bridge rehabilitation contract (13742-B) is intended to rehabilitate two underpass bridges within the I-93 and I-89 interchange. These two bridges carry I-93 northbound (Bridge No. 136/160) and I-93 southbound (Bridge No. 135/160) over Interstate 89 and the Turkey River. The third and final bridge rehabilitation contract (13742-A) is intended to rehabilitate the overpass bridge at Exit 12 (Bridge No. 203/087), which carries NH Route 3A over I-93 northbound and southbound.

The four bridges noted above are currently on the State's Red List, which indicates the bridges are structurally deficient, with one or more of their structural elements rating "4" or less out of "0" to "9" rating scale. It is currently anticipated these bridge rehabilitation projects will be funded exclusively by Turnpike revenues. These bridges are part of the State's initiative to reduce Red List bridges, in large measure, by utilizing the Turnpike revenue increases approved in the Fall of 2007.

## EXISTING CONDITIONS

### <u>Roadway</u>

Interstate 93 through Concord was constructed in phases beginning in the mid 1950's and completed in or around 1966. The intersection of what is now I-93 and what is now Loudon Road, and was then Bridge Street, was originally constructed as a traffic circle in 1948. The traffic circle was retained when the first interstate project was completed in 1957. Exit 14 in its current configuration was mostly constructed in 1966. The northbound exit ramp was re-aligned opposite Fort Eddy Road in 1989. While there is approximately 5,900 feet (1.1 miles) between Exits 13 and 14, there is only approximately 2,800 feet (1/2 mile) between Exits 14 and 15. See Drawing No. 1 – Existing Conditions Plan.

Interstate 93 through Concord is a four-lane divided urban principal arterial highway with full access control. The posted speed limit is 55 miles per hour with a design speed in excess of 60 miles per hour. The most recent Average Annual Daily Traffic (AADT) from 2007 indicates approximately 70,000 vehicles per day (vpd) passing through Exit 14. The volume is seasonal with increases during the summer vacation and winter skiing seasons (see Photos No. 1 & 2, Appendix A for aerial views of the project area).

In the vicinity of Exit 14, I-93 has two 12-foot lanes in each direction, 10-foot outside shoulders, and 4-foot inside shoulders. There is a raised 8-foot wide concrete median with a steel box beam rail in the center. To the north the median transitions to grass with a steel double faced guardrail (see Photos No. 3 & 4, Appendix A for views of Exit 14). The horizontal alignment over Loudon Road consists of a 7,900-foot horizontal curve that ends approximately 175 feet north of the bridge. I-93 is superelevated through the curve, over the bridge with the rate being approximately 2%. The vertical alignment of I-93 consists of a symmetrical crest vertical curve whose VPI is at the midpoint of the bridge. Grades of -2.12% extend north and south from the bridge with a 1,400-foot long crest vertical curve. The stopping sight distance over the crest curve exceeds 330 feet, which exceeds the requirements for 75 mph.

Exit 14 is a full diamond interchange providing access to Loudon Road (NH Route 9). There are four ramps associated with Exit 14 that are controlled by three signalized intersections. These are single-lane ramps, however, the southbound entrance ramp begins with two lanes and merges to one before merging onto I-93. The southbound exit and entrance ramps terminate at Loudon Road at a signalized intersection approximately 65 feet west of the bridge. The northbound entrance ramp has its own signal approximately 65 feet east of the bridge. The northbound exit ramp terminates at Loudon Road opposite Fort Eddy Road at a signalized intersection approximately 300 feet east of the bridge. There is a fourth signalized intersection west of I-93 that connects Stickney Avenue to Loudon Road. Altogether, the four signalized intersections are located along a length of approximately 625 feet.

Loudon Road provides access to downtown Concord and the State Capital to the west and to the east across the Merrimack River to the Heights district of Concord, including the commercial areas along Loudon Road as well as the State office complex. Under I-93 the 2005 AADT was approximately 25,000 vpd.

As it passes under I-93, Loudon Road is approximately 70 feet wide, curb to curb, with 5-foot sidewalks on both sides. There are a total of seven lanes under the bridge. Four of the lanes, two in each direction, are for through traffic. One lane is a left turn lane for eastbound traffic heading for the northbound entrance ramp and two lanes are for left turn lanes for westbound traffic heading for the southbound entrance ramp. This is a new configuration installed in 2007 and it eliminated the concrete island under the bridge (see Photos No. 5 & 6). Under the bridge the sidewalks are behind the curbs, but they are set back as they move away from the bridge with a buffer up to 15-feet. The sidewalks are bituminous with a minimum width of 5 feet. The vertical clearance under the bridge is approximately 15’-0”, the minimum occurs on the east side at the crown of Loudon Road.

## Bridge

The bridge was originally constructed in 1966, with minor bridge approach rail and deck joint rehabilitations completed in 1970 and 1977 respectively. The single span, slab-on-stringer bridge utilizes painted steel rolled beams (W36x230) composite with a concrete deck, and spans 82 feet between centerlines of bearings. There are a total of twelve stringers, six for each barrel of the interstate, which are spaced at 7’-6”. Welded, partial length cover plates (14”x 3/4”) are used on the bottom flanges of the rolled beam stringers, with the cover plates terminating approximately 15 ft from the beam ends. Steel rolled channel (C15x33.9) diaphragms are located at the girder ends and approximate quarter points of the span. See Drawing No. 2 – Bridge Typical Sections, which illustrates the existing bridge typical section.

The painted steel beams were noted to be in **SATISFACTORY** condition on the latest Bridge Inspection Report. The major deficiencies noted were peeling paint, light rusting, and minor section loss of the steel near the abutments. Scrapes and gouges are also visible on the steel cover plates, which is notable due to the fatigue-prone welds at the cover plate terminations (see Photo No. 7).

The reinforced concrete deck is from the original construction and consists of a 7” nominal thickness, topped with a waterproof membrane and asphalt wearing surface. Separate deck and framing systems are used for each interstate barrel, which are separated by a longitudinal expansion joint along the centerline of the bridge. Each framing unit supports two lanes of traffic and has a curb-to-curb width of 37’-3” and an out-to-out deck width of 43’-5”. The total out-to-out deck width for the combined framing units is 86’-10”.

The concrete deck was noted to be in **SEVERE** condition on the latest Bridge Inspection Report. There are several large deck delaminations on the underside of the deck, with heavy cracking and leakage evident throughout. Timber forms and temporary deck shoring have been in place below the structure for at least the past 10 years, and partial netting is also in place below the bridge to prevent deteriorated deck concrete from falling on motorists and pedestrians (see Photo No. 8).

The bridge superstructure is a fixed condition at the south abutment (Abutment A), and an expansion condition at the north abutment (Abutment B). A slab-over-backwall detail exists at Abutment A, and a deck expansion joint (neoprene compression seal) is used at Abutment B. Steel bridge shoes are used at both abutments, with fixed shoes at Abutment A and sliding shoes (bronze plate) used at Abutment B. Galvanized steel bridge railing (2-rail) is used along each bridge fascia, with weak-post box beam guardrail used along the median. The bridge rails, rail transitions, approach rails and approach rail terminations do not meet current design criteria for new construction. Bridge mounted sign support structures are located on both fascias of the bridge, and they are connected to both the steel stringers and the concrete brush curbs (see Photo No. 9).

The superstructure is supported by cast-in-place, full-height cantilever abutments and U-back wingwalls founded on pile supported footings. The steel bearing piles (HP 12x53) were presumably driven to bedrock, with estimated pile lengths varying between 75 ft and 95 ft based on information obtained from the original bridge plans. The original soil boring logs indicate the bridge is underlain by a thick layer of soft silt and clay soil (i.e. blow counts < 10). Buried approach slabs (20’ length) exist at each end of the bridge except within the median. The substructure was noted to be in **POOR** condition on the latest Bridge Inspection Report. The major deficiencies noted were large concrete spalls and delaminations, many with exposed and corroded reinforcing steel (see Photo No. 10).

