Modjeski and Masters, Inc. Consulting Engineers 04/18/06
St. Croix River Bridge – 3D Analysis Report

Introduction

This memo presents a summary of a three dimensional (3D) analysis of the "Organic" concept for the proposed St. Croix River bridge project. The Organic concept has several attributes that are best handled by a 3D analysis. The superstructure consists of twin concrete box girders, connected by cross beams and having a sidewalk cantilevered from one of the box girders. Extradosed cables are used to help support the box girders which are in turn supported on concrete piers. Understanding of the torsion introduced by the cantilevered walkway, as well as the support of the box girders at the piers, profits from a 3D analysis.

The analysis summarized herein is of a preliminary level of study. The intent is to confirm the general dimensions of the primary structural components, rather than to produce a design ready for final engineering. There exist several complex issues that need to be considered in depth before proceeding into the design stage, and they are discussed herein.

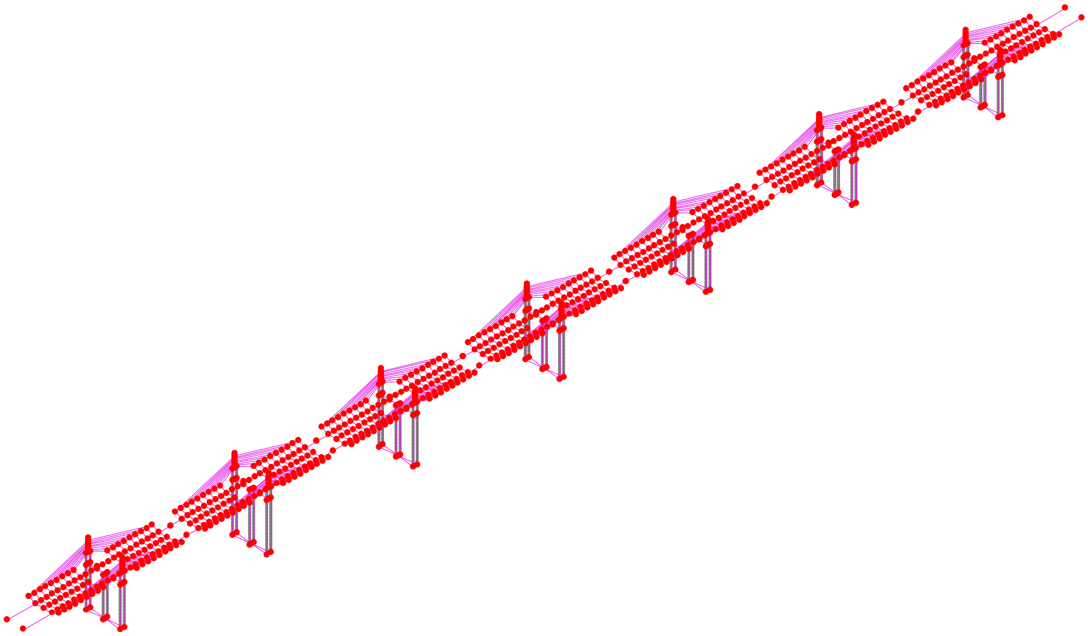

Figure 1. Finite Element model of river spans.

Modeling

A three dimensional finite element model of the river spans of the proposed St. Croix River bridge was developed based on the "Organic" concept. Drawings developed by TYLin were used for all preliminary member dimensions. All members were modeled with beam elements. The superstructure consists of twin concrete box girders, and these were modeled with beam elements. Cross beams connect the two box girders at cable anchorage locations. At the piers, a pier cross beam connects the box girders to the three pier legs. Figures 1 and 2 show global and detailed views of the model. The spans are 470 feet, the cable spacing is 18' and the first cable is located 54' from the pier centerline. The longest cable has an angle of 15 degrees with the horizontal. The piers have three sets of twin legs, two at the edges and one set on the centerline of the bridge.

