**DESIGN DETAILS – SUPERSTRUCTURE DESIGN**

**DESIGN ISSUES**
What is a Prefabricated Bridge Unit (PBU)?
A “mini bridge”; two beams with a composite deck – cast off-site prior to construction.

**PBU DESIGN CONSIDERATIONS**
The beam design is no different than in regular construction.
*ON 93FAST14, PROCUREMENT SCHEDULE RESULTED IN USING WELDED PLATE GIRDERs*
- Plate more readily available than rolled sections
- Rolled sections without cover plates will be heavier than welded plate girders
- Adding cover plates to rolled sections negates cost advantage
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SHIPPING AND ERECTION
Length is the most significant consideration to the practicability of using PBUs; Prefer to keep lengths to 120'-0" or less for shipping.

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SHIPPING AND ERECTION
Length needs to account for effect of skew and width.

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SHIPPING AND ERECTION
Prefer to keep the width of the PBU to 10'-0" and no more than 12'-0". Wider is certainly possible but it introduces shipping complications; permits, restricted travel routes, restricted travel hours, etc.
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SHIPING AND ERECTION
Closure pour details affect both width and weight of the PBU!

- Closure Pour with Dowel Bar Splicers
- Alternative Closure Pour with Hooked Bars
- Closure Pour with Extended Rebar if Dowel Bar Splicers are Not Used

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ERECTION CONCERNS
The pick weight of the PBU may be the most important factor – crane size and location.
MassDOT requires an additional safety factor of 150% - i.e. multiple the pick weight by 1.5 and include all rigging, blocking and/or spreaders.

- Wide closure with DBS = less PBU weight
- Narrow closure with hooked bars = more PBU weight

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CLOSURE POUR OPTIONS
Considered two options for this project:
1. Dowel Bar Splicers splicing top and bottom mat of steel to make a continuous deck.
   Width of the closure pour is a function of required lap for deck steel plus consideration of tolerances and clearances for concrete placement.
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CLOSURE POUR OPTIONS

2. The other option was a 16” closure pour using hooped #4 @ 5” o.c. with longitudinal bars “laced” through.

There were concerns over erecting and placing PBU’s with this type of closure pour.

#4 Hoops (180°) @ 5” o.c.
Theoretical clearance 2” – concerned impact on erection duration.

There were concerns over erecting and placing PBU’s with this type of closure pour.

Hooped Bar Closures will be wider and therefore heavier.
Dowel Bar Splicer Closures will require more closure pour concrete which can be expensive ($5k? per yard).
Dowel Bar Splicer Closures will have more rebar costs, but maybe lower forming costs.

<table>
<thead>
<tr>
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<th>DBS Closure</th>
<th>Hooped Bar Closure</th>
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<tbody>
<tr>
<td>Panel Width</td>
<td>PRO</td>
<td>CON</td>
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<tr>
<td>Panel Weight</td>
<td>PRO</td>
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<tr>
<td>Closure Pour Concrete</td>
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<tr>
<td>Constructability</td>
<td>PRO</td>
<td>CON</td>
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</table>
Hooped Bar Closures could be difficult to place – maneuvering the PBU with tightly spaced overhanging bars while hooked to a crane could lead to longer erection times.

Dowel Bar Splicer Closure will require more labor and time to place the rebar.

<table>
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<th>Closure Type</th>
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It was decided to use a 32" wide DBS Closure for this project:
- 2'-1" lap plus 4" for either side for concrete placement and rebar tolerance
- Rebar tolerance is equal to 1" on length
- Possible even on the Dowel Bar Splicer dowels both the male and female sections
- This however never actually became a problem.

Decision was based upon reducing pick weights and shipping. The result was a typical beam spacing of 6'-3".

Note, there is a third possible option using short straight dowels and Ultra High Performance Concrete (UHPC); however this was not considered for this project.
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DIAPHRAGM LAYOUT

- BTC documents had a typical diaphragm/cross frame layout
- Maximum spacing of 25'-0" – typical MassDOT standard
- No discontinuous lines of diaphragms
- Resulted in many diaphragms near the supports which causes fabrication and erection issues
- Especially concerned with installation of diaphragms between PBUs

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Final Design Framing Plan

**ELIMINATED** Diaphragm/cross frame near obtuse corners
- Allowed for discontinuous lines of diaphragms

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Base Technical Concept Framing Plan

Diaphragm/cross frame near obtuse corners
- Cause fabrication and erection problems
- Concerned with similar diaphragms erected during rapid weekend
- Maximum Spacing = 25'-0"
Diaphragms/cross frames have very little impact upon live load distribution, once the deck has been placed and cured
But are vital until that time – and they do participate in distributing wind loads
Revised layout based upon AASHTO LRFD Provisions

C6.7.4.1 - The arbitrary requirement for diaphragms spaced at not more than 25.0 ft in the AASHTO Standard Specifications has been replaced by a requirement for rational analysis that will often result in the elimination of fatigue-prone attachment details.