**Utilities**

The majority of existing above and below ground utilities in the project area run along Loudon Road. Overhead utilities include electric, telephone, and cable. Underground utilities include water, storm drain, sanitary sewer, gas, telephone, and traffic signal communications. There are also underground electric lines to power the lighting that exists along Loudon Road and the Exit 14 ramps.

There do not appear to be any utilities carried on the Exit 14 Bridge. Underground water and gas that run along Loudon Road are within 10 feet of the front face of abutments. These should not require relocation by the proposed rehabilitation, but temporary shoring could be required for options that utilize temporary abutments. There are electric transmission lines that run along Loudon Road and then cross over I-93. These transmission lines run just south of the southern bridge abutment and will likely be a constraint during construction.

## NATURAL & CULTURAL RESOURCES

The Natural and Cultural Resources for this study are based upon information presented in the *Summary/Classification Report* for the Bow-Concord Interstate 93 Transportation Planning Study completed in April 2008. Resources were mapped by using available mapping from the New Hampshire Geographically Referenced Analysis and Information Transfer System (NH GRANIT), other available published mapping, and by field review and study for some resources (historic and archaeological resources). Floodplain mapping within the City of Concord limits was provided by the City.

### Water Based Resources

Surface and Ground Waters: The most significant surface water within the study area is the Merrimack River. The Merrimack is a fourth order stream with a watershed that originates in the White Mountains and measures, in total, approximately 5000 square miles. The river flows south through New Hampshire and then east to Newburyport, Massachusetts where it flows into the Atlantic Ocean. Three bridges span the Merrimack River in Concord; at Exit 13 (Manchester Street), at Exit 14 (Loudon Road), and at Exit 15 (I-393). I-93 is west of and parallels the river near Exit 14. Just south of Exit 14, the buffer between the river and I-93 is as narrow as 20 feet.

North of Loudon Road, the river is classified on the National Wetland Inventory (NWI) map as Cowardin classification R2UBH, or lower perennial with an unconsolidated bottom, and permanently flooded. South of Loudon Road, the river is Cowardin classification L1UBHh, or lacustrine, limnetic (deepwater), with an unconsolidated bottom, permanently flooded, and impounded. The impoundment is created by the dam at Garvin Falls, about 5 miles downstream. The river quality is classified as Class B water, which means that it is suitable for recreational activities, such as swimming and fishing, but non-potable without treatment.

Portions of the riparian areas associated with the Merrimack River are highly developed with little natural buffer retained. Other portions are in agricultural use for corn or other crops, including the corn fields south of Exit 14, on the east side of the river. The banks are vegetated with silver maple, red maple, green ash, basswood, gray birch, and other species.

Floodplains: Large portions of the I-93 corridor lie within the 100-year floodplain of the Merrimack River, including the retail development on the east side of I-93 between Exits 14 and 15 and the industrial areas along Stickney Avenue. Portions of the floodplain are inundated seasonally, whereas other portions are inundated with less regularity. Drawing No. 3 – Water Based Natural Resource Plan, depicts the floodplains in the Exit 14 vicinity.

Wetlands: Wetlands were mapped using NWI data provided through GRANIT. Drawing No. 3 – Water Based Natural Resource Plan, depicts the Wetlands in the Exit 14 vicinity and indicates that none are present near the bridge or roadways. The wetlands run adjacent to the Merrimack River north of the Loudon Road Bridge.

**Land Based Resources**

Farmlands: Farmlands in the project vicinity include cornfields southwest of Exit 14 along Loudon Road and the east bank of the Merrimack River. These fields are in active cultivation but would not be affected by the proposed bridge rehabilitation.

Vegetation and Wildlife: Vegetation communities in the Exit 14 area include remnant floodplain forests and agricultural fields. Lower silver maple floodplain forests exist between Exits 14 and 15.

Wildlife habitat in the study area includes the Merrimack River and associated riparian areas, marshland, farm fields, and upland forest. The river corridor provides habitat for many species of fish, amphibians, reptiles, mammals, and birds. Although portions of the corridor are highly urbanized with little intact riparian buffer, there are stretches of expansive undisturbed vegetation. Even areas with very little intact buffer, such as along I-93 south of Exit 14, provide passage for aquatic mammals and fish.

Rare species information is available from the New Hampshire Natural Heritage Bureau, the New Hampshire Department of Resources and Economic Development – Division of Forests and Lands, the New Hampshire Fish and Game Department, and the US Fish and Wildlife Service. The rare species database identifies one potential species that exists near Exit 14. Species specific information is not provided.

Conservation/Public Lands: Conservation land within the study area is mapped using information provided by GRANIT. The nearest public lands include land protected by the City of Concord (in part associated with the landfill) north of Exit 13.

Hazardous Materials: An Environmental Database Study was conducted for the Bow-Concord I-93 Transportation Planning Study. The purpose of the database study was to identify potential oil and/or hazardous material sites through a database search and a windshield survey. No review of New Hampshire Department of Environmental Services (NHDES) files was performed as part of the Environmental Database Study. The search identified numerous sites surrounding Exit 14 where potential sources may exist. Three sites were identified along I-93 where the interstate and river are very close to each other. Several sites were identified along Stickney Avenue where the NHDOT maintenance yard and vehicle pool once existed.

## Cultural Resources

Historic / Architectural: A detailed description of historic resources can be found in the *Historic Resources Constraints Report* prepared for the Bow-Concord I-93 Transportation Planning Study. The resources in the vicinity of Exit 14 include the Page Belting Historic District adjacent to the southbound entrance ramp and the NHDOT maintenance buildings along Stickney Avenue.

Archeological: A detailed description of archeological resources can be found in the *Phase I-A Archeological Reconnaissance Technical Report* prepared for the Bow-Concord I-93 Transportation Planning Study. The report concludes that known and likely pre-contact Native American archeological resources exist throughout the project area. However, the report also revealed extensive disturbance in the area due to land clearing, road construction, farming and river damming where resources are believed to be absent. In the vicinity of Exit 14, the west bank of the Merrimack River north of Loudon Road was declared sensitive for archeological resources.

## GENERAL BRIDGE EVALUATION

### Fatigue Analysis

The existing bridge was reviewed for fatigue prone details to determine whether members should be retrofitted or replaced as part of the proposed rehabilitation. The only fatigue-prone details identified on the bridge were the partial length, welded cover plates connected to the bottom tension flanges of the rolled beam stringers.

The base metal adjacent to the welds at the ends of these cover plates represent a Stress Category E′ detail, which is now the lowest stress category recognized by the AASHTO LRFD Bridge Design Specifications. The allowable fatigue stress (LRFD Constant Amplitude Fatigue Threshold) for a redundant load path structure, such as this bridge, is only 2.6 ksi for an E′ detail.

The most important parameter in the fatigue evaluation process is the effective stress range at the detail under consideration. Several methods are available to obtain the effective stress range, including, but not limited to:

- Stress range histograms from field measured strain gauge data
- Truck gross weight histograms using weigh-in-motion measurements
- Analytical model of the bridge using an equivalent fatigue truck

The analytical model approach was selected for this bridge since it requires the least amount of effort and is known to produce conservative results on fatigue life estimates for typical stringer bridges. If the analytical results indicate the bridge has an unacceptably short, safe remaining fatigue life, the other more intensive and costly methods could be explored.

In order to assist with the fatigue analysis, the bridge was modeled using the Merlin-Dash (V 8.1) software program. For the fatigue analysis, the program was only used to calculate the live load moment at the end of the partial length cover plate, as well as to determine the LRFD live load distribution factor for fatigue.