Rigid links were used to connect the cable deck anchorages to the box girder beam elements, and to connect the pier cross beam to the pier legs. All three pier legs are connected at the base to a single node with rigid links. The piers were modeled as beam elements extending to the pile cap tops, which were set at 15 feet below river elevation. In order to simplify the model, the box girders were assumed to be level and the pier lengths were varied to account for the roadway geometry and varying ground elevation.

Support springs were used to model the foundation stiffnesses at all pier locations. These springs were determined using estimated soil properties and a soil/structure lateral response computer program (COM624P). Group effects were ignored. The foundation group at each of the piers consists of eight (8) ten foot diameter drilled shafts with free lengths that vary from 30 feet to 130 feet.

The connection of the box girders to the pier cross beam and the pier legs is a very complex region, and is not easily modeled with beam elements. In order to improve the accuracy of the model, a detailed shell element model of the connection region was created. A view of this model is shown in Figure 3. The stiffness of the detailed model was determined, and the properties of the beam elements in the global model were modified to provide equivalent stiffness.

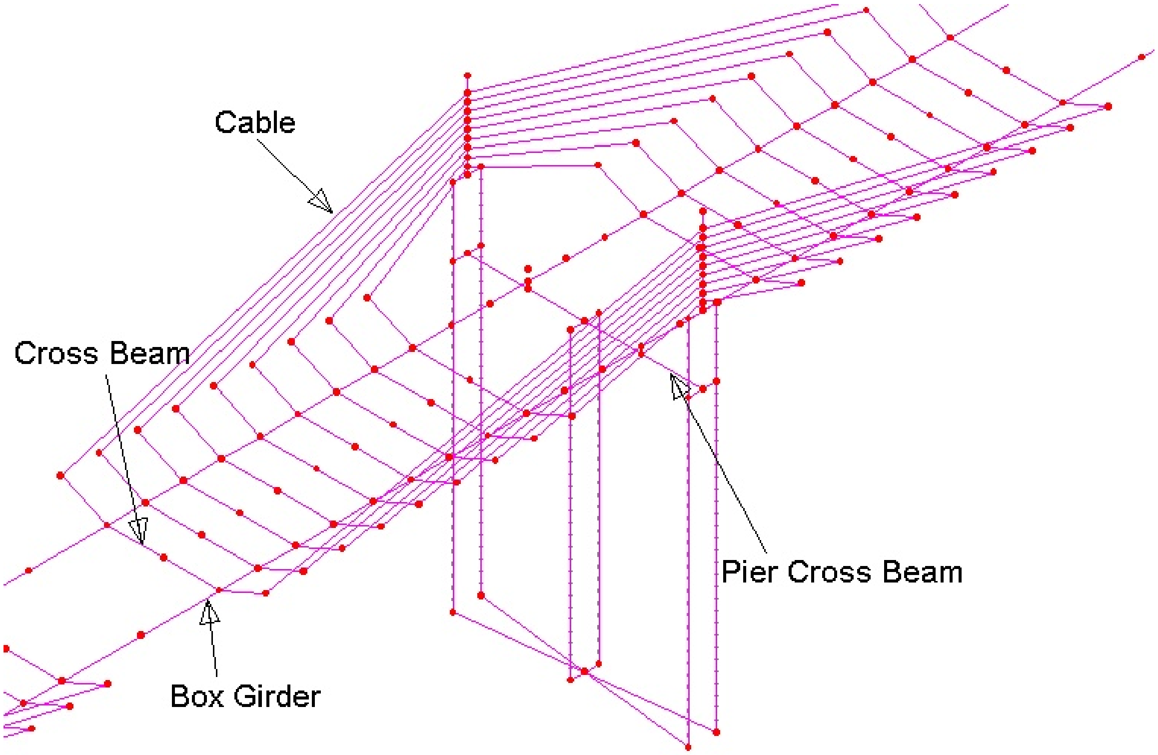

Figure 2. Model of one pier unit

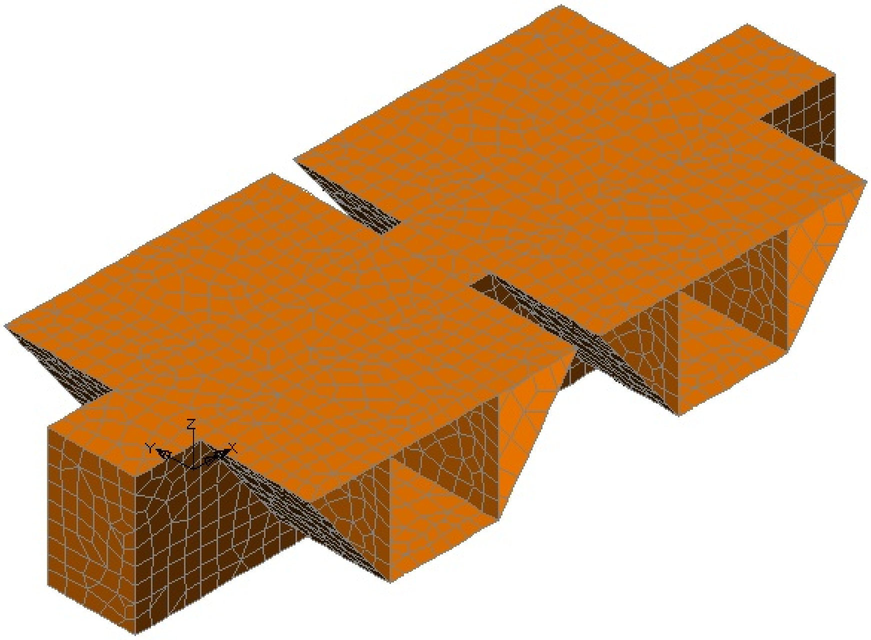

Figure 3. Detailed analysis model of Pier Cross Beam

