DURABLE, JOINTLESS, TWO SPAN, BULB-T OVERPASS

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ABSTRACT

Deficiencies of the previously existing bridge identified two critical design requirements of the replacement structure: to make the overall bridge corrosion resistant to counter effects of intense de-icing salts use; and to increase both vertical and horizontal clearances under the overpass. Failed bridge seals in conjunction with chloride contaminated run-off had severely deteriorated the previous structure's rolled steel beams, the bearings, the bridge seats, and the exposed sides of the substructure. Prestressed concrete beams are not only a more corrosion resistant material, their range of concrete thermal movements is less than steel, facilitating design and details for a jointless bridge.

Keywords: Connections, Creative/Innovative Structures, Sustainability

INTRODUCTION

Plant fabricated, high quality prestressed concrete beams were selected as the most viable and corrosion resistant superstructure elements for carrying Hammond Street over Interstate 95 in Bangor, Maine. The previously existing structure exhibited deteriorated steel beams and bearings, as well as deteriorated concrete bridge seats, pier caps, and other surfaces. The use of a jointless superstructure system could eliminate seal failures such as the one that resulted in the accelerated deterioration shown below.

Underneath Previous Bridge w/ Central Pier in Foreground

Another deficiency of the previously existing structure was the limited horizontal and vertical clearance for Interstate 95, the roadway underneath. The previous structure had four spans, with very short approach spans, spill through abutments, and very shallow steel beams. The Maine Department of Transportation was able to make room for a future additional southbound lane while simultaneously increasing vertical underclearance by two feet.by: raising the bridge profile no more than 1.6 feet, eliminating the approach spans; building tall, wall-type abutments; and minimizing the new superstructure depth.

HOW BAD COULD IT BE?

The set of functional demands on most bridges vary so much from one another that cookie cutter designs are over-priced, functionally insufficient, or both. At least, that's what the good ol' designers who trained me claimed. This section presents some of the physical characteristics and design assumptions of the replacement bridge. The following section provides some rationale for principle assumptions and details, as well as explanations of how they impacted construction and will satisfy functionality and durability needs.

The deck design consists of: an eight inch thick reinforced concrete structural slab with a normal 2% crown. Likely camber dimensions and the width of the Bulb-T top flanges were addressed by plan notes and haunch details in order to assure a minimum blocking depth of 0.5 inch. The deck is covered with a flame-applied "high performance" bituminous membrane and paved with three inches of hot bituminous mix.

There are nine beam lines; the top flange width for the New England Bulb-T 1000's is 47.5 inches. So, the clear span between the top flanges is less than 40 inches. All of the Bulb-T's, both the 87 foot and the 80 foot spans, both interior and exterior girders, have the same strand arrangement. There are only two strands in the top flange, located nearest the flange edges. The two bottom rows of the bottom bulb are filled with strands, but there are no strands in the third row. Only the higher pair, of the two strand pairs in the web, is draped. All strands, thirty per beam including the top pair, are fully tensioned full length.

Next, consider the geometrics. In preliminary design, Traffic Engineering requested that the bridge geometry allow for the addition of a lane on the southbound side. This necessitated the change from four spans, with short approach spans and spill-through abutments, to a two span superstructure supported by wall-type abutments. The new Span #1 on the westerly side is 87 feet long, while Span #2 is 80 feet long. This future lane addition will better address traffic merging from the on-ramp immediately north of the bridge, show with the underclearance warning sign below.

The need to increase vertical underclearance, particularly over the west edge of the southbound side, made the design more challenging. The target minimum underclearance was 16.5 feet; while underclearance for the previous structure was approximately 14.5 feet. This presented a severe restriction on the superstructure depth, particularly the beam depth. Shaving every inch possible from the superstructure depth, the new beam depth is 39.5 inches (1.00m). By this stringent limitation of the beam depth and a limited 1.6 foot grade increase of the bridge profile over the West abutment, this two-span superstructure actually provides nearly two feet additional underclearance compared to the four-span steel superstructure it replaces.