Once the live load stress range was determined, two hand calculation methods were used to estimate the safe remaining fatigue life of the girders. First, the girder was analyzed in accordance with the *AASHTO Guide Specifications for Fatigue Evaluation of Existing Bridges (1990)*. The Guide Specification evaluation approach has been used on thousands of bridges and is generally recognized as providing conservative estimates of safe remaining fatigue life.

The second method utilized the *AASHTO Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges (2003)*. This method uses a very similar approach as the 1990 Guide Specification. However it is based on current research, and is generally recognized to produce longer fatigue life estimates when compared to the Guide Specifications[1].

[1] *Comparative Study of Fatigue Provisions for the AASHTO Fatigue Guide Specifications and LRFR Manual for Evaluation* ASCE Journal of Bridge Engineering, Volume 11, Issue 5, pp. 655-660 (September/October 2006)

Although the two methods utilize different fatigue trucks, impact factors and distribution factors, they both indicated the fatigue-prone details have a finite remaining fatigue life. A summary of the parameters used to calculate the effective stress range for both methods is provided in the following table:

**Table 2 – Fatigue Stress Range Parameters**

| Guide Specifications (1990) | |
|---|---|
| Fatigue Truck | HS-15 |
| Load Factor | n/a |
| Impact Factor | 1.10 |
| Distribution Factor | 0.35 |
| Composite Section Increase | 15% |
| Effective Stress Range | 1.14 ksi |
| Maximum Stress Range | n/a |
| Limiting Stress Range | 0.9 ksi |
| Fatigue Life Result | **Finite** |

| LRFR Evaluation Manual (2003) | |
|---|---|
| Fatigue Truck | HS-20 |
| Load Factor | 0.75 |
| Dynamic Load Allowance | 1.15 |
| Distribution Factor | 0.38 |
| Composite Section Increase | n/a |
| Effective Stress Range ($\Delta f_{eff}$) | 1.48 ksi |
| Max. Stress Range (2*$\Delta f_{eff}$) | 2.96 ksi |
| Constant Amplitude Fatigue Threshold ($\Delta F_{TH}$) | 2.6 ksi |
| Fatigue Life Result | **Finite** |

Since the results of the analyses indicate the details have a finite remaining fatigue life, the safe remaining fatigue life of the detail was calculated using both evaluation methods for comparison purposes. The two primary variables that affect the remaining fatigue life are the age of the bridge and the number of stress range cycles experienced by the detail. Conservatively, we used current (2008) values for the average annual daily traffic (AADT) and percent truck traffic, which were 70,000 vehicles per day and 6% respectively. Had the predicted remaining fatigue life been deemed unacceptable, the calculation could have been refined to use AADT and % truck variables that were averaged over the life of the bridge.

The Guide Specification method indicates the detail has a safe remaining fatigue life of 91 years, while the LRFR method indicates the detail has a safe remaining fatigue life of 126 years. These results are consistent with the comparison study noted previously, whereby the LRFR method typically predicts longer finite life estimates than the Guide Specifications.

The results of the analyses indicate the fatigue-prone cover plate details on this bridge still have considerable safe remaining fatigue life. The predicated remaining life is much greater than the life expectancy of a replacement deck, and most likely greater than the remaining service life of the entire bridge. The analysis approach used is known to be conservative, and additional evaluation methods or procedures could have been employed to increase the fatigue life predictions even further, if required.

In light of the analysis results and the factors noted above, we do not believe it is necessary to implement a fatigue retrofit of the partial length welded cover plates. We do recommend that Department personnel continue to closely inspect and monitor these details on a regular basis, to look for possible load-induced or distortion-induced fatigue behavior. A copy of the fatigue analyses calculations is included in Appendix D.

## Seismic Evaluation

The existing bridge was reviewed for current seismic detailing to determine whether members should be retrofitted or replaced as part of the proposed rehabilitation. The AASHTO LRFD Bridge Design Specifications, 4th Edition (2008 Interim Revisions) was used for this review.

Seismic analysis is not required by the LRFD Specifications for single span bridges. The connections between the bridge superstructure and the abutments must be checked for adequacy, and the minimum support length requirements must also be satisfied. The existing 1" diameter anchor bolts at the fixed end of the bridge were evaluated and found to be adequate to restrain the bridge for the required longitudinal seismic connection design force.

The bridge seats were also evaluated and found to provide support lengths greater than the minimum required by AASHTO. A copy of the calculations from the seismic evaluation is included in Appendix D.

## Deck Expansion Joints

The bridge records on file at the Department indicate the expansion joints were last reconstructed in 1977, only 11 years after the original bridge construction. The existing transverse deck expansion joint at Abutment B consists of an armored, 3" neoprene compression seal below the roadway, and sliding steel plate assemblies at the median and brush curbs. The 1977 rehabilitation also included the installation of a longitudinal neoprene joint soil along the centerline of the concrete median. We were unable to verify whether this joint seal is still in place, but it is likely unserviceable due to its age and the fact it was installed in a vertical open joint without a formed header.

The proposed rehabilitation will require the elimination or replacement of the existing transverse deck expansion joint at Abutment B. The existing compression seal has failed and there is significant leakage through the joint. This joint leakage has undoubtedly contributed to the extensive spalling and delamination of the deck and abutment concrete visible at the north end of the bridge.

The length of bridge deck contributing to thermal movement is approximately 85 feet, and the bridge skew is less than 2 degrees. The expansion joint guidelines in the NHDOT Bridge Design Manual (Plate 641.2b) suggest that deck joints be avoided for bridges with movement lengths less than 80 feet. Since this bridge length is slightly greater than the 80 foot limit, an asphaltic plug joint system could be used.

An asphaltic plug joint system would provide a smoother riding surface and longer pavement life at the joint. However, these systems do have disadvantages, including susceptibility to rutting at high ambient temperatures from heavy truck traffic or from acceleration and braking forces. The Department has chosen to waive the joint requirement for this rehabilitation project and use a modified plug joint (reference Appendix F for Meeting Minutes). A deck-over-backwall retrofit is recommended at Abutment B to eliminate the open expansion joint.

The deck-over-backwall retrofit at Abutment B would add a level of complexity to the project if rapid construction methods are utilized, which will be discussed further in the Traffic Control Evaluation section of the report. However, we are confident that a suggested sequence of construction can be developed during the design process that will allow the backwall reconstruction to be completed while still utilizing rapid bridge construction methods.

The existing longitudinal neoprene joint seal along the median should also be replaced as part of the proposed rehabilitation. If concrete safety shape barriers are used on the rehabilitated bridge, a joint seal detail could be installed between the shapes to minimize leakage through the longitudinal median joint.

**Bridge Bearings**

The existing bridge bearings do not require seismic retrofit as discussed previously in the Seismic Evaluation section of the report. However, the bridge inspection reports on file at the Department indicate both the fixed and expansion bearings are rusted, and additional section loss is present on the expansion bearings. The extent of section loss on the bearings is currently unknown, but it should be accurately quantified prior to making a determination on rehabilitation or replacement of the bearings. This assessment could be made as part of the next regularly scheduled bridge inspection (6-month interval for a Red List bridge), or a special inspection could be performed.

A special investigation was conducted on November 4, 2008 to visually inspect and photograph the steel bridge shoes. The expansion shoe assemblies exhibit active corrosion and section loss and it would be preferable to replace these expansion bearings as part of the proposed rehabilitation (Reference Appendix F for the Special Inspection of Steel Bridge Shoes project memorandum).

As with a deck-over-backwall retrofit, replacing the bridge bearings would complicate the proposed rehabilitation if rapid bridge construction methods are utilized. It is unlikely the bridge jacking and bearing replacement work could be completed during the same weekend closure used under the precast deck replacement option. This would, therefore, require the jacking and bearing replacement be completed during a separate closure period, either prior to or following the closure for the precast deck construction. The weekend closure option and rapid bridge construction techniques will be discussed in more detail in the Traffic Control Evaluation section of the report.