C6.7.4.2 - Where support lines are skewed more than 20 degrees from normal, it may be advantageous to place the intermediate diaphragms or cross-frames oriented normal to the girders in discontinuous lines, such as those shown in Figure C4.6.2.7.1(b) and C6.10.3.4.

Intermediate diaphragms or cross-frames should be provided at nearly uniform spacing in most cases for efficiency of the structural design, for constructability, and/or to allow the use of simplified methods of analysis for calculation of flange lateral bending moments, such as those discussed in Articles C4.6.2.7.1(b) and C6.10.3.4.

WHAT DO DIAPHRAGMS DO?
- Transfer lateral wind load to the supports
- Stability for all bottom flange when in compression
- Stability of top flange when in compression prior to curing of the deck
- Consideration of any flange lateral bending effects
- Distribution of vertical loads

WHAT ARE THE APPLICABLE DESIGN PROVISIONS?
- C4.6.2.7.1 – lateral forces in flanges due to wind – this was applied to all beams
- C6.10.3.4 – calculates the effect of concrete deck placement on lateral flange bending
  - This should include the force effects of deck overhang brackets
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The result is that each PBU was considered a separate bridge for its fabrication.
Diaphragms were centered around midspan at the traditional 25'-0" spacing.

This is the location where flange stresses are the highest and therefore diaphragm spacing the most critical.

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For the remainder of the span they were spaced at longer intervals based upon the calculations.

Eliminating the need to have continuous lines of diaphragms was the biggest benefit.

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WHAT TO DO FOR DIAPHRAGMS BETWEEN THE PBUs?

Because of the uncertainty, during preliminary and final design, regarding the strength of the closure pour we made the arbitrary decision to add additional diaphragms between the PBUs.

The concept was that if the strength of the concrete was low we wanted to minimize the differential deflection between PBUs and therefore minimize any cracking.

Diaphragms were lined up as much as possible with the ones in the PBUs.

And then the diaphragms at midspan, between PBUs, were doubled up.
USED OVERSIZED HOLES TO AID IN THE FIELD ERECTION

Wherever there was a discontinuous line of diaphragms a full height stiffener was placed on the opposite face

- The intent was to minimize any distortion induced stress about the web to flange connection – keep the flanges square to the web

TWO OTHER MATTERS CONCERNING DIAPHRAGMS

- BTC allowed for cold bent plate diaphragms with rolled sections the use of optional
- We used the rolled sections;
- Costs?
- Procurement schedule?

Diaphragm depth requirements;

6.7.4.2 … but as a minimum should be at least 0.5 of the beam depth for rolled beams and 0.75 of the girder depth for plate girders

All of the girders were welded plate girders; regardless of the depth – web plates from 44” to 20” deep

On all of the girders it was decided to followed the MassDOT standard for diaphragm sizes, with the biggest being MC18x42.7
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OTHER ERECTION CONCERNS:
Typically place lifting devices (Lugs) at ¼ points

Strength of the beams during the pick is typically not a problem but again still checked

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Lifting lugs – welded to top flange

Lug is then burnt off and pocket filled with grout

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DECK DESIGN
Used AASHTO 4.6.3.2.1 for the ability to conduct a refined analysis (FEM)
WHY?
Schedule and ...
Needed to plan for a contingency where the closure pour did not get placed or cured
AASHTO Appendix A4 – Deck Design Table requires the deck to be continuous over three or more beams
Unclear if AASHTO Equivalent Strip method (Table 4.6.2.1.3-1) has the same requirement
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Deck would be a simple span with overhangs. The overhangs would support steel plates to span over the closure pour and carry traffic.

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

Used FEM to design moments and shears.

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0.75" φ H.S. BOLT (A325) – 6'-8" O.C. 20' LONG x 3'-1" WIDE x 1.5" THICK STEEL PLATE .5 SHIM PLATE
Design at “INVENTORY” with 4,000 psi and check for OPERATING with 2,000 psi concrete. The challenge became detailing... DEVELOP NEGATIVE MOMENT STEEL FOR OVERHANG. However, overhangs were not long enough to develop bar. INSUFFICIENT DEVELOPMENT LENGTH.

Therefore used a 180° hooked bar. However, depth of slab was not enough for a #5 therefore needed to design for #4 bar. #4 @ 6' WITH 180° HOOK.
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PROVIDE SHEAR KEY FOR CLOSURE POUR
Needed to provide a stable shelf for temporary plate to cover closure pour > 1.5” deep
Also needed a flat surface to attach female end of DBS

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EFFECT OF SKEW ON REINFORCING
DBS “wants” to be placed 90 to the face of the form

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All of the main deck steel was perpendicular to the beams so this was not a problem for the link slabs
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

However, it was an issue for the Link Slabs which generally have the rebar running parallel to the beams and therefore skew to edge of the form. Instead of bending all of the female DBS, worked with fabricator to come up with a way of placing DBS on skew.

