Ultra-High Performance Concrete (UHPC) Link Slab Design Example

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FOREWORD

In 2021, ultra-high performance concrete (UHPC) for bridge preservation and repair was rolled out as one of the innovative technologies in the Federal Highway Administration's Every Day Counts program. UHPC link slabs were identified as one of the most promising applications of UHPC for preservation and repair. UHPC link slabs can be used to eliminate deck joints between simple spans to help extend the service life of existing bridges. The information presented in this document provides background, context, and foundational knowledge to bridge owners and designers interested in using UHPC link slabs for preserving our Nation's highway bridges.

This report will be of interest to bridge owners and bridge designers looking for a simple and innovative solution to retrofit deteriorated or leaking bridge deck joints and preserve the superstructure and substructure elements below them.

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Notice

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The initial development of the design calculation package was performed by Calvin Chong in FHWA's Eastern Federal Lands Highway Division. Michael McDonagh began the transformation of the calculation package into the format of a design example. David Garber further developed and refined the example, carrying it through to publication.

16. Abstract

This report explains the fundamental concepts behind and provides the design calculations for an ultra-high performance concrete (UHPC) link slab. In this design example, an existing four-span, steel simple-span composite bridge containing traditional expansion joints is used as the base structure. The example steps through the process of designing link slabs that will replace the three interior expansion joints on the bridge. Construction details for the existing structure and link slab design are provided. Calculations are provided to determine the longitudinal horizontal loads for the structure to evaluate the effect of the link slabs on the existing substructure and foundation elements. The redesign of the bearings is also included.

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CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOTATION

A	plan area of elastomeric element or bearing (inch^2)
A_c	gross area of the cross section of the member (inch^2)
A_{min}	minimum bearing area (inch^2)
A_s	longitudinal reinforcement density (inch^2/ft)
A_{sb}	total steel area (inch^2)
A_{st}	transformed steel beam area (inch^2)
A_{WLe}	area exposed to wind load, existing (per pier) (ft^2)
A_{WLp}	area exposed to wind load, proposed (per fixed pier) (ft^2)
a	creep / initial deflection ratio
B	bridge width (ft)
BR_e	controlling existing vehicular braking force (kip)
BR_{e1}	existing vehicular braking force due to 25% of axle weights (kip)
BR_{e2}	existing vehicular braking force due to 5% of lane loading (kip)
BR_p	controlling proposed vehicular braking force (kip)
BR_{p1}	proposed vehicular braking force due to 25% of axle weights (kip)
BR_{p2}	proposed vehicular braking force due to 5% of lane loading (kip)
C	resultant compression force (kip/ft)
C_D	drag coefficient
C_{FAT}	resultant compression force for the fatigue analysis of reinforcement (kip/ft)
C_{SER}	resultant compression force for the service limit case (kip/ft)
C_{STR}	resultant compression force for strength limit case (kip/ft)
c	assumed neutral axis depth (inch)
c_{cyc}	neutral axis depth for the cyclic analysis of UHPC (inch)
c_{FAT}	neutral axis depth for the fatigue analysis of reinforcement (inch)
c_{it}	new assumed neutral axis depth (inch)
c_{iti}	new assumed neutral axis depth (inch)
c_{SER}	neutral axis depth for the service limit case (inch)

c_{STR} neutral axis depth for the strength limit case (inch)

D_{col} diameter of column (ft)

DC component dead load (kip)

DW wearing surface dead load (kip)

d_h hole diameter (in)

d_t depth to tensile reinforcement (inch)

$d_{t,cr}$ distance to the elastic tensile strain limit from centroid (inch)

$d_{t,cr,FAT}$ distance to the elastic tensile strain limit from centroid for the fatigue analysis of reinforcement (inch)

$d_{t,cr,SER}$ distance to the elastic tensile strain limit from centroid for the service limit case (inch)

$d_{t,cr,STR}$ distance to the elastic tensile strain limit from centroid for the strength limit case (inch)

E_c modulus of elasticity of concrete (ksi)

E_s modulus of elasticity of steel reinforcement (ksi)

err neutral axis depth error (inch)

err_L debond length error (inch)

F_{TU} thermal load on piers (kip)

f'_c compressive strength of concrete for use in design (ksi)

$f_{ca,FAT}$ allowable stress at extreme compression fiber of the UHPC section for the fatigue limit case (ksi)

$f_{ca,SER}$ allowable stress at extreme compression fiber of the UHPC section for the service limit case (ksi)

$f_{c,SER}$ stress at extreme compression fiber of the UHPC section for the service limit case (ksi)

$f_{c,STR}$ stress at extreme compression fiber of the UHPC section for the strength limit case (ksi)

f_{min} minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the unfactored permanent loads or the unfactored permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

f_s stress in steel (ksi)

$f_{sa,SER}$ allowable stress in steel for the service limit case (ksi)

$f_{s,FAT}$ stress in steel for the fatigue analysis of reinforcement (ksi)

$f_{s,SER}$ stress in steel for the service limit case (ksi)

$f_{s,STR}$ stress in steel for the strength limit case (ksi)

$f_{t,cr}$ effective cracking strength of UHPC for use in design (ksi)

$f_{t,loc}$ crack localization stress of UHPC for use in design (ksi)

$f_{t,max}$ stress at extreme tension fiber (ksi)

$f_{t,max,FAT}$ stress at extreme tension fiber for the fatigue analysis of reinforcement (ksi)

$f_{t,max,SER}$ stress at extreme tension fiber for the service limit case (ksi)

$f_{t,max,STR}$ stress at extreme tension fiber for the strength limit case (ksi)

f_y specified minimum yield strength of reinforcement (ksi)

f_{SH} stress due to shrinkage of UHPC (ksi)

