93 FAST 14
SUPERSTRUCTURE DESIGN

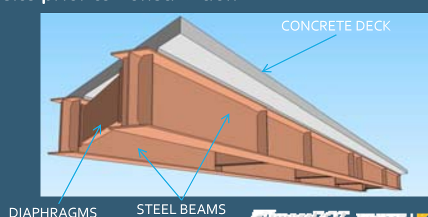
ABC-UTC In-depth Web Training
MassDOT's 93FAST14 Project
November 4, 2014

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



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

Photograph courtesy of J. F. White Contracting Co.

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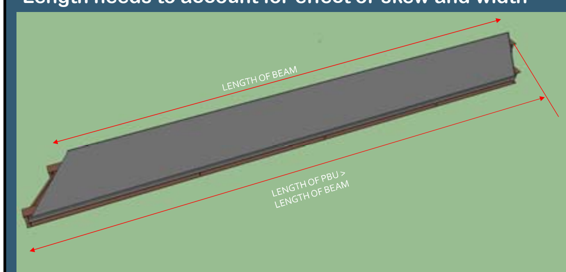
SHIPPING AND ERECTION

Length needs to account for effect of skew and width



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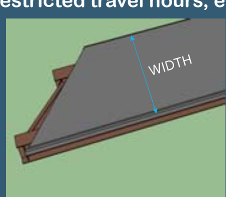
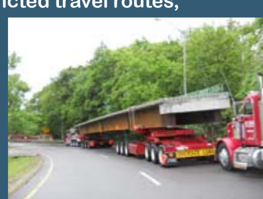
SHIPPING AND ERECTION

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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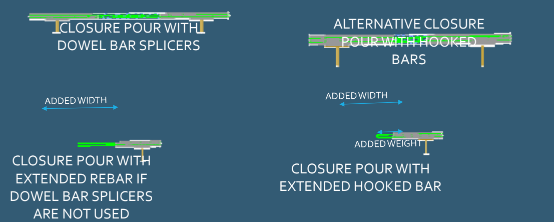
SHIPPING AND ERECTION

Closure pour details effect both width and weight of the PBU!



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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

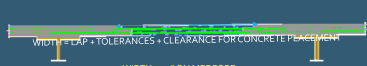
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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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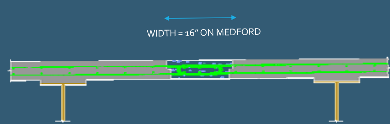
CLOSURE POUR OPTIONS

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



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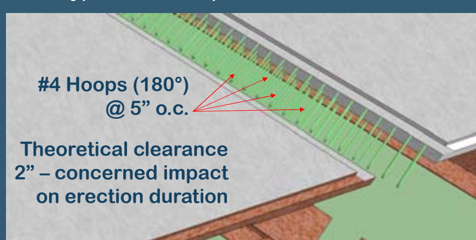
CLOSURE POUR OPTIONS

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



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DESIGN DETAILS – SUPERSTRUCTURE DESIGN

	DBS Closure		Hooped Bar Closure	
	PRO	CON	PRO	CON
Panel Width	(green)			(red)
Panel Weight	(green)			(red)
Closure Pour Concrete		(red)	(green)	
Constructability	(green)	(red)	(green)	(red)

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

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DESIGN DETAILS – SUPERSTRUCTURE DESIGN

	DBS Closure		Hooped Bar Closure	
	PRO	CON	PRO	CON
Panel Width	[green]			[red]
Panel Weight	[green]			[red]
Closure Pour Concrete		[red]	[green]	
Constructability	[green]	[red]	[green]	[red]

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

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It was decided to use a 32” wide DBS Closure for this project

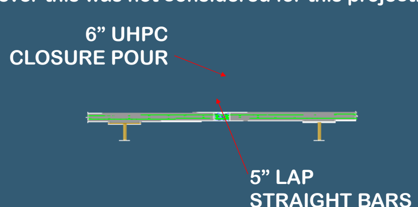
- 2’-1” lap plus 4” for either side for concrete placement and rebar tolerance
- Rebar tolerance is equal to 1” on length
- Possible even on the Dowel Bar Splicer dowels both the male and female sections
- This however never actually became a problem

Decision was based upon reducing pick weights and shipping

The result was a typical beam spacing of 6’-3”

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

- 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.