Assumptions as to the cross-section dimensions were made in order to estimate the superstructure weight. These assumptions were based on work done by TYLin, and were discussed with them. The cable forces were then set to provide a total vertical force to the box girder equal to 30% of the total dead load. For simplicity, all cables were stressed to the same load, creating equal sized cables, however the vertical force supplied to the box by each cable will vary with the inclination angle of the cable. The typical method in cable stayed bridges is to set the vertical force constant and vary the cable force to suit. Equal cable forces result in higher moments in the box girder at the pier, due to the centroid of all cable vertical loads being located closer to the pier, but minimize the maximum cable size and associated anchorage requirements, and to some extent standardize the cable anchorage details. Decisions regarding how to assign cable forces can be finalized during the design stage.

Piers

Transverse

A significant out-of-plane load is created due to the transverse incline of the cables. This incline is due to the cables not being arranged in a vertical plane passing through the tower, but rather traveling from the tower inward to the anchorages at the edge of the deck. This out-of-plane load on the towers above the deck pulls the tower tops inward approximately 2” under dead load. Figure 4 shows this deflection magnified 30 times; the deflection is displayed in feet. The estimated shear in the tower from this load is 1300 kips. The current cross-sectional dimensions of the tower appear to be adequate for the load, but care should be taken during design to ensure long term creep deflections are accounted for. Changing the locations of the tower anchorages will not likely affect the displacement, as the magnitude of the induced moment is controlled by the relative transverse location of the tower centroid and the deck anchorages. Mitigation by the use of prestressing in the tower appears to be a viable mitigation method.