G gust effect factor

G_{max} maximum shear modulus of elasticity (ksi)

G_{min} minimum shear modulus of elasticity (ksi)

H average annual ambient relative humidity (percent)

H_w height of structure area exposed to wind (ft)

H_e controlling existing longitudinal load (kip)

$H_{e,STRIII}$ existing Strength III longitudinal load (kip)

$H_{e,STRV}$ existing Strength V longitudinal load (kip)

H_p controlling proposed longitudinal load (kip)

$H_{p,STRIII}$ proposed Strength III longitudinal load (kip)

$H_{p,STRV}$ proposed Strength V longitudinal load (kip)

h link slab thickness (inch)

h_b total bearing height (inch)

h_d total deck thickness (inch)

h_g girder height (inch)

h_h haunch thickness (inch)

$h_{min,FAT}$ minimum thickness of steel reinforcement for fatigue (inch)

$h_{min,SER}$ minimum thickness of steel reinforcement for service limit case (inch)

h_r distance from center of rotation to top of support (inch)

h_{ri} thickness of *i*th internal elastomeric layer (inch)

h_{rt} total elastomer thickness (inch)

$h_{rt,min}$ minimum elastomer thickness (inch)

h_s thickness of steel reinforcement (inch)

I_{col} moment of inertia of the column gross concrete section about the centroidal axis, neglecting the reinforcement (inch^4)

K_z pressure exposure and elevation coefficient

K_1 correction factor for modulus of elasticity to be taken as 1.0 unless determined by physical test, and as approved by the owner

K_4 correction factor for shrinkage to be taken as 1.0 unless otherwise determined by physical tests and as approved by the owner

k_f factor for the effect of UHPC strength

k_{hs} humidity factor for shrinkage

k_{pier} longitudinal stiffness of pier (kip/ft)

k_s factor for the effect of the volume-to-surface ratio of the component

k_{td} time development factor

L plan dimension of the bearing perpendicular to the axis of rotation under consideration (generally parallel to the global longitudinal bridge axis) (inch)

L_{col} height of pier (ft)

L_{DB} debond zone length (inch)

Length bridge length (ft)

L_{cyc} debond length for the cyclic analysis of UHPC (inch)

L_{it} initial assumed debond length (inch)

L_{iti} new assumed debond length (inch)

L_{shr} debond length for typical UHPC shrinkage (inch)

LL live load (kip)

n modular ratio = E_s/E_c or E_p/E_c

n_{col} number of columns

n_{fix} number of fixed piers

n_{ri} number of elastomer layers

n_s number of steel layers

n_δ minimum bearing layers, based on deflection

$P_{SER\ I}$ Service I load (kip)

$P_{STR\ I}$ Strength I load (kip)

P_z design wind pressure (ksf)

$ratio_L$ ratio of debond length for the fatigue analysis of UHPC determined by analysis to the provided debond length

SH force effect due to shrinkage

S_i shape factor of the *i*th internal layer of an elastomeric bearing

$S_{i,hole}$ shape factor of the *i*th internal layer of an elastomeric bearing with hole

$S_{max,A}$ maximum shape factor for using Method A per AASHTO LRFD BDS 14.7.6.1

$Span$ span length (ft)

T resultant tension force (kip/ft)

T_d design temperature range (°*F*)

T_{FAT} resultant tension force for the fatigue analysis of reinforcement (kip/ft)

T_{SER} resultant tension force for the service limit case (kip/ft)

T_{STR} resultant tension force for the strength limit case (kip/ft)

TU thermal load on piers (kip)

t_d deck thickness (inch)

tol neutral axis depth error tolerance (inch)

tol_L debond length tolerance (inch)

t_{ws} future wearing surface over link slab thickness (inch)

V design 3-second gust wind speed (mph)

W width of the bearing (inch)

WL_e existing wind load per pier (kip)

WL_p proposed wind load per fixed pier (kip)

WS_{60Le} longitudinal wind load, 60-degree, existing (per pier) (kip)

WS_{60Lp} longitudinal wind load, 60-degree, proposed (per fixed pier) (kip)

X distance from thermal center (ft)

Z approximate height of structure (ft)

α_s steel thermal expansion coefficient (1 / °F) (Chapter 4)

α_u reduction factor to account for the non-linearity of the UHPC compressive stress-strain response

γ_u reduction factor to account for the variability in the UHPC tensile stress parameters

Δ_b horizontal deflection at top of roller or bearing support

Δ_{DW} wearing surface deflection (inch)

ΔF_{TH} constant amplitude fatigue threshold taken from AASHTO LRFD BDS Table 6.6.1.2.5-3 for the detail category of interest (ksi)

Δ_{LL} live load deflection (inch)

ΔT assumed thermal range (°F)

ΔT_{max} maximum temperature range (°F)

Δ_s shear deformation (inch)

Δ_t temperature deformation (inch)

Δ_{TU} thermal expansion between fixed piers (inch)

$\Delta_{percent}$ percent difference between proposed and existing controlling longitudinal loads (kip)

Δ_{value} difference between proposed and existing controlling longitudinal loads (kip)

δ_c deflection due to camber (inch)

δ_d initial dead load deflection (inch)

δ_{da} allowable dead load deflection (inch)

δ_{DL} deflection due to dead load (inch)

δ_L instantaneous live load deflection (inch)

δ_{La} allowable live load deflection (inch)

δ_{LL} deflection due to live load (inch)

δ_{lt} long-term dead load deflection (inch)

ε_c compressive strain in extreme compression fiber of the UHPC section (inch/inch)

$\varepsilon_{c,FAT}$ compressive strain in extreme compression fiber of the UHPC section for the fatigue analysis of reinforcement (inch/inch)

ε_{cp} elastic compressive strain limit (inch/inch)