In an ideal situation, the bearings would be replaced prior to construction of the new deck to avoid unnecessary stress on the deck from the bridge jacking operations. However, due to the severely deteriorated condition of the existing deck, it would seem unwise to perform the jacking operation on the existing deck, and expect it to remain serviceable until the next weekend closure period. The decision to rehabilitate or replace the bearings should be made in conjunction with the selected deck replacement type, and a “one size fits all” approach may not be appropriate in this case.

## Substructure Repairs

The concrete spalls and delaminations at both abutments should be repaired as part of the proposed rehabilitation. This work would require a temporary closure of the sidewalk and at least one travel lane, and therefore, it may be appropriate to perform these repairs during off-peak travel times. The partial-depth repairs would remove the deteriorated concrete to a depth at least one inch beyond the front mat of reinforcing steel. Once all of the repairs have been completed, an epoxy coating could be applied to the entire exposed surface of the abutments for increased protection of the concrete and improved aesthetics.

## Steel Painting

As discussed previously, the paint on the existing steel stringers is peeling and light rust is visible throughout. On October 28, 2008, the Department's in-house paint specialist reviewed the condition of the existing paint and recommended the existing paint be completed removed and the steel repianted with a three coat system. For additional information on his recommendations, reference Appendix F for the Bridge Painting and Recommendation & Estimate Memorandum.

# DECK REPLACEMENT OPTIONS

As part of the initial project scoping, the following three deck options were selected for evaluation as part of this study: cast-in-place deck, full-depth precast deck panels, and precast Exodermic™ deck panels.

In addition to discussing these deck replacement options, this section of the report also evaluates two options for replacement of the entire superstructure. The first superstructure replacement option discussed is a system using prefabricated bridge units (Inverset™ type). The final superstructure replacement option discussed is a system using self-propelled modular transporters (SPMTs). All deck replacement options discussed below would provide a durable deck solutions.

## Cast-In-Place Deck

This option involves replacing the existing reinforced concrete deck with a similar cast-in-place (CIP) deck, although the use of partial-depth, precast stay-in-place (SIP) forms may be used.

This deck option would be used for the three-phase temporary deck widening, and the two-phase temporary bridge traffic control

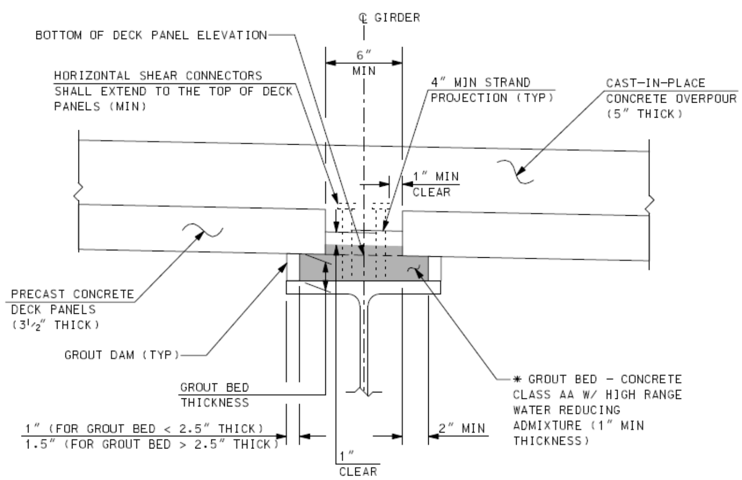



**Figure 2 – Precast SIP Panel Detail**

alternatives, which will be discussed further in the next section of the report.
The advantages of this deck system include low initial cost and good long-term durability. The primary disadvantage of this system is longer construction duration with a greater disruption to traffic.

Another potential disadvantage of the CIP deck system is the total thickness and weight of the deck would likely increase under this option. The Department's current approved precast SIP deck panel system uses a 3-1/2" precast deck panel with a 5" thick concrete overpour, for a total structural deck thickness of 8-1/2". The existing deck has a nominal thickness of only 7", and the additional weight and geometry implications of the CIP deck option would need to be evaluated further.

From a structural perspective, the additional deck weight may not be a significant design issue. Based on our review of the Bridge Capacity Summary (Form 4) on file at the Department, the existing steel stringers appear to have significant reserve capacity. From a geometric perspective, however, the increased deck weight would reduce the vertical clearance under the bridge by approximately ½" due to the additional stringer deflections. This issue could be mitigated by either replacing or shimming the existing bridge bearings to retain the minimum vertical clearance below the bridge.

**Full-Depth Precast Deck Panels**

This deck option involves the use of full-depth precast concrete deck panels in combination with rapid bridge construction methods. The precast panels can be custom designed and fabricated for each bridge location to match the required bridge width, skew, and structural deck thickness. Longitudinal post-tensioning is typically used to provide a net compressive stress along the transverse joints between the precast panels.

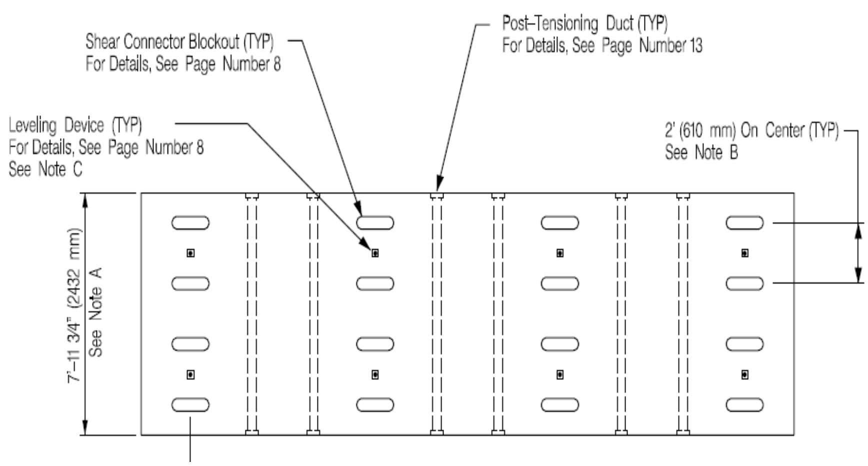



**Figure 3 – Precast Deck Panel Plan**

In special circumstances, such as a bridge with limited expected service life, the post-tensioning can be replaced with welded or mechanical shear connections for faster installation. Pre-tensioned panels can also be used to reduce the thickness and weight of the panels, as well as improve long-term durability. Specifying pre-tensioned panels can add cost to the project, and may restrict local precast plants, since PCI certification would be required when using pre-tensioned panels.

The primary advantage of this deck system is its speed of construction. Since the major components are prefabricated off-site, the panels can be installed much more rapidly than a conventional CIP deck. The primary disadvantage of this system is its higher cost compared to a CIP deck. Because of this cost differential, this system is best suited for locations where the higher cost is warranted to reduce user impacts, or where significant cost savings can be realized through the elimination of temporary bridges and/or detour roadways.

**Precast Exodermic™ Deck Panels**

This deck option uses precast Exodermic™ deck panels, in combination with rapid bridge construction methods, and is comparable to the full-depth precast concrete deck panel option discussed previously.