Also concerned with congestion of reinforcing in acute corners.
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DESIGN OF TEMPORARY ROAD PLATES
Again used FEM to design plate and determine reaction on slab overhang
Longitudinal plates ran abutment to abutment, with the transverse plates cut to fit the skew
Basic design became 1.5” thick plate Fy=50 ksi
At intersection of transverse and longitudinal plates additional stiffening was required – transvers plates supported longitudinal

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LINK SLAB DESIGN
What is a Link Slab?
Elimination of a roadway joint using a continuous deck at a pier where the beams are simply supported
Can be a tool for new bridges when …
• Accelerated bridge construction techniques
• If cost savings due to bolted field splice elimination exceed added steel weight

Article in September Modern Steel Construction – by Michael Culmo

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

DESIGN PROCEDURE
Based upon “Behavior and Design of Link Slabs for Jointless Bridge Decks” by Caner and Zia published by PCI
Design for slab moment due to beam rotation
Rotation result from live load and applicable superimposed dead load
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LINK SLAB CONSTRUCTION

1. Pour deck, block out link slab section.
2. Place rebar and pour link slab.
3. Shear connectors eliminated from link slab.

The total number of shear studs required to meet strength requirements must still be provided.

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LENGTH OF LINK SLAB

% LEFT
% RIGHT

% = 5% - 7% of span length
Considered a variable – the higher the % used the lower the Ma
Debonded length does not have to be the same percentage for each span

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LINK SLAB DESIGN

\[ M_a = \begin{cases} 12\theta_{LL} + 2\theta_{SDL} \frac{I_g}{L'} & \text{for } L' \leq 16\Delta \frac{E_c}{2}\theta_{LL} \\ 0 & \text{for } L' > 16\Delta \frac{E_c}{2}\theta_{LL} \end{cases} \]

\( \theta = \frac{16\Delta}{2\theta_{SDL} \frac{E_c}{2}\theta_{LL}} \)

Ec = the modulus of elasticity of the link slab concrete
Ig = uncracked moment of inertia of the slab
L' = length of link slab = debonded length + distance between bearings

Use AASHTO Load Case Service I \( \gamma = 1.0 \) for all loads
Live Load for Link Slab LRFD AASHTO 2.5.2.6.2 and 3.6.1.3.2

2.5.2.6.2 - Criteria for Deflection
... When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally

Also applied Multiple Presence Factor (AASHTO ...)
Similarly superimposed dead load was distributed equally

3.6.1.3.2 - Loading for Optional Live Load Deflection Evaluation
... the deflection should be taken as the larger of:
* That resulting from the design truck alone, or
* That resulting from 25 percent of the design truck taken together with the design lane load

Check design moment Ma against cracking moment of the slab – if Ma > Mcr then reinforcing is required
AASHTO LRFD Eq 5.7.3.6.2-2 \[ M_{cr} = f_{t} \frac{d}{2} \]

Check crack control
AASHTO LRFD Eq 5.7.3.4-1 \[ s \leq \frac{700 \sigma_t}{f_y} - 2d_c \]
Where \( \beta_s = 1 + \frac{d_e}{0.7(h - d_c)} \)
\( \gamma_s = \) exposure factor
1.00 for Class 1 exposure condition
0.75 for Class 2 exposure condition
\( d_e = \) thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement
\( f_{ts} = \) calculated tensile stress in mild steel reinforcement at the service limit state not to exceed 0.60 (ksi)
\( h = \) overall thickness or depth of the component (in.)

BEARING DESIGN
Originally designed according to Method A
Actual construction tolerances accompanying accelerated bridge construction – initiated questions on the long term performance of the bearings
Field measured all bearings (1008) to determine bearing area in contact and rotational measurements. Fabricator had internal QC material testing results meeting the requirements of Method B.

AASHTO LIMIT - 5.0

MassDOT Limit – 4.75

ACTUAL ROTATIONS

MassDOT Limit – 0.030 radians

AASHTO Limit – 0.005 radians
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Suggest design according to Method B
Increased costs due to additional testing requirements are minor when compared to addresses issues during accelerated bridge construction
Increase Uncertainty Tolerances
0.030 radians (MassDOT) versus 0.005 radians for AASHTO LRFD
Reduce “allowable” on Equation 14.7.5.3.1 Limits
4.75 (MassDOT) versus 5.0 for AASHTO LRFD

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WATERPROOFING
Consider using spray applied membrane waterproofing system
Three coat cold liquid spray applied methylmethacrylate system (or polyurea); span cracks up to 1/8" wide
Required traffic to run on exposed deck

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BARRIERS
Depending on required Crash Test Level – precast barriers may not be an option
93Fast14 was on Interstate Highway – MassDOT requires TL-5
Presently unaware of any precast crash tested barriers meeting TL-5
Permeant barriers were cast in place
Used shoulder with the New York State Temporary Concrete Barrier with Box Beam Stiffener to protect work zone
Crash tested to Mash Test Level 3 and accepted by FHWA