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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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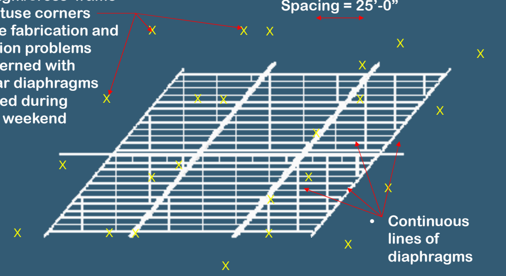
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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”



- Continuous lines of diaphragms

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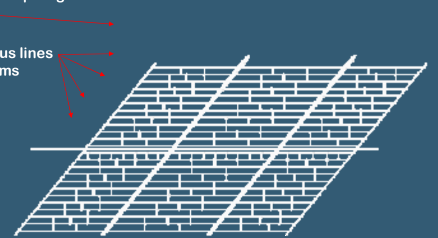
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

Final Design Framing Plan

ELIMINATED Diaphragm/cross frame near obtuse corners

- Allowed for discontinuous lines of diaphragms



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DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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.

From 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 in such a manner that the transverse stiffness of the bridge is reduced, particularly in the vicinity of the supports …

Removal of highly stressed diaphragms or cross-frames, particularly near obtuse corners, releases the girders torsionally and is often beneficial as long as girder rotation is not excessive.

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From C6.7.4.2 … Intermediate diaphragms or cross-frames should be provided at nearly uniform spacing in most cases, for efficiency of the structural design, for constructibility, and/or to allow the use of simplified methods of analysis for calculation of flange lateral bending stresses, such as those discussed in Articles C4.6.1.2.4b, C4.6.2.7.1 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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

WHAT ARE THE APPLICABLE DESIGN PROVISIONS?

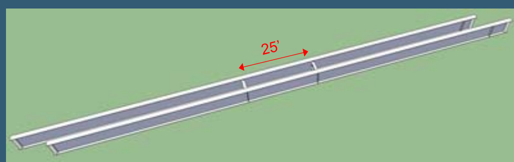
- C4.6.2.7.1 – lateral forces in flanges due to wind – this was applied to all beams
- C6.10.3.4 – calculates the effect of concrete deck placement on lateral flange bending
  - This should include the force effects of deck overhang brackets

$$P = \frac{\Sigma M}{H}$$

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

The result is that each PBU was considered a separate bridge for its fabrication

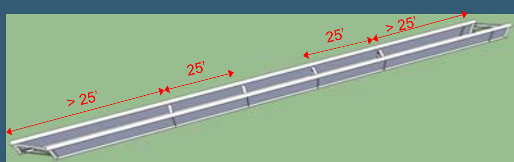
Diaphragms were centered around midspan at the traditional 25’-0” spacing



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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

WHAT TO DO FOR DIAPHRAGMS BETWEEN THE PBUs?

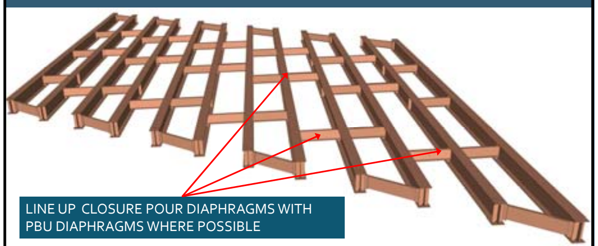
Because of the uncertainty, during preliminary and final design, regarding the strength of the closure pour we made the arbitrary decision to add additional diaphragms between the PBUs.

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.