Longitudinal

The longitudinal behavior of the bridge is dominated by two effects: creep and shrinkage, and temperature changes. An approximate, long term construction analysis was conducted to evaluate the magnitude of the creep and shrinkage deformations and the induced loads on the towers. This analysis included the post-tensioning loads and their effect on concrete creep. The structure is currently envisioned as continuous over the river spans without expansion joints or bearings at the piers. All deformations from creep and shrinkage and from temperature changes will therefore be accommodated by flexure in the piers. The piers have intentionally been made flexible in the longitudinal direction to reduce the induced moments. However, the long-term effects of these displacements on the durability of the piers must be examined before implementing this type of structure. From the analysis, the long-term creep and shrinkage displacements are approximately 6” at the Wisconsin pier (pier 7). Depending on the design temperature, the movements due to temperature change in one direction range from 6” to 8.5”. A conservative reading of AASHTO results in an 80-deg F. temperature change. The displacement range listed corresponds to a change in temperature ranging from 60 to 80 deg. F.

Assuming fully cracked section properties, the induced moments and axial loads are within the capacity of the section. The piers will be bent in double curvature, with the maximum moments occurring at the connections to the pier cross beams and the pile caps. The region of the pier from the pile cap to just above the water surface thus poses a potential durability concern if crack widths become large. Assumptions as to construction duration and sequencing of closure pours were found to have only a minor effect on the total displacements imposed; however, the use of jacks to impose an initial displacement counter to those expected from creep and shrinkage may have a larger impact.

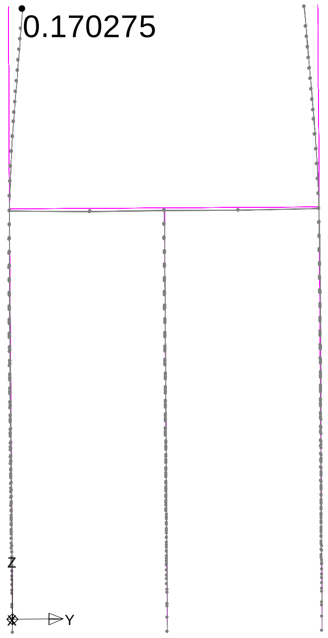

Figure 4. Displacement of the tower tops inward due to out-of-plane cable forces, magnified 30 times.

Box Girders

Bending

From the 3D analysis model, forces in the box girders due to dead, live, and wind loads were determined. The dead load used in design is the state immediately after span closure, before any redistribution from creep. This is the most critical case for bending over the piers, which determines the required structure depth and the bottom flange thickness. Several different configurations of live load were applied to approximate the most severe loading for moment and shear. In general, the live loading was very small compared to dead load and therefore small changes in the live load effects will not have an impact on the design. Figures 5 and 6 show the dead and live load moments (unfactored) in one box girder. The dead load moment includes the cable forces and dead load of the structure, but not the effects of prestressing.

The box girder with the sidewalk cantilevered from one side carries significantly higher loads than the box without the sidewalk. The higher loads are used to check the adequacy of the cross-sections, as it is assumed that both box girders will have the same cross-sections.

The structure was analyzed without any prestressing in the model. The required level of post-tensioning can then be calculated from the dead and live load moments. Initially, the box was assumed to be 20’ deep. With the current cross-section arrangement and dead load, it was found that an 8’ thick bottom flange was required to carry the compressive forces at the pier. This would be difficult to provide in a pre-cast section, and therefore alternatives were sought to reduce the section at the pier. Widening the bottom flange and deepening the girder resulted in a reduced bottom flange thickness. Based on a concrete strength of 6000 psi concrete, the following design values were determined for the section of one box girder at the pier:

Depth at pier - 22’
Width of Bot. Flange - 25’
Thickness of Bot.Flng.- 5’
PT force req’d - 45,000 k/box

These dimensions do not represent the only set that results in an acceptable solution. A shallower section with a thicker, and perhaps wider, bottom flange would also be acceptable. The compressive forces in the bottom flange are very large and require a wide, thick flange to carry the load. This is partly due to the use of twin boxes rather than a single, multi-cell cross section. Since the two boxes are connected at the top flange, if a bottom flange was added between them, a thinner bottom flange and shallower box, of only 20’ depth, could be used. Figure 7 shows the box girder cross section at the pier used in the analysis.