$\varepsilon_{c,SER}$ compressive strain in extreme compression fiber of the UHPC section for the service limit case (inch/inch)

$\varepsilon_{c,STR}$ compressive strain in extreme compression fiber of the UHPC section for the strength limit case (inch/inch)

$\varepsilon_{ca,STR}$ allowable net compression strain in extreme compression fiber of the UHPC section for the strength limit case (inch/inch)

ε_{cu} ultimate compressive strain of UHPC for use in design (inch/inch)

ε_{DL} compressive strain in bearing for dead load (%)

ε_{LL} compressive strain in bearing for live load (%)

ε_{s} net tensile strain in the extreme tension steel (inch/inch) (Chapter 2)

$\varepsilon_{s,FAT}$ net tensile strain in the extreme tension steel for the fatigue analysis of reinforcement (inch/inch)

ε_{SH} UHPC shrinkage strain at a given time (inch/inch)

$\varepsilon_{s,SER}$ net tensile strain in the extreme tension steel for the service limit case (inch/inch)

$\varepsilon_{s,STR}$ net tensile strain in the extreme tension steel for the strength limit case (inch/inch)

$\varepsilon_{sa,STR}$ allowable net tensile strain in extreme tension steel for the strength limit case (inch/inch)

ε_{t} net tensile strain in extreme tension fiber of the UHPC section (inch/inch)

$\varepsilon_{ta,SER}$ allowable net tensile strain in extreme tension fiber of the UHPC section for the service limit case (inch/inch)

$\varepsilon_{ta,STR}$ allowable net tensile strain in extreme tension fiber of the UHPC section for the strength limit case (inch/inch)

$\varepsilon_{t,cr}$ elastic tensile strain limit of UHPC corresponding to a tensile stress of $\gamma f_{t,cr}$ (inch/inch)

$\varepsilon_{t,cyc}$ net tensile strain in extreme tension fiber of the UHPC section for the cyclic analysis of UHPC (inch/inch)

$\varepsilon_{t,FAT}$ net tensile strain in extreme tension fiber of the UHPC section for the fatigue analysis of reinforcement (inch/inch)

$\varepsilon_{t,loc}$ crack localization strain of UHPC for use in design (inch/inch)

$\varepsilon_{t,SER}$ net tensile strain in extreme tension fiber of the UHPC section for the service limit case (inch/inch)

$\varepsilon_{t,STR}$ net tensile strain in extreme tension fiber of the UHPC section for the strength limit case (inch/inch)

ε_{TU} strain due to thermal load (inch/inch)

ε_y yield strain for the reinforcement (inch/inch)

θ_1 girder end rotation in Span 1 (rad)

θ_2 girder end rotation in Span 2 (rad)

θ_{DW} rotation due to wearing surface over link slab (rad)

θ_{FAT} rotation for fatigue limit case (rad)

θ_{fat} rotation due to fatigue load (rad)

θ_{LL} rotation due to live load (rad)

θ_{SER} rotation for service limit case (rad)

θ_{STR} rotation for strength limit case (rad)

θ_T total rotation in link slab (rad)

σ_{allow} allowable service stress (ksi)

σ_{DL} bearing stress due to dead load (ksi)

σ_{LL} bearing stress due to live load (ksi)

$\sigma_{SER\ I}$ bearing stress due to Service I load (ksi)

$\sigma_{s,max}$ maximum service bearing stress (ksi)

ϕ_w hollow column reduction factor as defined in Article 5.6.4.7.2c of AASHTO LRFD BDS

ψ_{avg} average curvature in debond zone length of link slab caused by girder end rotation (rad/in)

ψ_{FAT} average curvature in debond zone length of link slab caused by girder end rotation due to Fatigue I load (rad/in)

ψ_{SER} average curvature in debond zone length of link slab caused by girder end rotation due to Service I load (rad/in)

ψ_{STR} average curvature in debond zone length of link slab caused by girder end rotation due to Strength I load (rad/in)

CHAPTER 1. INTRODUCTION

INTRODUCTION

This report explains the fundamental concepts behind and provides the design calculations for an ultra-high performance concrete (UHPC) link slab. In this design example, an existing four-span, steel simple-span composite bridge containing traditional expansion joints is used as the base structure. The example steps through the process of designing link slabs that will replace the three interior expansion joints on the bridge. The redesign of the bearings is also included. This report supports the Federal Highway Administration's (FHWA) Every Day Counts (EDC) innovation titled *UHPC for Bridge Preservation and Repair*.

DESIGN CRITERIA

This example references the following specifications:

- AASHTO Load and Resistance Factor (LRFD) Bridge Design Specifications, 9th Edition, 2020; referred to hereafter as "AASHTO LRFD BDS".
- AASHTO Guide Specification for Structural Design with Ultra-High Performance Concrete, 2023; referred to hereafter as "UHPC Guide".

This example also references recommendations for link slab design made in:

- Haber, Z.B., Foden, A., McDonagh, M., Ocel, J.M., Zmetra, K. and Graybeal, B.A.. 2022. Design and Construction of UHPC-Based Bridge Preservation and Repair Solutions (No. FHWA-HRT-22-065). Washington, DC: Federal Highway Administration. Office of Infrastructure Research and Development. Referred to hereafter as FHWA-HRT-22-065.

Specific recommendations for link slab design are made in FHWA-HRT-22-065 Chapter 4.

The original design (1969) for the existing bridge to which the UHPC link slab example is based on was based on HS20-44 design loading. The UHPC link slab design presented herein is based on HL-93 loading as described in AASHTO LRFD BDS (2020).

GENERAL PROCEDURE FOR LINK SLAB DESIGN

A general procedure for the design of UHPC link slabs is provided in FHWA-HRT-22-065. UHPC link slabs are designed for service, strength, and fatigue limit states.

