Mr. James N. Carter, P.E.,
Chief Engineer Bridges and Structures
Norfolk Southern Corporation
1200 Peachtree Street, NE
Atlanta, GA 30309

RE: PN2662
NORFOLK SOUTHERN
Bridge S.R.-361.66 at Portageville, New York
Rehabilitation Feasibility Letter Report

Dear Mr. Carter:

The alternative that would repair and/or retrofit the existing bridge poses significant engineering concerns due to the integrity of the structure, and such concerns would largely remain even after a retrofit of the current bridge. These primary concerns are discussed individually below:

- Structural deficiencies
- Remaining fatigue life of the existing superstructure
- Feasibility of performing rehabilitation

**Structural Deficiencies**
A visual inspection of the structure was performed from August 18 through August 21, 2008, by Modjeski and Masters. The inspection was performed to determine the general condition of the Bridge and gather any information missing from or illegible in the available plans.

The inspection of the concrete-encased masonry piers resulted in the discovery of cracking, deterioration, and spalling in most of the existing concrete. The concrete piers were designed and constructed for the original timber bridge in 1852, when seismic loading, a requirement under current design codes, was not considered during design. There are no contract drawings of the piers; therefore, it is difficult to determine if the piers are reinforced; however based on the era of construction, they likely are not and, as such, are not well-suited to resist the loads produced during a seismic event.

Similarly to the substructure, the inspection of the superstructure documented numerous structural deficiencies, including missing and broken rivets, deformation in structural members, cracks in members, corrosion, section loss and a bearing not in full contact with the pier under self weight dead load.

A finite element analysis was conducted to rate the members for Cooper E80 Loading with reduced impact for a maximum speed of 10 MPH. This finite element model was created assuming no section loss to the members. The results of the analysis revealed that the lower portions of the tower legs and the majority of the deck truss members (as shown in Figures A1 and A2) were inadequate for Cooper E80 Loading at as-built condition. While the contract drawings do not state the original design live load, the rating results support historical data
suggesting that the bridge was originally designed for Cooper E40 Loading, half of today’s current design load.

**Remaining Fatigue Life of the Existing Superstructure**

The design of the existing superstructure in 1903 preceded the implementation of fatigue design requirement by AREMA standards (formerly American Railway Engineering Association [AREA]) in 1910. Thus, it is very likely that fatigue was not considered during the design of the existing structure. The bottom chord and diagonal eyebar counter-members of the deck truss spans are prone to fatigue since cyclic loading creates tensile stress on the members. A fatigue analysis was performed on these members and the fatigue stress range was compared to the allowable fatigue stress range as defined by AREMA. Eyebars are designated as Category E fatigue details with an allowable fatigue stress range of 8 ksi due to their susceptibility for stress concentrations around the pin hole. All of the diagonals and some of the bottom chords on the deck truss spans are eyebars with this fatigue-sensitive detail. The results of the fatigue analysis showed that all of the bottom chord members and diagonal members, except the two interior diagonals at the center panel, are stressed beyond the allowable fatigue stress range (See Figures A1 and A2), as would be expected for a bridge of this age. Similarly to the strength rating of the members, the fatigue analysis neglected section loss and deterioration and assumed that all members are in as-built condition. Though the bridge continues to be a suitable rail structure in the short term and is inspected frequently to ensure continued safety of operations, given the fact that the existing structure is over 100 years old, there is a high degree of uncertainty of the structure’s remaining fatigue life as a freight rail carrying structure, and indeed based on recent inspections, it is believed that the Bridge is now nearing the end of its life as a rail structure. Due to NS’s diligent inspection and maintenance the Bridge continues to be a suitable rail structure in the short-term; however, the frequency and expense of such inspections and maintenance are ever increasing in order to ensure continued safety of operations.

![Figure A1](image1.png)  
**Figure A1 – Elevation of Span 9 with Member Rating Less than E80 Noted (Span 7 Similar)**

![Figure A2](image2.png)  
**Figure A2 – Elevation of Span 11 with Member Rating Less than E80 Noted**
Feasibility of Performing Rehabilitation

As previously stated, the majority of the members in the deck truss spans (Spans 7, 9, and 11) are deficient for strength and/or fatigue. To accommodate the increased load capacities, each of these deficient members must be strengthened or replaced by a new member.

Retrofit and repair is complicated by the pin-connections of the truss members. In order to utilize the additional capacity, additional steel and replacement members must be adequately connected through the truss joints. The ideal method for conducting this type of rehabilitation is to remove the truss pins for installation of the new members and/or reinforcing plates; however, removal of the truss pins will require temporary support for the dead loads and restriction of live loads until the truss pins are replaced. Temporary support at the joint may be performed using a device, known as a "spider", that provides an alternate load path until the pin-connection is restored. Joint spidering is a labor-intensive process, resulting in high costs and lengthy bridge shutdowns during which trains must be rerouted.

In addition to difficulties encountered with temporary support, detailing at the pin connections will complicate the fit of new and/or strengthened members. Currently, eyebar members are stacked on the pins. New member and/or reinforcing plate sizes must be selected carefully to avoid interference with adjacent members. Since many of the new and/or retrofitted members are eyebars, the restrictive Category E allowable fatigue stress range will still apply. For these members, a significant amount of additional material will be necessary to reduce the fatigue stress, thus compounding the issue of interference with adjacent members. Working with the existing details may preclude the possibility of adding adequate additional steel to reduce the fatigue stress range to acceptable levels. Assuming that it is possible to add the material, the cumulative fatigue damage in the existing members cannot be completely remedied. Therefore, the best that can be expected is that the rate at which future damage is realized is reduced; however, the maintenance associated with the existing 100 plus year old components of the Bridge will remain.

Structural deficiencies on the towers are found in the lower portions of the tower legs. These towers legs may be able to be strengthened using reinforcing plates, although the installation of reinforcing plates may be complicated by the riveted composition of the member and plates installed during previous strengthening efforts.

To support higher load demands, the current condition of the masonry substructure must be thoroughly evaluated through an in-depth rating and examination of the piers. Tests must be performed on the piers to evaluate the degree of deterioration in order to determine if the piers can be reused. If the piers are found to be unsound, they will have to be repaired or replaced. Repair and/or replacement of the piers will be a difficult task since they are currently supporting the towers and superstructure.

While the condition of the paint system is not directly related to the load-carrying concerns mentioned above, the existing structure should be repainted to provide protection to the steel against further deterioration. Prior to paint removal, testing must be performed on the existing paint system to detect the presence of any hazardous materials. In the event that hazardous materials that may be disturbed by such painting activities are detected, the blast-cleaning process of the structure must be carefully controlled to minimize such materials from being released in uncontrolled or dangerous quantities. These containment requirements during paint removal, as well as disposal of hazardous waste from blast-cleaning (if required) and other associated tasks will be a significant expense in addition to the structural repairs previously described.
Conclusions and Recommendations

Based on the findings of the visual inspection and the results of the rating analysis, Modjeski and Masters recommends that the existing bridge be replaced rather than rehabilitated. This recommendation represents the most prudent engineering solution in our opinion. Rehabilitation of the bridge, while possible, is not practical and would not provide the number of years of continued service from the Bridge required by Norfolk Southern.

Very truly yours,

[Signature]

Kevin W. Johns, P.E.,
Senior Associate

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