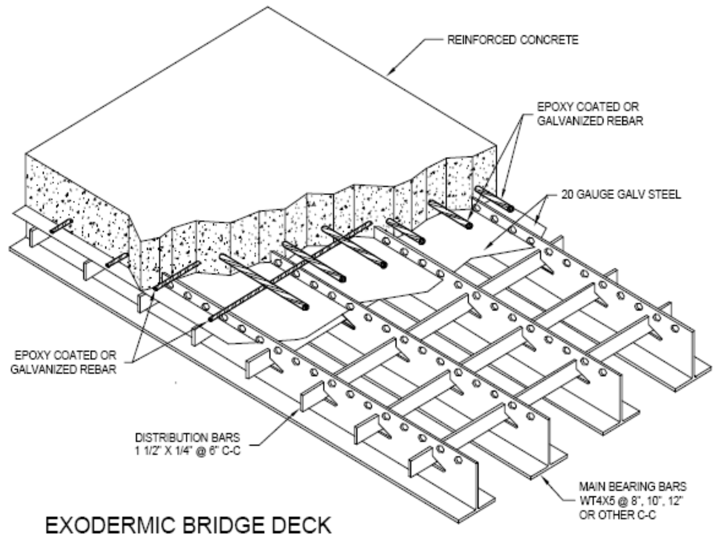



**Figure 4 – Exodermic™ Bridge Deck Detail**

The Exodermic™ bridge deck is typically comprised of a 4” deep steel grid composite with a 4-1/2" reinforced concrete slab. A portion of the steel grid extends one inch into the bottom of the concrete slab for horizontal shear connection. This results in a deck with a total structural thickness of 7-1/2”. The panels are custom designed and fabricated to match the required bridge deck width. Full-depth grouted shear keys are used between adjacent panels for both load distribution and improved deck durability.

The primary advantages of this deck system are its light weight and speed of construction. This system can provide a deck that is approximately 40% lighter than a typical cast-in-place deck. The lighter deck is particularly advantageous for bridges with lower live load ratings, and can often eliminate the need for structural strengthening of the bridge. Because the deck system does not use longitudinal post-tensioning, the installation times are even faster than with the full-depth precast concrete deck panel option.

The primary disadvantages of this system are higher cost and decreased long-term durability. The cost of an Exodermic™ bridge deck is similar to full-depth precast concrete deck panels, however, it does utilize a proprietary system with a limited number of licensed vendors. Because longitudinal post-tensioning is not utilized, the transverse joints between panels can, over time, be subjected to net tensile stresses due to concrete shrinkage. This tensile stress can cause the joints to open and allow salt laden moisture to pass through the deck. This system can be a good choice for the rapid replacement of decks for bridges with insufficient load carrying capacity, or where only a limited service life is required.

**Prefabricated Bridge Units (Inverset™)**

This bridge system type was first introduced and patented under the brand name Inverset™, although the patent has since expired and the system is no longer proprietary. This report will refer to this system as the Inverset™ type, due to its wide name recognition.

The system utilizes multiple prefabricated bridge units, which when connected together, comprise a nearly complete bridge superstructure. Each bridge unit consists of a pair of steel stringers (rolled shapes or plate girders) stiffened by steel channel diaphragms, and acting composite with a concrete deck slab. The unit widths vary depending on the individual bridge requirements and shipping limitations, but generally range between 8 feet and 12 feet.

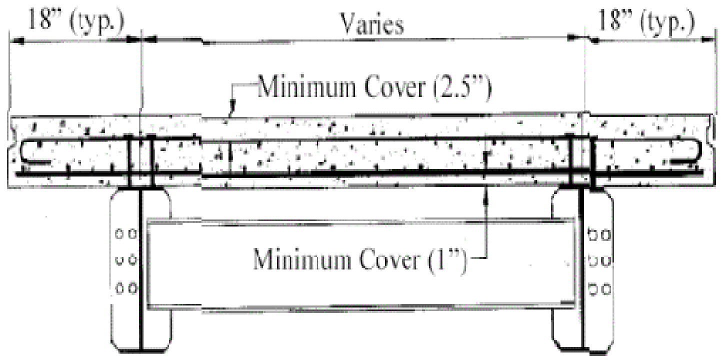



**Figure 5 – Typical Inverset™ Unit**

The units can be designed for single spans up to approximately 120 feet in length, although shipping weights start to become problematic for span lengths exceeding approximately 80 feet. A longitudinal, grouted shear key is used between adjacent units to create a water-tight joint and to assist with load transfer. The system does not use transverse post-tensioning, and instead relies on short (18") overhangs and stiff diaphragm action to minimize differential deflections at the joint lines.

One of the unique features of this system is the inverted deck casting method that is used. The fabricated girder pairs are assembled and then inverted in a casting yard, where the deck is then poured using formwork hung below the girders. This system allows the deck and portions of the steel stringers to be pre-compressed, which greatly improves long-term durability and also allows shallower stringers to be utilized.

The primary advantages of this system are speed of construction, deck durability, and the possibility of increased vertical clearance below the bridge. For this bridge, the minimum vertical clearance could be increased by approximately 4 to 6 inches through the use of the Inverset™ type system.

The primary disadvantages of this system are its relatively high cost, heavy weights (50-ton + units), and lack of transverse post-tensioning between the units. This option is best suited for bridge locations where both the steel stringers and the deck slab require replacement, or where the higher cost of the system can be justified by improved serviceability (e.g. vertical clearance), and reduced user impacts (e.g. eliminate repainting). With proper planning and scheduling, it appears possible to replace one half of the bridge superstructure (i.e. entire interstate barrel) over one weekend using an Inverset™ type system. See Drawing No. 4 – Prefabricated Bridge Units (Inverset™) for a typical bridge section illustrating this system.

**Self-Propelled Modular Transporters (SPMTs)**

SPMTs are computer-controlled platform vehicles that can move bridge systems weighing up to several thousand tons with precision to within a fraction of an inch. These units allow the bridge superstructure to be prefabricated offsite under controlled conditions, and then rapidly installed in a few minutes to a few hours with minimal impact to traffic. The FHWA recommends SPMTs be considered for all bridge projects in locations with high traffic volumes, such as those with 40,000 or more vehicles per day either on the bridge or on the roadway below the bridge


**Figure 6 – FHWA SPMT Manual**

The disadvantage of this system is its high cost. Although it is difficult to estimate the precise cost of using SPMTs on this project, it would certainly add several hundred thousand dollars to the overall project cost. Two recently completed, similar projects incurred SPMT costs of $570,000 and $800,000, respectively.

It is difficult to justify the use of this system on typical overpass structures unless delay-related user costs are considered. When delay-related user costs are included in the cost analyses for moderate and high traffic volume locations, SPMTs can result in the best-value solution. Unfortunately, the best-value approach assumes unlimited funding sources are available, which often does not match the reality of transportation funding programs.

## TRAFFIC CONTROL EVALUATION

MJ staff met with the Department on several occasions as part of the initial scoping process. Through those meetings and subsequent discussions with Department staff, the following three traffic control alternatives were selected for formal evaluation as part of this bridge rehabilitation study:

Alternative A: 2-Phase Construction with 3-Lanes using Night or Weekend Lane Closures

Alternative B: 3-Phase Construction with 4-Lanes using Temporary Deck Widening

Alternative C: 2-Phase Construction with 4-Lanes using Temporary Bridge and Detour Roadway

A description of each traffic control alternative and a general discussion of the construction sequencing are provided below.

## Alternative A

Traffic Control Alternative A proposes replacing the bridge deck in 2 phases using rapid construction methods. Each phase would place traffic on one of the two bridge barrels while the other barrel's deck is replaced. Several deck types are possible under this Alternative, including full-depth precast concrete deck panels (Alternative A1), Exodermic™ panels, and the Inverset™ type prefabricated superstructure system (Alternative A2).

Rapid construction methods allow one of the bridge decks to be replaced over one weekend, or over a series of nightly closures. For the weekend closure, the contractor would be allowed to begin construction on Friday evening at 6:00 p.m. and would have until 6:00 a.m. the following Monday morning to complete the work. The construction weekend would give the contractor up to 60 hours to complete each bridge barrel. Incentive and disincentive clauses or other innovative procurement methods (e.g. A+B bidding, Lane Rental, etc.) should strongly be considered to encourage timely completion of the work. For additional information on the Accelerated Bridge Construction Option, reference Appendix F.

A construction time estimate was prepared for the full-depth precast concrete deck panel option. This time estimate was prepared with the aid of contractor work rates provided by the Department, to evaluate whether a maximum 60-hour weekend lane closure would be adequate to perform the work required.