Primary Assumption: The primary assumption made for link slab design is that the center of rotation at the end of the girder moves from the bearing level, where it resides prior to link slab installation, to the centroid of the UHPC link slab as shown in Figure 1. Therefore, the link slab only experiences rotation, not axial demand from girder flexure. Given this assumption, the bearings need to be designed to accommodate traditional demands as well as axial deformations caused by girder end rotation.

The primary design parameters for UHPC link slabs include:

- Link slab thickness (h),
- Debonded length (L_{DB}), and
- Longitudinal reinforcement (A_s).

The design of UHPC link slabs typically requires a local design of the link slab itself (Chapter 3 of this report), global design of the structure to determine the effect of link slab installation on the behavior of the bridge as a system (Chapter 4), and design of bearings (Chapter 5).

The basic design steps for the local design of a link slab include:

1. Calculate girder end rotations from loads applied after link slab installation (e.g., live load, future wearing surface) for each limit state (Service I, Strength I, and Fatigue I). Assume simply supported behavior when calculating deflections and associated girder end rotations.
2. Assume initial values for link slab design parameters: h, L_{DB}, and A_s.
3. Calculate strains and stresses in link slab caused by girder end rotations for each limit state. Assume that the girder end rotation occurs uniformly over the length of the debonded length, i.e., $\psi = \theta / L_{DB}$, to find strains across the section depth.
4. Compare calculated strains and stresses against strain and stress limits specified in UHPC Guide.
5. Modify the link slab design parameters as needed until calculated strains and stresses are less than specified limits.

The installation of link slab(s) will affect the global behavior of the bridge as the link slab(s) create connectivity across the superstructure at the deck level that would not have been present with conventional expansion joints. The horizontal or transverse forces acting on the existing substructure and foundation elements after the installation of link slabs are found through a structural analysis and should include the redistribution of braking, wind, and seismic loads, and potential changes in thermal restraint.

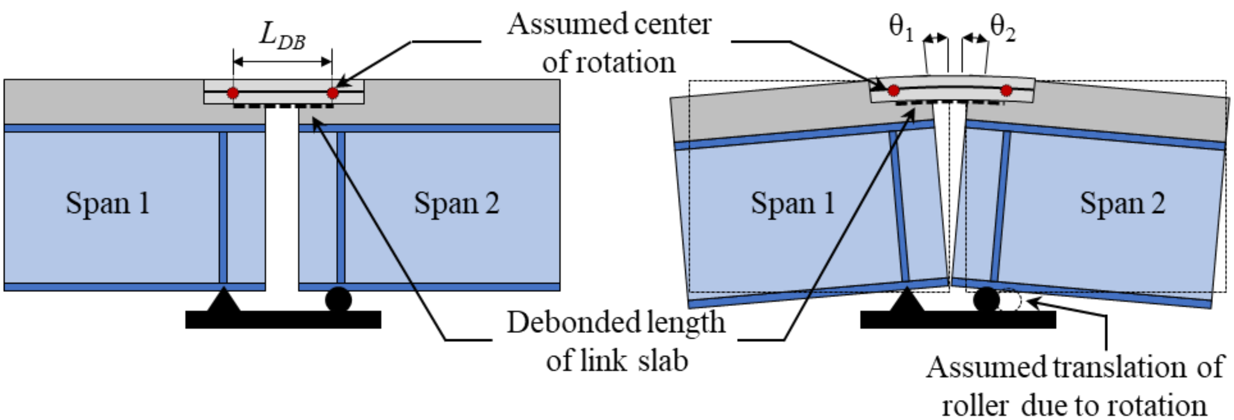
Bearings also need to be redesigned as part of a link slab installation. Elastomeric bearings are preferred for link slabs as they can better withstand repetitive horizontal movements from girder translation and rotation.

Local Demand for Link Slabs

A link slab is designed locally to resist girder end rotation that is caused by load effects after the link slab is cast (live load, wearing surface, and thermal and time-dependent effects (e.g., shrinkage)). Only loads applied after the placement and curing of the UHPC in the link slab should be considered when calculating the demand on the link slab. This example steps through the link slab design considering girder end rotations due to live load and wearing surface. Details on how to also consider thermal and time effects are provided in the Appendix. Strains due to thermal effects were found to be minor in this example; however, for longer span structures these effects may be more significant. Strains due to shrinkage of the UHPC lead to a longer required link slab; using a lower-shrinkage UHPC mixture is generally considered to be advantageous for link slabs.

Girder End Rotation

Girder end rotations are found using conventional structural analysis techniques for the simple span on each side of the link slab, as shown in Figure 1. The center of rotation is assumed to be in the link slab approximately at the end of the debonded length, which results in negligible deformations occurring in the link slab from girder flexure. Rather, the rotation will result in a translation of the roller support. Note that, similar to rollers, elastomeric bearings allow for horizontal translation; the shear resistance provided by an elastomeric bearing is neglected.



Source: FHWA.

Figure 1. Illustration. Assumed rotation demand from each span and debonded length for link slab.

The girder end rotation in this example is calculated based on the relationship between end rotation and maximum deflection for simply supported beams found using Bernoulli beam theory for simply supported beams.

For simply-supported prismatic beams with distributed loads: $\theta = (16 \times \Delta_{mid}) / (5 \times Span)$

For simply-supported prismatic beams with point loads: $\theta = (3 \times \Delta_{mid}) / Span$

Given that the predicted demand for simply-supported beams with distributed loads is larger, this relationship is conservatively used in this design example.

The total rotation demand for the link slab is the summation of the rotation from each span.