Shear

The web thicknesses appear to be adequate for shear, although local thickening at the piers may result in a more efficient design. In the cross-section analyzed, the interior webs were 2’ thick while the inclined exterior webs are 8” thick. This distribution of concrete towards the inner vertical webs results in a more efficient use of material.

Efficiency

It is instructive when developing the cross-sectional dimensions of a concrete box girder bridge to compare the resulting quantities of the concrete with approximate equations developed for estimating purposes. In "Prestressed Concrete Bridges" by Menn, several equations are presented for determining bridge quantities. For concrete, the volume can be approximated by:

$$V = L \times W \times (1.15 + 0.0045\, L)$$

where
L = length of span
W = width of deck

For one river span of one box girder, $L=470'$ and $W=49'$

$V=75{,}200\ ft^3$ which equates to a weight of 11,300 kips. From the analysis, the weight of one box girder in a span is approximately 17,000 kips. This represents a 50% increase in concrete over the predicted amount. Some of the difference can be attributed to the long span length, which may not have been accounted for in developing the equation. Also, the cantilevered sidewalk is adding load which requires a larger concrete section. However, a significant portion of the additional concrete arises from the need for substantial cross-beams linking the two box girders at each cable location. A reduction in the amount of concrete used in the cross-beams will improve the efficiency, and thus lower the cost, of the bridge. In the design stage, options such as reducing the depth and width of the cross-beams, or even reducing the number of cross-beams used should be investigated.