Looking Southerly at West Half of the Previously Existing Bridge

The geometrics of the Hammond Street Bridge, old and new, make it susceptible to salt water spray from the interstate. The fascia to fascia width of the new bridge is 64 feet, the same as the previous bridge. Given that the transverse width of the deck above is basically four times the increased but still limited vertical underclearance, the space under the bridge is obviously ‘tunnel-like’. Given the heavy use of de-icing salts, and the frequency of salt spray from traffic under the bridge during wet weather, corrosion resistant steel was rejected for use at this interstate overpass.[1]

To avoid exorbitant roadway costs, the new structure was built at a <u>23 degree</u> skew, matching the existing horizontal alignments. The heavy skew presented considerable difficulty in the forming of the cast-in-place beam splices and end diaphragms, that also required block-outs for utilities.. Forming the superstructure and substructure was also complicated with longitudinal construction joints since the bridge had to be constructed in stages to maintain traffic.

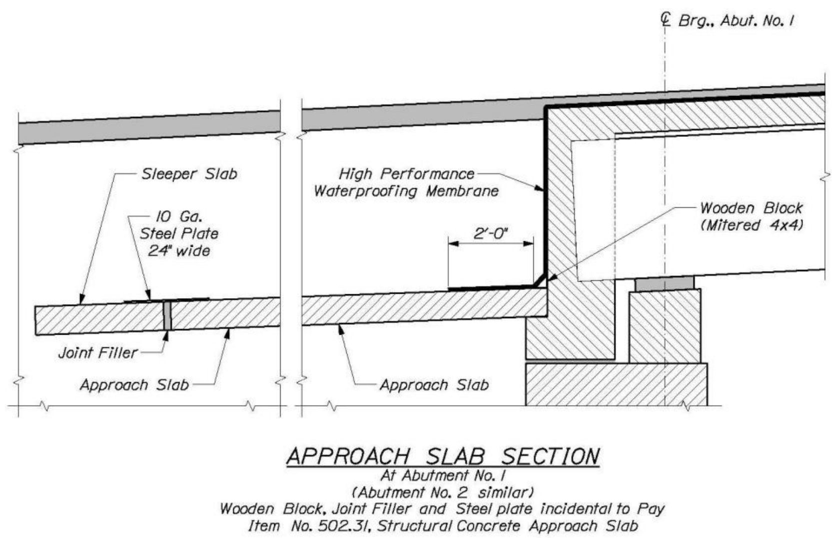
Looking Westerly with North Half of Previous Bridge Removed

One lane of traffic in both directions was maintained at all times on the bridge via the staged construction. Behind the abutments, maintenance of the two lanes required supporting live traffic on a split roadway excavation approximately 30 feet deep. The only exception to continuous maintenance of traffic on the bridge and on the interstate below was for strictly limited night hours for new beam erection. The Bulb-T's have been designed and erected as simply supported spans for all dead loads, and continuous for live loads. This feature made the 39.5 inch beam depth viable. All substructure units are cast-in-place concrete supported on spread footings.The two-span bridge is fixed at the median pier. Above the pier bearings cast-in-place "continuity collars" are used to splice the two spans together to resist negative live load moments at the pier.

Behind the expansion bearings of both abutments, the roadway is placed over an approach slab, up to "floating" end diaphragms or backwalls. An inch vertical gap is required between the bottom of the end diaphragms and the bridge seat. That gap and the bridge seat area are protected from the intrusion of the backfill by a plain neoprene sheet attached to the floating backwall and extended below the bridge seat. The bridge seat is level from fascia to fascia.

The beams sit on steel reinforced bearings on concrete pedestals. The pedestals are tall at the center line due to the cross-slope. However, the shortest pedestal height is only 0.56 feet. Easterly pedestals are finished flat, but westerly pedestals are graded to match the profile, increasing in height by 1 inch in 20 inches from back to front. The depth of the pedestal shown below may seem excessive, but it not only accommodates the cross-slope, but is a minimal fit for the utility block-outs in the end diaphragm and the approach slab seat.

The larger length of approach slab at each abutment is fastened to the backwall to move with thermal expansion/contraction. Provision for all thermal movements, of the superstructure and the utilities that "ride" on the tied slab, is made only at the joint between the tied approach slab and the sleeper slab. The full thermal expansion range for the longer span is only 0.5 inch, which makes concrete beams advantageous for jointless bridges.