The results of the construction time estimate indicate it appears feasible to perform this work over an extended weekend closure, without impacting the Monday morning peak traffic period. A more detailed CPM construction schedule will be developed during the final design phase to assist with the development of the contract specifications. We also recommended conducting a partnering session during the preliminary design phase with the local chapter of the Association of General Contractors to discuss the proposed construction timeline.

**Table 3 – Sample Weekend Closure Timeline (Accelerated Bridge Construction Method)**

| Construction Work Task | Hours | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 2 | 4 | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 | 52 | 54 | 56 | 58 | 60 |
| Shift Traffic | 2 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Remove Deck | | | | | | | | | 16 | | | | | | | | | | | | | | | | | | | | | |
| Place Precast Panels | | | | | | | | | | | | | | | 12 | | | | | | | | | | | | | | | |
| Install Post-Tens. Strands | | | | | | | | | | | | | | | | 2 | | | | | | | | | | | | | | |
| Grout Transverse Joints | | | | | | | | | | | | | | | | | 2 | | | | | | | | | | | | | |
| Grout Cure Wait Period | | | | | | | | | | | | | | | | | | | 4 | | | | | | | | | | | |
| Post-Tension Strands | | | | | | | | | | | | | | | | | | | | 2 | | | | | | | | | | |
| Grout Post-Tens. Ducts | | | | | | | | | | | | | | | | | | | | | | 4 | | | | | | | | |
| Pour Deck End Haunch | | | | | | | | | | | | | | | | | | | | | | | 2 | | | | | | | |
| Concrete Cure Wait Period | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 12 | |
| Place Shear Studs | | | | | | | | | | | | | | | | | | | | | | | 6 | | | | | | | |
| Grout Stud Pockets | | | | | | | | | | | | | | | | | | | | | | | | 2 | | | | | | |
| Grout Cure Wait Period | | | | | | | | | | | | | | | | | | | | | | | | | | 4 | | | | |
| Shift Traffic | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | 2 |

It is not possible to accommodate the existing four lanes of I-93 traffic on one of the barrels. Only three lanes can be accommodated during the construction weekends. During these weekends, two lanes would be provided for the barrel with the higher traffic volume and one lane for the opposing barrel. The one lane of traffic provided would only be 12-feet wide. This would not allow wide loads to use I-93 during the weekends. Oversize Vehicle Permits should be eliminated during the weekends. Oversize vehicles could use the ramps as a detour to get past the bridge during the construction weekend. The speed limit on I-93 through the construction zone should be reduced to 45 mph. See Drawing No. 5 for typical sections of the two phases of Alternative A.

The ideal time for the construction weekends would be two consecutive weekends during off season traffic. I-93 experiences peak seasonal flows during the summer for those heading for the Lakes Region, in the fall for leaf season, and in the winter for skiing in the White Mountains. There is no weekend from Memorial Day to Labor Day that would be appropriate due to the volume of traffic. The choice of weekends is also weather dependent as inclement and cold weather could cause construction delays.

Two potential timeframes have been indentified based on a review of traffic data on I-93. The preferred time would be in early to mid May. The average temperature in Concord in May is around 55° F with average low temperatures around 42° F. Rain is a possibility, with average rainfall about 4" during the month, but this is a similar average with other months under consideration. There are no major events in May prior to Memorial Day (May 25, 2009) that would be affected by the reduced lane capacity on I-93.

The other possible timeframe would be in late September to early October. The average temperature in Concord in September is around 60° F with average low temperatures around 48° F and around 49° F with average low temperatures around 37° F in October. The selected weekends would have to be after the NASCAR Race at NH Motor Speedway, which is scheduled for September 13, 2009. There are concerns with this timeframe because the Deerfield Fair is scheduled for September 26-27, 2009 and leaf season typically begins in early October around Columbus Day (October 11-12, 2009). Impacts to traffic would be greater during this period.

Based on the available traffic data, two lanes would be provided northbound and one lane southbound when the construction would begin on Friday afternoon. The southbound lane drop would occur at Exit 15 where the outside lane would be directed to the eastbound I-393 exit ramp. Early Sunday morning the lane configuration would be shifted to provide two lanes southbound and one lane northbound. The northbound lane drop would occur as the outside lane would be directed to use the northbound Exit 14 exit ramp.

The peak demand periods during these weekends would still exceed the capacity that can be provided under Traffic Control Alternative A. It would be necessary to manage the demand during these construction weekends to ensure the roadway network continues to operate. Through public outreach efforts discussed below, it should be possible to reduce demand to provide acceptable levels of service along I-93 and surrounding roadways.

The consensus is that maintaining traffic flow on I-93 is important during these weekends. One option to maintain an acceptable Level of Service (LOS) on I-93 during peak traffic times during the weekend would be to close ramps on the Interstate and divert traffic onto local roadways. It is possible to manage the peaks on the local roads more effectively than on I-93. During peak hours of traffic on Saturday, the I-93 southbound on-ramp at Exit 15 could be closed to reduce the traffic through the one-lane work zone heading southbound. This would provide an acceptable LOS on the Interstate. Traffic from North Concord wanting to travel south on I-93 would use North Main Street and Loudon Road to access the southbound on-ramp at Exit 14. The intersection at North Main Street and Loudon Road would need to be temporarily modified to provide two left turns lanes on North Main Street southbound to accommodate the increase in traffic at this intersection. Uniformed officers would also be used during these peak periods to manage the traffic.

During peak hours of traffic on Sunday, the I-93 northbound to I-393 eastbound off-ramp could be closed to reduce traffic through the one-lane work zone heading northbound. This would provide an acceptable LOS on the Interstate. Traffic wanting to access I-393 eastbound from I-93 northbound would be diverted to the Exit 14 northbound off-ramp and use Fort Eddy Road to access I-393 eastbound. With modifications to the signal timings along Fort Eddy Road and at Loudon Road, Fort Eddy Road would function at an acceptable LOS. Uniformed officers would be used at the key intersections during these peak periods to manage the traffic.

A matrix is included in Appendix G that details the expected queuing and delay on the Interstate with the various options discussed above. Queue lengths and delay were calculated using QuickZone software and were based on traffic demands as shown on the figures included in Appendix G. The existing traffic volumes were determined using the permanent counter located south of Exit 13 and factoring the counts using ratios from the Central NH Region Model. The traffic volumes for the Loudon Road closure were determined from the most likely detour route a vehicle would take during the closure. Synchro software was used to determine the LOS for the signalized intersections along Fort Eddy Road and at the intersection of North Main Street and Loudon Road.

Exit 14 operations would also be impacted during the construction weekends. Loudon Road under the bridge must be closed for the entire construction weekend period to allow the contractor complete access to the bridge. Based upon the construction requirements to replace the deck on the southbound barrel, the southbound exit ramp would need to be closed during that weekend. This closure is required to give the contractor adequate space to locate and operate a crane for the redecking operation. The other ramps could remain open with traffic control as depicted on the plan views on Drawing No.'s 6 & 7.

Loudon Road is a key arterial in the City of Concord and closing it would require close coordination with city and state officials. The bridge over the Merrimack River would remain open as well as access to Fort Eddy Road. Traffic that normally uses Loudon Road to cross I-93 would be directed to use Exit 13 and Manchester Street or Exit 15 and I-393. Because Loudon Road is used by emergency responders to cross I-93 and the Merrimack River, provisions may be required to provide temporary fire personnel on the west side of the river. The Heights Fire Station is located on Loudon Road about 1 mile from the project site with an engine and ambulance permanently assigned there. The main concern is the only ladder truck owned by the Concord Fire Department is located on North State Street. The ladder truck would have to use I-393 and Exit 1 at Fort Eddy Road to access a fire on the east side of the river. However, there are few buildings on the east side that would require the ladder truck.