Total rotation for design of link slab (service and strength limit states): $\theta_T = \theta_1 + \theta_2$

AASHTO LRFD BDS Article 3.6.1.3.1 states that “90 percent of the effect of two design trucks… combined with 90 percent of the effect of the design lane load” be used for finding "negative moment between points of contraflexure under a uniform load on all spans". This does not apply to link slabs as they are assumed to only provide continuity for live load within the deck between spans, and the intent of this provision is negative load redistribution of the superstructure elements. For this reason, link slabs should be designed assuming maximum loading on Span 1 and Span 2 independent of each other (i.e., calculating girder end rotations θ_1 and θ_2). In cases where the spans are equal length and have the same superstructure elements (like this example), the total rotation for slab design is simply two times the girder end rotation for one of the spans.

Total rotation for design of link slabs with similar adjacent span details (service and strength limit states): $\theta_T = 2\theta_1 = 2\theta_2$

Per AASHTO LRFD BDS Article 3.6.1.4, the fatigue load shall be one design truck with a constant spacing of 30.0 ft between the 32.0-kip axles. Because it is a single design truck, the rotation demand on a link slab for fatigue should only be for one span. The total rotation used for design of the link slab for the fatigue limit state can be taken as the maximum value of girder end rotation from the two spans as caused by the fatigue load.

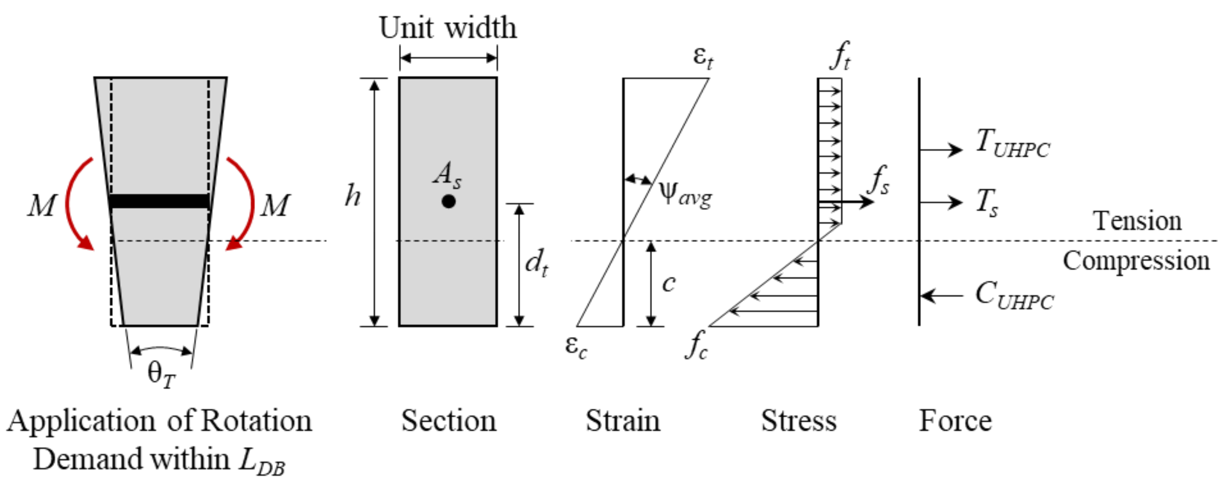
Total rotation for design of link slab (fatigue limit state): $\theta_T = \max(\theta_1, \theta_2)$

Average Curvature, Strains, and Stresses in Debonded Length

The girder end rotation is assumed to be applied uniformly over the debonded length, shown in Figure 1, which results in the curvature equal to the rotation divided by the debonded length.

Average curvature for design of link slab: $\psi_{avg} = \theta_T / L_{DB}$

The average curvature caused by the girder end rotation will result in a strain variation over the depth of the section, as shown in Figure 2.



Source: FHWA.

Figure 2. Illustration. Strains, stresses, and forces associated with assumed rotation demand.

The strain at the extreme tension and compression fibers of the section and at the level of the reinforcement can be found based on the curvature and the neutral axis depth. The following relationships ignore strains caused by shrinkage and thermal effects.

Strain at extreme compression fiber: $\varepsilon_c = c \times \psi_{avg}$

Strain at extreme tension fiber: $\varepsilon_t = (c - h) \times \psi_{avg}$

Strain at level of reinforcement: $\varepsilon_s = (c - d_t) \times \psi_{avg}$

The stress in the UHPC and in the reinforcement can be found using the assumed stress-strain relationships. The forces in each component can be found based on stresses integrated over the respective areas.

The curvature is known based on the girder end rotations, so the neutral axis depth *c* can be varied until equilibrium is satisfied.

Iterate *c* until $\sum C = \sum T$

Link slab design diverges from conventional reinforced concrete or prestressed concrete member design at this point. For link slabs, the strains and stresses found for a given girder end rotation are checked versus appropriate strain and stress limits (based on the limit state being investigated). For conventional members, the moment capacity would be compared to the demand moment.

Shrinkage Strains in Link Slab

A link slab is restrained on each end by its connection to the deck and girder. Thus, shrinkage that occurs in the link slab may result in shrinkage-induced stresses. A prediction of the shrinkage in the UHPC can be calculated using UHPC Guide Article 1.4.2.8.3-1.

Strain due to shrinkage for UHPC: $\varepsilon_{SH} = k_s \times k_{hs} \times k_f \times k_{td} \times K_4 \times 0.6 \times 10^{-3}$

The full shrinkage strain found using this equation would not necessarily lead to additional shrinkage-induced stresses in the link slab for several reasons.

- A significant portion of shrinkage strains occur during the initial hydration of the UHPC when it has a smaller modulus of elasticity.
- The UHPC may exhibit tension creep relaxation when subjected to sustained tensile stress from shrinkage.
- The UHPC link slab may crack which can relieve a portion of the shrinkage-induced stresses.