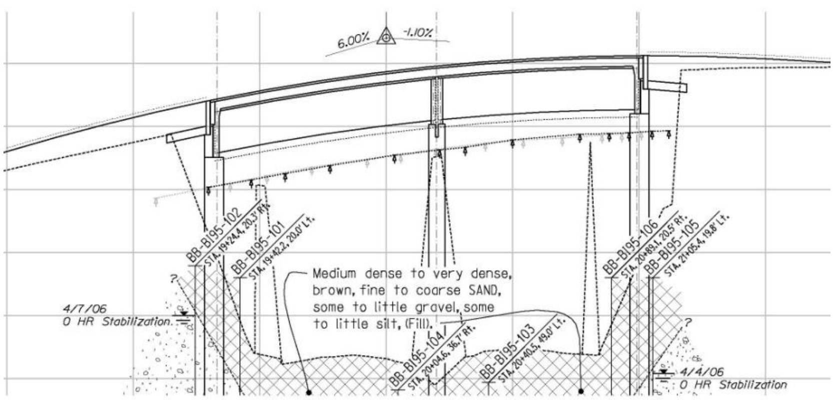
Section through Beam End, Abutment Bridgeseat, and Approach Slab

THE CONTRACTOR'S GLORY AND HEARTACHE

The unusual traffic pattern of the Hammond Street Bridge both helped and hurt the design process. The bridge is relatively short and provides the only opportunity for westbound traffic that has merged on to Hammond Street from the south to shift lanes to the north before encountering a northbound turn and as it is the only opportunity for eastbound traffic that has merged onto Hammond Street from the north to shift to the south curb before immediate turns and ramps. This bridge is on the southerly side of Bangor, with the Bangor International Airport to the northwest and the city centers of Bangor and Brewer to the northeast. Existing traffic is continually changing lanes on the bridge such that it isn't feasible for vehicles to travel abreast despite the fifty foot curb to curb width. Because of this odd traffic-determined lane use, the Traffic Division suggested that the new bridge be marked for the same traffic pattern as the previous structure. So, the new bridge pavement is marked with one fourteen-foot lane, one ten-foot shoulder, and one six-foot sidewalk with a standard bridge rail for each direction.

It was determined in the Preliminary Design Report, that the bridge would be marked and designed for one lane of vehicular traffic in each direction. The largest Live Load distribution factor considered was for two lanes loaded. This relatively low Live Load distribution factor, plus the continuity for Live Load were essential for providing the shallow beam design, and thereby the needed increase in underclearance.

This bridge is tunnel-like, and in that way, typical of many bridges that require better corrosion resistance than corrosion resistant steel beams provide. The details that allow this two-span concrete bridge to be jointless are certainly advantageous and applicable to typical overpasses. It is not very typical of tunnel-like bridges for its limited lane usage.. However, it is useful to explore the practical limits of superstructure depth minimization, since significant property and roadway costs may be avoided. In that sense, this case study illustrates a practical limit for Bulb-T depth minimization for a continuous-for-live-load design with a hand-calculated, live load distribution factor of 0.616 axles for flexure (0.617 as calculated via ConSpan).



Foundation Profile: Exterior Previous Beam & New Bridge @ CL

It may interest some readers to note that the City of Bangor initiated a change order to widen the sidewalks by a half foot. Despite the sidewalk's low 1% cross-slope, sidewalk snowplows tend to slide away from the rail when plowing and get stuck with a wheel line hanging over the gutter. The half foot width was subtracted from the wide shoulders at the beginning of the Construction phase, at the cost of a small amount of sidewalk concrete.

As implied above, considering its traffic volume the bridge was and remains wide to accommodate continuous traffic shifting from side to side, i.e. from centerline to gutter line or vice versa. Construction signing and activities effectively slow traffic. As a result, the staged construction width reduction to one ten-foot lane in each direction had little impact to traffic flow compared to most bridges of similar curb to curb width. The stage construction forced lateral traffic shifts off of the bridge during construction.