Loudon Road also provides the only access across I-93 for pedestrians and bicyclists. During the construction weekends or nights, no pedestrians or bicyclists would be allowed the use Loudon Road under the bridge. The closest crossings are Delta Drive approximately 1 mile north and Manchester Street approximately 1 mile south. Temporary shuttles could be provided to mitigate this impact.

An extensive public outreach and notification program would be required to successfully use rapid construction methods. The program would utilize a wide array of communication techniques leading up to the construction weekends. These techniques would begin several months prior to construction to ensure all potential users are aware of the reduced capacity of I-93 and well as the closures at Exit 14.

The techniques that would be utilized include changeable message signs placed along I-93 north and south of Concord, notices in local and regional newspapers, notices on Concord and other regional cable access channels, postings on state and local websites, and broadcasts on radio and television stations. These techniques have been used successfully in other parts of the country on roadways with higher traffic volumes. They are also successfully used twice a year on this corridor during the NASCAR races.

Traffic control during the weekends would also be critical to the success of this Alternative. Changeable message signs would be placed at key approaches to Concord to inform motorists of the closures and provide alternate routes. A significant law enforcement and NHDOT presence should be used to make sure any issues are quickly evaluated and resolved. The law enforcement and NHDOT presence is one reason the traffic control used during the annual NASCAR events is so successful.

Temporary lane closures during off-peak travel times would also be required leading up to and following the construction closures to prepare for and clean up after the rapid construction operations. Portions of the I-93 median would need to be paved temporarily to accommodate the crossovers needed for the detours. The temporary modifications would be restored once the bridge decks are replaced.

In order for the Rapid Redecking Method to be successful, the contractor needs to have access to the site and sufficient laydown area to store the precast panels. The use of the former NHDOT facility on Stickney Avenue would appear to be an ideal site for the contractor to use. Trucks carrying the panels could wait there until the panels are needed.

The primary advantages of Alternative A are that it provides the shortest construction duration and the lowest cost compared to the other alternatives studied. The estimated construction duration for this alternative is three months. The deck replacement, including preparation and cleanup, could likely be completed in less than two months, with the final month consisting of abutment patching and/or steel cleaning and painting. Although the rapid construction methods do add cost to the project, they are more than offset by the elimination of temporary bridge and detour roadway costs.

The primary disadvantage of this alterative is the impact to traffic during the night/weekend lane closures. Although the public outreach and notification program will help to mitigate some of this impact, the level of service of I-93 and Loudon Road will be lower during these short-term lane closure periods.

A hybrid of this alternative is to perform the work during several consecutive night shifts (6 p.m. to 6 a.m.) while maintaining four lanes of traffic during daytime peak traffic hours. This would lessen the impact to peak traffic on the weekend but would likely take up to a week to complete the replacement of each deck barrel. There are also constructability and durability issues with this approach that would need to considered. The deck demolition would need to occur in smaller segments, post-tensioning would be problematic, and the Inverset™ type system could not be used with this weeknight hybrid option.

## Alternative B

Traffic Control Alternatives B1 and B2 propose replacing the bridge deck in 3 phases using conventional cast-in-place concrete decks. Each phase would accommodate four lanes of traffic while roughly one third of the bridge deck is replaced. Alternative B1 represents temporary bridge widening and Alternative B2 represents permanent bridge widening.

In order to accommodate four lanes of traffic during each of the three traffic phases, the bridge must be temporarily widened by approximately 7 feet. This widening is required in order to provide four 12-foot wide traffic lanes, a 2-foot shoulder on each side of traffic, and temporary concrete barriers. One barrier is required between the two barrels and one is required to protect the construction zone. The temporary widening would reduce the vertical clearance under the bridge by approximately 3" during Phases 1 and 2, although this could be mitigated by using a shallower temporary stringer. The abutments, wingwalls, and roadway approaches to the bridge would also need to be temporarily widened. See Drawing No.'s 8 thru 13 for typical sections and plan views of the phases of Alternative B.

The existing median of the bridge and approaches would also require modifications because traffic will need to be accommodated in the median. During two phases of construction, traffic would be running over the median area, with a deck break point (low point) centered on one of the lanes. This is an undesirable condition, but there is no way to avoid it under this alternative.

Alternative B would have little reduction of capacity on I-93. The speed limit on I-93 through the construction zone should be reduced to 45 mph. A minor reduction in capacity would result from the reduced speed and reduced shoulder widths. Loudon Road would remain open with lane closures during off-peak periods only. The diamond interchange at Exit 14 would function as normal.

There is also a 3-phase option that would not require a temporary bridge widening. However, less than desirable lane/shoulder widths would be required. The available width for traffic over the bridge would be as little as 24 feet in one of the phases for each direction of travel. In the other two phases, 26 feet of width could be provided for each direction of travel. With this configuration the decks would likely have to be conventional cast-in-place concrete and temporary steel diaphragms would be required between the median fascia stringers. The less than desirable widths would be in place for approximately 2 months for each phase.

The primary advantage of this alternative is that temporary lane closures would be utilized along Loudon Road, compared to the complete weekend closure that is required under Alternative A. This alternative would likely require an eight month construction duration using normal work shifts. Although this is the same as duration as the temporary bridge and detour option (Alternative C), it is significantly longer than the rapid construction option (Alternative A).

The disadvantages of this alternative are numerous. The contractor would be required to work in the median with split traffic during at least three separate stages of construction. This median work presents site safety challenges due to potential conflicts with the construction vehicles and motorists traveling along I-93. In addition, the temporary bridge widening would add significant cost to the project. Finally, the roadway width would complicate the September NASCAR Race when both directions of traffic are carried on one barrel. Currently, two directions of traffic share 36-feet of width during the race weekend where they would be carried on 26-feet of width through Exit 14 under Alternative B. From a geotechnical perspective, the temporary bridge abutments and wingwalls would likely require either ground improvement or pile supports due to the compressible soils below the bridge site.

Also, there are significant structural risks with this alternative due to the deteriorated condition of the existing deck slab. While the multiple stages of partial deck demolition under this alternative may appear feasible on paper, the deteriorated deck slab will be subjected to heavy vibrations and decreased structural continuity. It is unknown whether the existing deck slab could accommodate this additional vibration and stress. In addition, the durability of the new concrete deck slab would undoubtedly be reduced due to the multiple construction joints and active traffic adjacent to the newly poured concrete. Lastly, the use of partial-depth precast SIP panels would be problematic for this alternative. Due to the need to match the existing 7” deck thickness during multiple construction stages, the deck would most likely utilize temporary conventional CIP construction with temporary formwork

The deck widening required for this alternative could be constructed as a permanent feature. Alternative B2 represents constructing permanent deck widening and would result in an outside shoulder width of seventeen feet in the northbound direction. The additional cost for Alternative B2 is approximately $300,000 compared to Alternative B1.

## Alternative C

Traffic Control Alternative C proposes replacing the bridge deck in 2 phases using a temporary bridge and detour roadway. The most economical deck type for this alternative would be cast-in-place concrete with mandatory use of the partial-depth, precast stay-in-place (SIP) forms. To replace the northbound deck system, the northbound traffic would be carried on the temporary detour and bridge while southbound traffic would be carried on the existing corridor. The temporary bridge would have two 12-foot lanes, a 2-foot inside shoulder, and an 8-foot outside shoulder. The wide outside shoulder is a result of the standard width of the most commonly available panel bridges for this application (Acrow Interstate Bridge). See Drawing No. 14 for typical sections of the two phases of Alternative C.

The temporary detour and bridge would be located east of the existing Interstate and would carry northbound traffic. The detour would impact the existing overhead sign structure south of the bridge. The northbound exit ramp could remain open with some potential temporary modifications. The northbound entrance ramp could remain open, however, the embankment for the detour would need to be supported by a temporary earth support system or retaining wall. See Drawing No.'s 15 and 16 for plan views of Alternative C.