It would be unnecessarily conservative to include the full shrinkage strain in the design calculations by subtracting ε_{SH} from the allowable tensile strain in the UHPC.

The appropriate load factor should be applied to the shrinkage strain if it is included in the design. The force effect due to shrinkage (*SH*) is defined as a permanent load in AASHTO LRFD BDS Article 3.3.2. The load factors for the three limit states of interest are as follows:

- Service I: 1.00*SH*
- Strength I: 1.00*SH*
- Fatigue I: not a consideration

The load factor for *SH* for Strength I is found in AASHTO LRFD BDS Table 3.4.1-3. The relevant load factor is "Concrete Superstructure – nonsegmental."

This calculation is shown in the Appendix and was found to be more than half of the UHPC tensile strain limit in this example. If included in the design calculation, the shrinkage strain would lead to a debonded length (L_{DB}) twice as long as when the shrinkage strain is ignored.

Strains Due to Thermal Movement

The thermal load (F_{TU}) on the pier can be found based on the assumed thermal range (ΔT), coefficient of thermal expansion, distance between fixed piers, and the longitudinal stiffness of the pier (k_{pier}). The thermal load is calculated in Chapter 4. The strain caused by the thermal movement can be found based by taking the thermal load divided by the modulus of elasticity of the conventional concrete deck and the transformed area of the composite girder.

Strain due to thermal movement: $\varepsilon_{TU} = F_{TU} / [E_c \times (A_c + A_{st})]$

The strain due to thermal movement can be included in the same manner as shrinkage strain, by subtracting ε_{TU} from the allowable tensile strain in the UHPC.

The appropriate load factor should be applied to the strain due to thermal movement. Two load factor values are provided in AASHTO LRFD BDS Table 3.4.2-1 for force effects due to uniform temperature (*TU*). The larger of these two values shall be used for deformations and the smaller for all other effects. Strain due to thermal movement is deformation-related, so it is recommended to use the larger value.

- Service I: 1.20*TU*
- Strength I: 1.20*TU*
- Fatigue I: not a consideration

The strain due to thermal movement is calculated in the Appendix. It is determined to be negligible for this design example.

Limit States and Strain and Stress Limits for Link Slab Design

The strain and stress limits for design of link slabs are based on the limit state being investigated. Link slabs should be designed for Service, Fatigue, and Strength Limit States.

Service Limit State:

The Service I load combination (AASHTO LRFD BDS Table 3.4.1-1) is used for these strain and stress checks.

Design girder rotation for Service I: $\theta_{SER} = \theta_{LL} + \theta_{DW}$

A link slab is a nonprestressed component, so provisions in UHPC Guide Article 1.5.2.2 apply.

- Allowable tension strain in UHPC (Article 1.5.2.2.2): $\varepsilon_{ta,SER} = min\ (0.25\varepsilon_{t,loc}, 0.001)$
- Allowable compression stress in UHPC (Article 1.5.2.2.3): $f_{ca,SER} = 0.60\phi_w \times f'_c$
- Allowable reinforcing steel stress (Article 1.5.2.2.5): $f_{sa,SER} = 0.8f_y$

An additional design check is required for components subjected to cyclic tensile stresses (UHPC Guide Article 1.5.2.3).

- Allowable tensile stress in UHPC for components subjected to cyclic stresses (Article 1.5.2.3): $f_{ta,SER} = 0.95\gamma_u f_{t,cr}$

This additional requirement often controls the design of nonprestressed components as it essentially requires components to remain uncracked during service. By design, link slabs are designed to crack and designing around these requirements would lead to a much longer debond length (five times as long in this example as shown in the Appendix). It is recommended that this cyclic tensile stress check not be required for link slab design.

Strength Limit State:

The Strength I load combination (AASHTO LRFD BDS Table 3.4.1-1) is used for these strain and stress checks. The girder rotation due to live load and wearing surface is calculated first. Assuming linear elastic behavior, the rotations caused by live load and the application of the wearing surface can be factored and added. However, if the strength limit state controls the final design, it is suggested that this linear elastic assumption be verified.

Design girder rotation for Strength I: $\theta_{STR} = 1.75\theta_{LL} + 1.5\theta_{DW}$

As previously discussed, checking strains and stresses under ultimate loads is not common for traditional concrete members. For this reason, some of the provisions in the UHPC Guide are presented here in the form of stress and strain checks. There are two additional recommendations from FHWA-HRT-22-065 for link slab design:

- The maximum tensile strain in UHPC should be limited to the localization strain of the UHPC.
- All other materials in the link slab should remain linear-elastic at the ultimate limit state.

The allowable strains and stresses for the Strength Limit State from the UHPC Guide include:

- Allowable tension strain in UHPC (Article 1.6.3.2.2): $\varepsilon_{ta,STR} = \gamma_u\varepsilon_{t,loc}$
- Allowable compression strain in UHPC (Article 1.6.3.2.2): $\varepsilon_{ca,STR} = \varepsilon_{cu}$

- Allowable strain in reinforcing steel (Article 1.6.3.2.2): $\varepsilon_{sa,STR} = \varepsilon_{su}$

The recommendations that other materials be kept linear elastic (other than maximum tensile strain in UHPC) from FHWA-HRT-22-065 place stricter requirements on the allowable compression strain in UHPC and the allowable stress in the reinforcement.

- Allowable compression strain in UHPC (based on recommendations from FHWA-HRT-22-065): $\varepsilon_{ca,STR} = \varepsilon_{cp}$
- Allowable reinforcing steel stress (based on recommendations from FHWA-HRT-22-065): $f_{sa,STR} = f_y$

These limits need to be checked under the Strength I load combination, but generally are less likely to control the design than the Service Limit State checks.