DESIGN DETAILS – SUPERSTRUCTURE DESIGN



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

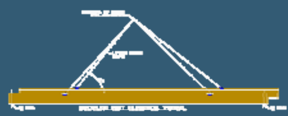
All of the girders were welded plate girders; regardless of the depth – web plates from 44” to 20” deep

On all of the girders it was deicide to followed the MassDOT standard for diaphragm sizes, with the biggest being MC18x42.7

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

OTHER ERECTION CONCERNS;

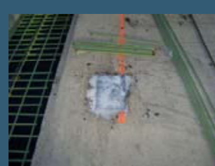
Typically place lifting devices (Lugs) at ¼ points



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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

Lifting lugs – welded to top flange



Lug is then burnt off and pocket filled with grout

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

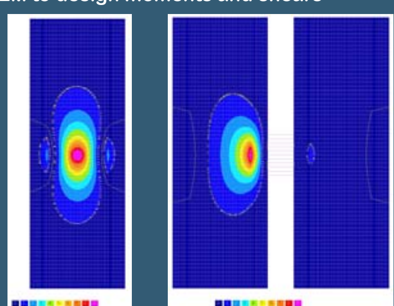
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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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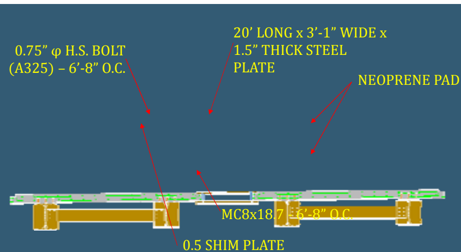

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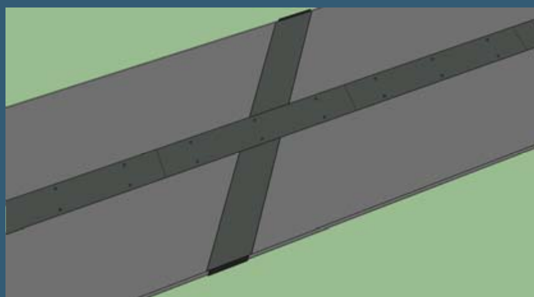


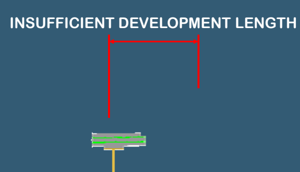
DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

Therefore used a 180° hooked bar

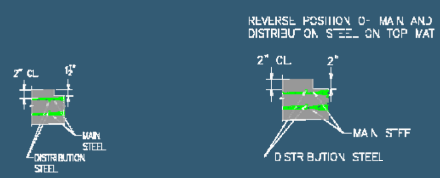
However, depth of slab was not enough for a #5 therefore needed to design for #4 bar

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

PROVIDE SHEAR KEY FOR CLOSURE POUR

Needed to provide a stable shelf for temporary plate to cover closure pour > 1.5” deep

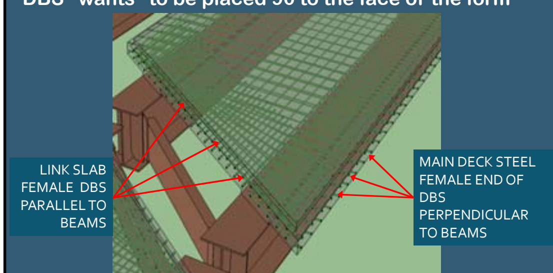
Also needed a flat surface to attach female end of DBS



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

EFFECT OF SKEW ON REINFORCING

DBS “wants” to be placed 90 to the face of the form



DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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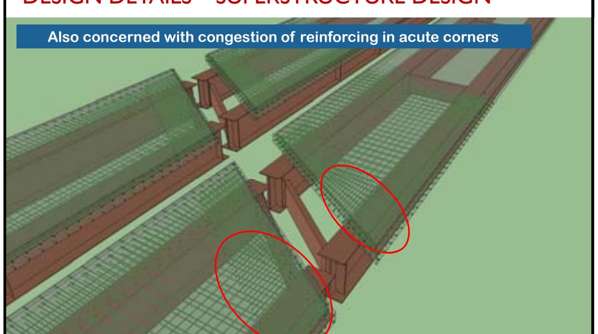


DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

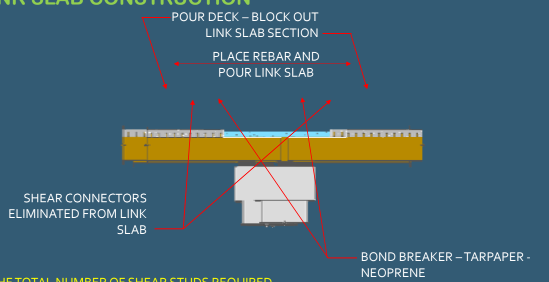
Based upon "Behavior and Design of Link Slabs for Jointless Bridge Decks" by Caner and Zia published by PCI

Design for slab moment due to beam rotation

Rotation result from live load and applicable superimposed dead load

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

LINK SLAB CONSTRUCTION



THE TOTAL NUMBER OF SHEAR STUDS REQUIRED TO MEET STRENGTH REQUIREMENTS MUST STILL BE PROVIDED

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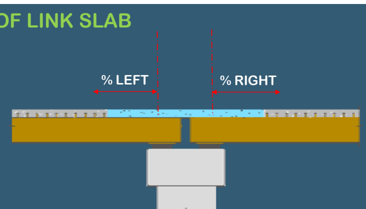
LENGTH OF LINK SLAB



% = 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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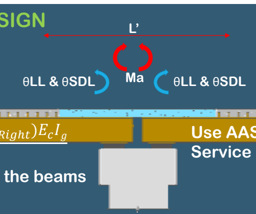
LINK SLAB DESIGN



$$M_a = \frac{(\Sigma\theta_{Left} + \Sigma\theta_{Right})E_c I_g}{L'}$$

Use AASHTO Load Case Service I - $\gamma = 1.0$ for all loads

Θ = the rotation of the beams

$$\theta = \frac{16\Delta}{5Span}$$

Ec = the modulus of elasticity of the link slab concrete

I_g = uncracked moment of inertia of the slab

$$I = \frac{Beam\ Spacing \times Slab\ Thickness^3}{12}$$

L' = length of link slab = debonded length + distance between bearings

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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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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 $M_{cr} = fr\frac{I_g}{y_t}$

Check crack control

AASHTO LRFD Eq 5.7.3.4-1 $s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2dc$

Where $\beta_s = 1 + \frac{d_c}{0.7(h-dc)}$

γ_e = exposure factor
1.00 for Class 1 exposure condition
0.75 for Class 2 exposure condition

de = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement

fss = calculated tensile stress in mild steel reinforcement at the service limit state not to exceed 0.60 *fy* (ksi)

h = overall thickness or depth of the component (in.)

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

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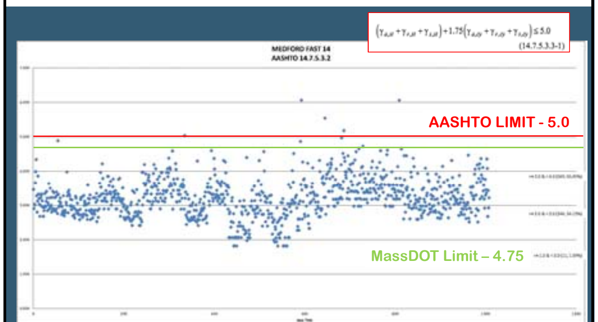

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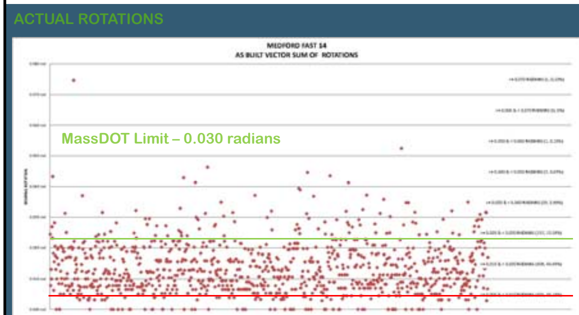
ACTUAL ROTATIONS



AASHTO Limit – 0.005 radians

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DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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 Equation14.7.5.3.3-1 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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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

DESIGN DETAILS – SUPERSTRUCTURE DESIGN

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QUESTIONS