Alternative C would have little reduction of capacity on I-93. The speed limit on I-93 through the construction zone would be reduced to 45 mph. A minor reduction in capacity would result from the reduced speed and reduced shoulder widths. Loudon Road would remain open with lane closures during off-peak periods only. The diamond interchange at Exit 14 would function as normal, provided the northbound entrance ramp could remain open.

The temporary bridge would impact the existing signal mast arm that controls eastbound Loudon Road to northbound I-93 traffic. The mast arm could be temporarily removed and the signal heads mounted on the temporary bridge. The northbound entrance ramp may need to be modified to ensure adequate sight distance and turning radii for vehicles making a left turn.

It is important to note that the areas impacted by the temporary bridge and detour must be restored to their existing configuration. The temporary bridge, abutments, pavement, embankments, retaining walls, and other features must be removed and the area restored. The signal mast arms, drainage structures, landscaping, and guardrails must also be replaced or restored.

The primary advantage of this alternative is that it provides the least impact to traffic. Four lanes of traffic would be provided along I-93, and as with Alternative B, only temporary lane closures would be required along Loudon Road below the bridge. In addition, the work can occur during the regular construction season using single work shifts.

The primary disadvantages of this alternative are the high cost and length of construction. The temporary bridge and detour roadway add significant cost to this option. It would also likely take an entire 8-month construction season to complete the deck replacement, including construction and removal of the temporary bridge and detour roadway, and restoration of the Exit 14 interchange. The issues during the NASCAR Race would be the same as described for Alternative B. Lastly, as with Alternative B, the temporary bridge abutments, wingwalls and possibly even the detour roadway would likely require either ground improvement or pile supports due to the compressible soils below the bridge site.

## CONSTRUCTION COSTS

Preliminary construction cost estimates were prepared for each alternative using the Department's weighted average bid prices, recent bid results from similarly sized projects, and engineering judgment. A contingency percentage was added to all cost subtotals to account for incidental work items.

The associated project costs are separated by roadway and bridge costs for each alternative. Roadway costs generally include roadway embankment, pavement, subbase, barrier, guardrail and traffic control costs such as temporary barriers, flaggers and uniformed officers. It is assumed that the existing box beam rail north and south of the bridge would be replaced with concrete median barrier within the limits disturbed. Bridge costs include structure removal, concrete, reinforcing steel, and other miscellaneous superstructure costs. A summary of the estimated construction costs for each alternative are provided below in Table 4. Preliminary, itemized construction cost estimates for each alternative are included in Appendix C.

**Table 4 – Construction Cost Summaries**

| ALTERNATIVE A1 | |
|---|---|
| Bridge Cost | $1.68M |
| Temp. Bridge Cost | $0 |
| Roadway Cost | $0.20M |
| Temp. Roadway Cost | $0.43M |
| **Total Cost =** | **$2.31M** |

| ALTERNATIVE A2 | |
|---|---|
| Bridge Cost | $2.66M |
| Temp. Bridge Cost | $0 |
| Roadway Cost | $0.20M |
| Temp. Roadway Cost | $0.43M |
| **Total Cost =** | **$3.29M** |

| ALTERNATIVE B1 | |
|---|---|
| Bridge Cost | $1.28M |
| Temp. Bridge Cost | $0.26M |
| Roadway Cost | $0.47M |
| Temp. Roadway Cost | $0.66M |
| **Total Cost =** | **$2.67M** |

| ALTERNATIVE B2 | |
|---|---|
| Bridge Cost | $1.90M |
| Temp. Bridge Cost | $0 |
| Roadway Cost | $1.07M |
| Temp. Roadway Cost | $0 |
| **Total Cost =** | **$2.97M** |

| ALTERNATIVE C | |
|---|---|
| Bridge Cost | $1.11M |
| Temp. Bridge Cost | $0.50M |
| Roadway Cost | $0.50M |
| Temp. Roadway Cost | $1.65M |
| **Total Cost =** | **$3.76M** |

We understand the NHDOT has budgeted approximately $11M for rehabilitation of the four Red List bridges within the project corridor. The current Ten Year Transportation Improvement Plan lists a construction budget for this bridge of approximately $2.4M. As the design of the bridge rehabilitation is progressed, we will work with the Department to reduce project costs wherever possible, to stay within the allocated budget for this project.

## CONCLUSIONS & RECOMMENDATIONS

Throughout this report, information has been provided on various deck replacement options and traffic control alternatives. Advantages and disadvantages have also been provided for each. The key considerations in determining the recommended alternative are Traffic Impacts and Construction Cost.

The alternative with the least impact to traffic results in the highest estimated construction cost. Conversely, the alternative with the lowest estimated construction cost results in significant impacts to traffic, especially over the weekend closure periods. The recommended alternative will likely be the one that best balances these two considerations. A Decision Matrix (Table 5) is provided on the following page to assist in selecting the preferred alternative.

Alternative A1- Accelerated Bridge Construction (full-depth precast concrete deck panel) is the recommended alternative. While this alternative has the most impact to traffic, this alternative also utilizes the shortest construction duration and has the lowest construction cost. The risk of specialized construction is out weighed by these advantages.

**Table 5 – Decision Matrix**

| CRITERIA | TRAFFIC CONTROL ALTERNATIVE | | |
|---|---|---|---|
| | ALTERNATIVE A | ALTERNATIVE B | ALTERNATIVE C |
| **Deck Type Options** | Full-Depth Precast Panels<br>Precast Exodermic™<br>Inverset™ | Cast-in-Place | Cast-in-Place |
| **Estimated Construction Duration** | **3 Months Total**<br>• 4 week off-peak preparation period.<br>• Followed by two weekend closures to replace decks.<br>• A final 6 week off-peak period to complete contract | **8 Months** | **8 Months** |
| **Lane Configuration** | • Three 12-foot Lanes (Two in peak flow direction)<br>• No Shoulders | • Four 12-foot Lanes (Two in each direction)<br>• 2-foot shoulders | • Four 12-foot Lanes (Two in each direction)<br>• 2-foot shoulders |
| **Traffic Impacts** | • Lane Reductions on I-93<br>• Loudon Road closed at bridge during both weekend closures<br>• Reduced Speed Limit<br>• Oversize vehicle restrictions during both weekend closures | • Three phases of detour configurations on I-93<br>• Reduced Speed Limit | • Two phases of detour configurations on I-93<br>• Reduced Speed Limit |
| **Estimated Construction Cost** | Bridge = $1.68M<br>Temp. Bridge = $0<br>Roadway = $0.20M<br>Temp. Roadway = $0.43M<br>**Total = $2.3M** | Bridge = $1.28M<br>Temp. Bridge = $0.26M<br>Roadway = $0.47M<br>Temp. Roadway = $0.66M<br>**Total = $2.7M** | Bridge = $1.28M<br>Temp. Bridge = $0.50M<br>Roadway = $0.50M<br>Temp. Roadway = $1.65M<br>**Total = $3.8M** |
| **Durability** | Unknown<br>The precast deck systems are relatively new to NH to determine long term durability | Moderate<br>Demolishing existing decks alongside newly poured decks raises concerns | Excellent |
| **Pros** | • Shortest construction duration<br>• Least impact to peak traffic<br>• Lowest Cost<br>• No conflicts with NASCAR | • Limited impact to traffic | • Limited impact to traffic<br>• Fewest changes in traffic pattern<br>• Least Risk |
| **Cons** | • Traffic queues and/or ramp closures during weekend closure peak hours<br>• Specialized construction<br>• Most Risk | • Lower deck durability<br>• Construction required in median between traffic<br>• Potential conflict with NASCAR Races | • Highest Cost<br>• Temporary Bridge<br>• Potential conflict with NASCAR Races |