Fatigue Limit State:

The Fatigue I load combination (AASHTO LRFD BDS Table 3.4.1-1) is used for these strain and stress checks.

Design girder rotation for Fatigue I: $\theta_{FAT} = 1.75\theta_{fat}$

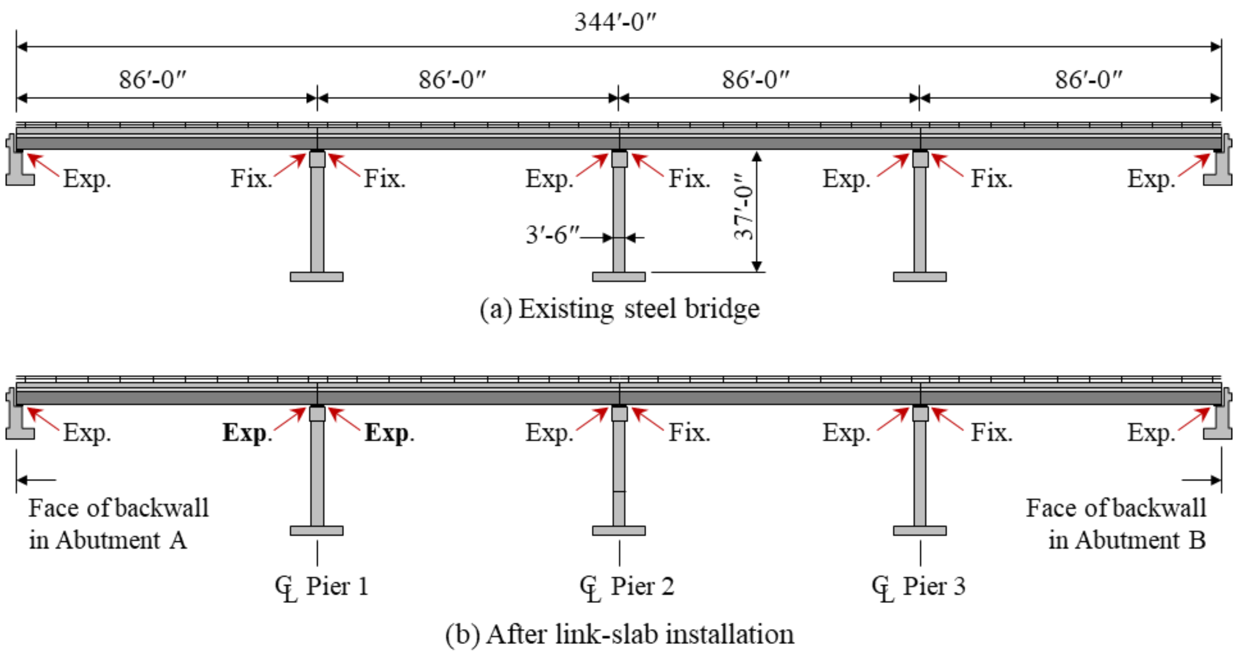
UHPC Guide Article 1.5.3 requires that discrete steel elements embedded in UHPC be checked for fatigue in accordance with AASHTO LRFD BDS Article 5.5.3. There is also a fatigue check for the compressive strength due to Fatigue I loading.

- Allowable tensile stress in reinforcement, defined as the constant amplitude fatigue threshold ΔF_{TH} in AASHTO LRFD BDS Article 5.5.3.2: $\Delta F_{TH} = 26 - 22 \times (f_{min} / f_y)$
- Allowable compression stress in UHPC Guide Article 1.5.3: $f_{ca,FAT} = 0.40\, f'_c$

CHAPTER 2. PROBLEM STATEMENT AND MATERIAL PROPERTIES

INTRODUCTION

This example problem demonstrates the design of a UHPC link slab to be installed in an existing four-span steel bridge with composite deck originally designed and constructed in 1969. The bridge is located in northcentral North Carolina (for wind speed calculations). An illustration of the elevation perspective of the existing steel bridge is shown in Figure 3 (a). The bridge consists of four 86-foot-long girders with an overall length of 344 feet. The pier height is 37 feet, and the diameter of the cylindrical pier is 3 foot 6 inches.

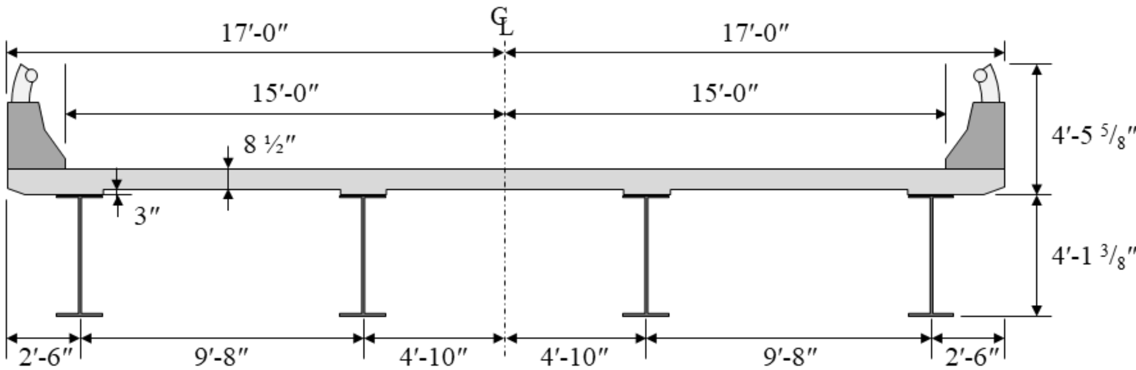


Source: FHWA.

Figure 3. Illustration. Elevation perspective of bridge (a) existing steel bridge and (b) bearing configuration after link slab installation.