Stage Construction of 2nd Half of Abutment #2 Footing – SE Corner

Note the lighter shoring extending from the centerline shoring. The Contractor effectively used the same centerline shoring for both halves of the abutment backfill excavation.

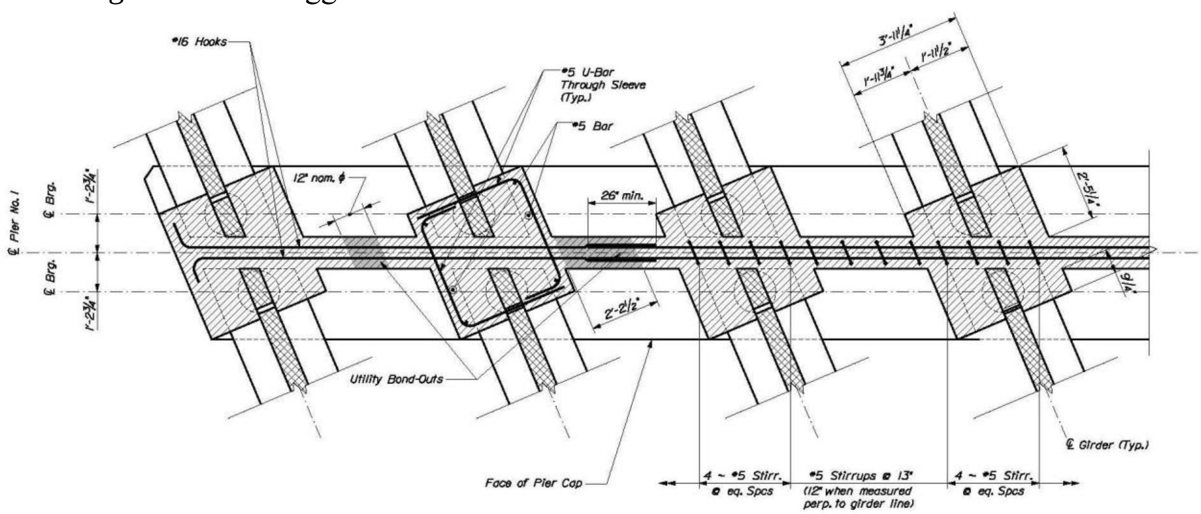
For the designer, one nightmarish staged construction concern was the maintenance of traffic over half of the existing bridge, which meant supporting trucks on half of each deteriorated, four column pier. The previously existing piers suffered visible deterioration, some apparently from Silica Alkaline Reactivity. Making matters worse, the original piers were for lighter loads. They did not have the reserve strength the replacement piers would. Without careful demolition of the first half removed, the Contractor could easily weaken the half bents kept to support the existing bridge and all vehicular traffic through the first stage of construction. Surprisingly, analysis showed that the pier bents would only need bracing if truck loads were transversely located such that an outside truck wheel-line was offset from the projected plan area of one of the remaining columns.

Half Existing Bridge Supported on Half of Westerly Pier

The Maine Department of Transportation is careful not to imply or dictate construction methods. Maintaining traffic safely is clearly the Contractor's responsibility, as prescribed by standard specification. However, if a project has a special sensitivity to procedures indicated in the plans, in this case staged construction, the Maine Department of Transportation will flag any recognized, unusual concerns in the plan documents. As for some previous projects, a General Construction Note stated, "the Contractor shall submit a Stage Removal and Construction Plan prepared and stamped by a licensed engineer." For this project the note added, "The submitted plan must provide for safe support of partially deconstructed structures under traffic loadings, such as the three existing pier caps . . ."

The Contractor's engineer for the stage construction plan, Calderwood Engineering Etc, solved the issue neatly. Instead of providing for the maximum temporary traffic width possible, and crowding the protected work area, the consulting engineer limited the Stage 1 traffic placement to a twenty foot width. The temporary traffic plan for Stage 1 centered the traffic between the two remaining columns such that it minimized stresses due to the possible truck wheel line locations. Admittedly, twenty feet is a small width for bi-directional traffic, but the reduced width area and its approaches are tangent, and construction traffic speeds were reduced. The safety measure gained from sharply reducing live load stresses was considered more than sufficient reason for limiting the temporary traffic width.