Figure 5. Live load moment diagrams for one box girder

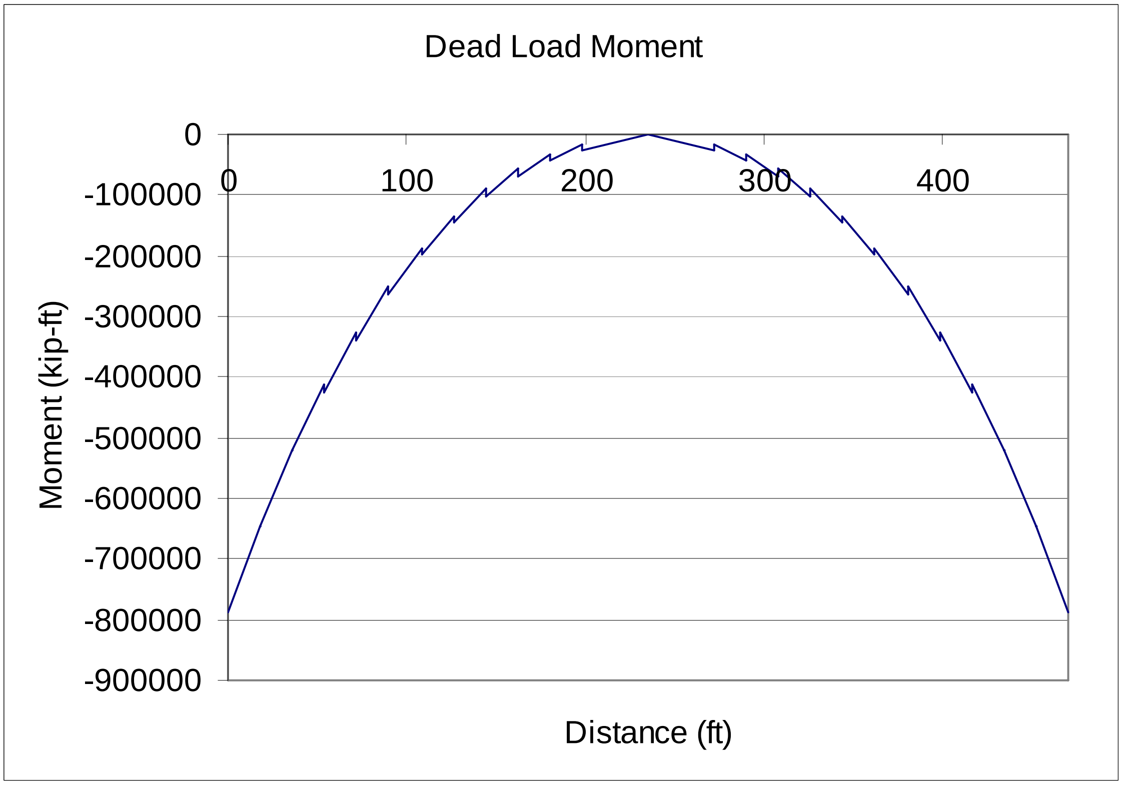

Figure 6. Dead load moment diagram for one box girder

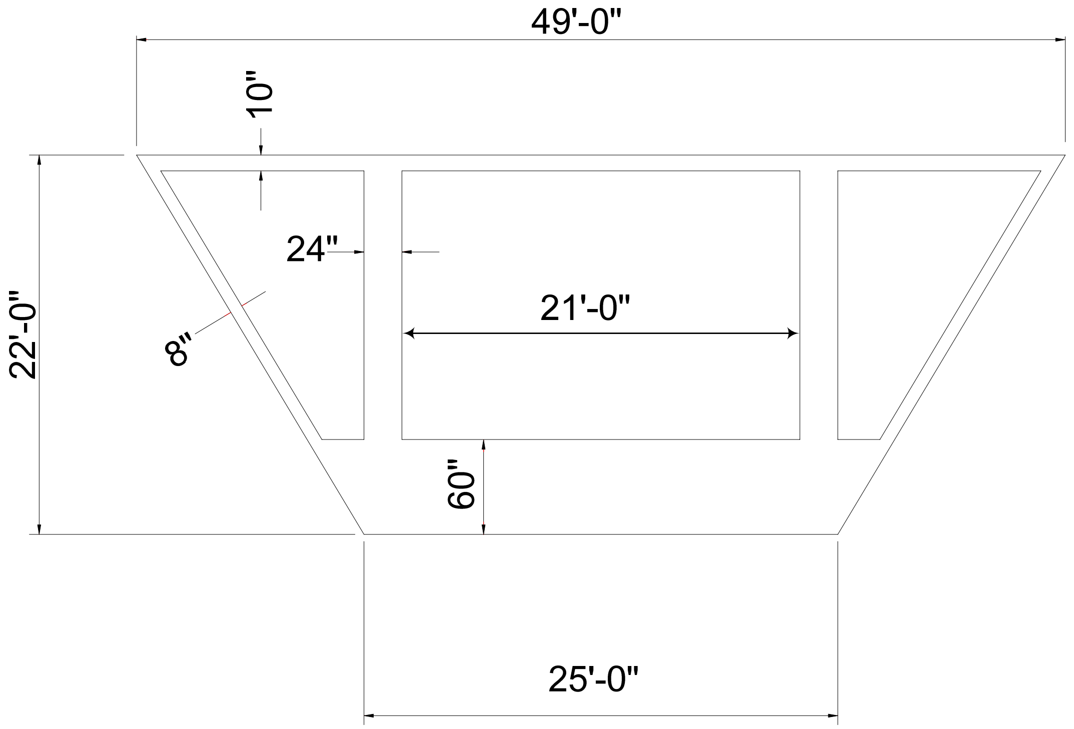

Figure 7. Cross section at pier

Conclusions

In general, the dimensions of the main structural members of the proposed St. Croix River bridge are adequate for the loads applied to them, however some increases in the dimensions of the box girder will probably be required to improve the efficiency of the structure. Several issues were identified that will need to be addressed during final design:

- Assignment of cable forces amongst cables to maximize economy. (This does not concern the maximum 30/70 split between cables and girder.)
- Transverse creep deflections of the towers, and possible use of prestressing to counteract them.
- Effect of temperature and creep and shrinkage induced cracking on long term durability of the piers.
- Improvements in the efficiency of the cross section and cross-beams. Significant savings in materials and construction costs may be realizable.