One surprise construction problem, still not understood in the office, was the discovery that some diaphragm reinforcement, placed at the stage construction joint before a winter break, had become misaligned. The plans called for the use of beam inserts for anchoring of the reinforcement of the two intermediate diaphragms at the construction. So, it seems the “misalignment” could only have occurred at the support diaphragms. The joints in support diaphragms were aligned with the face of the completed Beam #4 continuity collar. Presumably Stage 1 reinforcement steel extended a lap length beyond the joint at completion of the first stage of construction. If the presumption is correct that inserts were used as shown at the intermediate diaphragms, the problem must have occurred near the supports, where camber magnitudes and possible differential camber are very low. In retrospect, the beam to beam reinforcement for the stage construction intermediate diaphragms should have been lengthened and flagged “cut to fit”.



PIER DIAPHRAGM PLAN

Planview of Pier Diaphragm and Integral C.I.P. Continuity Collars

Close inspection of the detail on the previous page indicates that reinforcement passes through sleeves in the Bulb-T webs. A pair of two inch diameter sleeves is stacked vertically. More such sleeves and reinforcement would fit in taller Bulb-T’s to strengthen the cast-in-place splice. The sleeves are as close to the ends of the beam webs as practical. The designer chose to limit the length of the collars to 51.5 inches such that the collar corners did not extend beyond the pier cap (An observer would have to stand against the pier column to see this.). The width of each collar matches the width of the top flange of the Bulb-T, the larger flange. The above detail is based on the 2008 version of the Massachusetts D.O.T. LRFD Bridge Details as published on the internet. There were no intentional modifications, but also no consultation with any representative of the Massachusetts Department of Transportation. So, any deficiencies should be credited to the author.

Have you heard it said that God is in the details? On a good day yes, but in cases of skews well over seven degrees, it seems more appropriate to claim the devil is in the details.

The 23 degree skew complicated the project's construction formwork and structure dimensioning significantly. This was particularly troublesome at each support diaphragm and bridge seat area. The alignments of utility block-outs, and therefore the layout of somewhat crowded reinforcement around them had to be carefully considered. Consider the back of the approach slab, the line for joints and devices to accommodate thermal expansion/contraction. It must be square to the beams while the front of the slabs rests on a skewed backwall. Both approach slab areas are about sixty feet wide. Each slab system is divided into three segments. The alignment of the three approach slab expansion joints varies by 22 feet.

Framing for Formwork of the Pier Continuity Collars and Diaphragm

As noted previously, vehicular traffic on the bridge was maintained at all times except for nighttime girder erection. Safe crane positioning also required brief closures of the interstate in one direction, as shown below for the southbound lanes.

Erection of Northerly Span #1 Girders, Showing Back of Abutment #1

DETAILS FOR BEST PERFORMANCE

The author's personal preference is to avoid going to extremes to reduce the depth of prestressed concrete members, but his practice runs counter to his preference. Still, the designer and the Lead CADD Technician can find a great deal of satisfaction in meeting geometric requirements and making the design and plan as practical as we were able, considering durability, function, constructability, and fabrication. Sometimes bid prices will provide an indication of how well the design team did its task. The total bridge replacement contract was bid for $4.22 million, within approximately 5% of the bid estimate. From the back of one end diaphragm to the other the bridge length is 173.5 feet, and the superstructure width is 64 feet. Cost data for the superstructure versus the tall substructure is unavailable. Still, the designer presumes to consider this an economic success.

The rationale and advantages of many of this project's details have been presented above. To avoid repetition, this section may be weighted towards the disadvantages of this project's details. The choice of a jointless bridge system seems to be an undeniable advantage overall, and warrants emphasis. The motivation to design jointless bridges stems from a history of several bridge rehabilitation and replacement projects that were needed prematurely because of failed bridge seals, in my opinion.

Because of various material properties including lower thermal conductivity, concrete beams exhibit a smaller annual range of thermal expansion/contraction than steel beams. A smaller effective expansion/contraction range, promotes jointless designs for more span arrangements. From the AASHTO "Cold" climate thermal ranges for expansion/contraction[2] it can be determined that concrete superstructures need only be designed for 53% of the movement range of steel superstructures (conservatively assuming that the thermal expansion coefficient for reinforced concrete fully matches the constant for steel).

Several aspects of the combination tied-approach-slab and sleeper slab have been discussed already. The Maine Department of Transportation has been exploring at grade approach slabs for integral and semi-integral abutment bridges, whereas buried approach slabs may be considered “old school”. However, the MDOT Bridge Maintenance office has tested and negatively reviewed a few pavement compression joint systems. A good argument is made in studies of approach slab systems, that the soil pressure bulb at an approach slab end has a less effect on the pavement if it is buried. See the following page for a picture of: the tied approach slab; the sleeper slab; the back of the skewed Abutment #1backwall with a utility block-out; and the top of the return wingwall.

Approach Slabs: No Bond to Wingwall at the Slab Level

There are a few obvious disadvantages to the approach slab system. With the expansion/contraction system placed twenty feet behind the backwall, this work causes a longer traffic disruption to the approach roadway. The entire roadway (and utilities) over the tied approach slab will move longitudinally with annual thermal cycles, so the pavement cast against the stationary return wingwalls will tend to shear at the gutter line. Hopefully frictional forces from the sides of tied approach slabs do not move the adjacent sleeper slabs during expansion movements. With the predictive total range of expansion/contraction only ½ inch, there is hope that these effects will be insignificant. Since superstructure flexure ends at the floating backwall and the expansion/contraction movement is carried to the end of the tied approach slab, the gas utility company found it necessary to install joints to allow for the movement at both locations. Normally one joint fixture would suffice.

The author has had no prior experience with the massive “continuity collars” that were based on a Massachusetts D.O.T. detail. A colleague, the Preliminary Design Engineer for this Hammond Street Bridge project, used this type of continuity device for the Calais International Bridge between the Province of New Brunswick and the State of Maine. So far, I am not aware of any problems with the function of this type of continuity splice. However, with sincere thanks to Dr. Frank Russo and William Nickas P.E., I learned that in some instances enveloping, cast-in-place, support diaphragms have cracked around the bottom of prestressed concrete beams. The cracking is the result of rotation of the beam ends due to continuing camber growth after erection.

Intermediate Diaphragm Placement w/ Beams Awaiting Pier Splice

In my opinion, despite scheduling challenges and the potential to increase bid costs, it would be worthwhile for my agency to use more exacting procedures to assure that every beam has aged at least three months outside of the prestressing beds, and to track camber of beams to provide for valid bearing seat adjustments just prior to beam erection and valid slab blocking adjustments immediately after erection. Significant camber growth after erection is a serious problem, particularly for prestressed beams that are made continuous.

CONCLUSION

Winters are long, cold, and hard on bridges in Bangor Maine. Quality, plant-fabricated concrete is a superior beam material for resisting the direct corrosion effects of de-icing salts. In addition, concrete beams go through a much smaller range of annual thermal movements than steel beams, making concrete beams advantageous for jointless bridge designs.

The project's bid contract was awarded in November, 2008 for $4.22 million, which included a total span length of 167 feet, and 533 feet of approach roadway. The project ran behind schedule the first year, but was finished on time at the end of the second construction season. The projected 2015 Annual Average Daily Traffic is 14,700 vehicles per day.

PROJECT TEAM

Project Manager: Devin Anderson, PE

Project Designer: Robert Bulger, PE

Preliminary Designer: M. Asif Iqbal, PE

Geotechnical Engineer: GZA GeoEnvironmental Inc.

Project Checker: Costello Lomasney & Denapoli, Inc.

Lead CADD Technician: David Shaw

Project Resident (Field Engineer): Phil Roberts, PE

Prime Contractor: T Buck Construction, Inc.

Chief Inspector: Guy Hews

REFERENCES

[1] FHWA Technical Advisory 5140.22, issued October 3, 1989. See subparagraph 3b.1 "Grade Separations in "Tunnel-Like" Conditions for the environmentally dependent susceptibility of "corrosion resistant' steel to corrosion where beams are frequently sprayed with salty water.

[2] 2012 AASHTO LRFD Bridge Design Specifications, Article 3.12.2.1 and Table 3.12.2.1-1