Link slabs will be designed to go over Piers 1, 2, and 3. Because of the current configuration is four simple spans, Pier 1 has fixed bearings for both spans sitting atop it. Fixed-fixed bearings will not work for link slabs because it will put the link slab under axial loading, Thus, with deck continuity provided by the link slabs, the bearings will have to be reconfigured for Pier 1 as shown in Figure 3 (b). Fixed-expansion bearings are located at Pier 2 and Pier 3. The selection of which locations retain fixed bearings along with the deck continuity of the link slab will affect the distribution of thermal, wind, seismic, and braking forces and the substructure will also require investigation if it can tolerate the new load. Likewise, thermal length will increase, and bearings and expansion joints at the abutments will have to be checked and or redesigned for this effect.

The typical section for the bridge is shown in Figure 4. The typical section consists of four 49 3/8-inch-deep steel girders spaced at 9 foot 8 inches with an 8.5-inch-thick composite conventional concrete deck. There is a 3-inch haunch between the top of the steel girder and bottom of the conventional concrete deck. The distance between the inside face of the railing to the centerline of the structure is 15 feet.



Source: FHWA.
Note: There is no wearing surface in the current configuration.

Figure 4. Illustration. Typical section.

STRUCTURE GEOMETRY

Details on the structural geometry, including the future wearing surface that will be considered in the design, are summarized below.

Bridge length: *Length* = 344 ft + 0 inch

Span length: *Span* = 86 ft + 0 inch

Bridge width: B = 34 ft

Future wearing surface over link slab thickness: t_{ws} = 2 inch

Deck thickness: t_d = 8.5 inch

MATERIAL PROPERTIES

The material properties used in this example are summarized in this section.

Conventional Reinforcement

The reinforcement used in the link slab has the following properties:

Reinforcing steel elastic modulus: E_s = 29000 ksi

Reinforcing steel yield strength: f_y = 60 ksi

UHPC Material Properties

Based on the UHPC Guide, UHPC must have the following minimum properties (Article 1.1.1):

- Minimum compressive strength (f'_c): 17.5 ksi
- Minimum effective cracking strength ($f_{t,cr}$): 0.75 ksi
- Minimum crack localization strength ($f_{t,loc}$): $f_{t,loc} \geq f_{t,cr}$
- Minimum crack localization strain ($\varepsilon_{t,loc}$): 0.0025

The material properties for the UHPC used in this example are summarized below:

UHPC compressive strength: $f'_c = 18$ ksi

UHPC effective cracking strength: $f_{t,cr} = 0.75$ ksi

UHPC localization strength: $f_{t,loc} = 0.75$ ksi

UHPC crack localization strain: $\varepsilon_{t,loc} = 0.0025$

In practice, it is anticipated that the UHPC material properties would be based on the reported properties of prequalified products (as provided by UHPC vendors) or known properties of local non-commercialized UHPC mixtures. The properties would be verified through standardized quality control testing during the construction of the link slabs. The compressive strength would be assessed based on test cylinders produced, tested, and evaluated in accordance with ASTM C1856/C1856M (2017). The tensile properties would be assessed based on AASHTO T 397 (2022).

The elastic modulus for the UHPC can be measured via standardized test method or can be calculated using the relationship in UHPC Guide Article 1.4.2.3-1. In this design example, the elastic modulus is calculated based on the predictive relationship with an assumed elastic modulus correction factor (K_1) of 1.0.

UHPC compressive elastic modulus: $E_c = 2{,}500\, K_1 \times f'^{\,0.33}_c = 6{,}489$ ksi

UHPC Compression Behavior (UHPC Guide Article 1.4.2.4)

The compression behavior of the UHPC is idealized as elastic-plastic with a maximum compressive stress equal to $\alpha_u f'_c$, as discussed in UHPC Guide Article 1.4.2.4. A reduction factor for compressive stress-strain non-linearity (α_u) of 0.85 will be used in this example.

Reduction factor for compressive stress-strain non-linearity: $\alpha_u = 0.85$

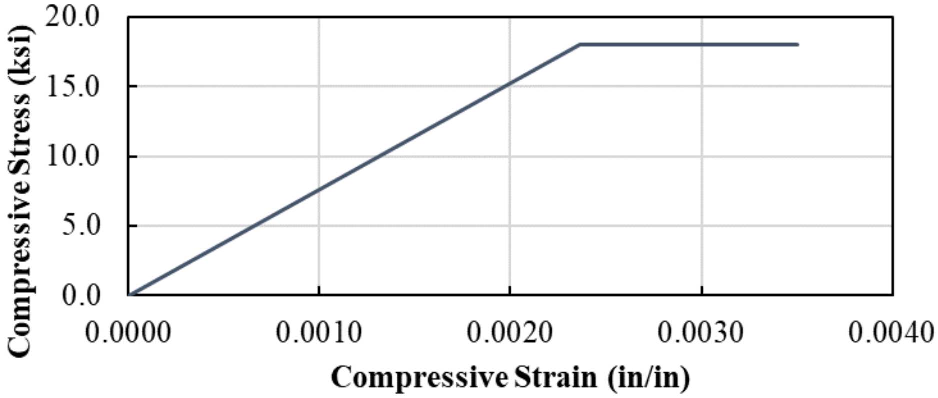
The UHPC is assumed to be linear elastic with a slope equal to the elastic modulus up to the elastic compressive strain limit (ε_{cp}), which is found using this equation with a minimum allowable value of 0.0035 (UHPC Guide Article 1.4.2.4.2).

$\varepsilon_{cp} = (\alpha_u \times f'_c) / E_c = 0.002360$

The ultimate compressive strain (ε_{cu}) may be determined through physical testing or as assumed to be the maximum of 0.0035 or the elastic compressive strain limit.

UHPC ultimate compressive strain: $\varepsilon_{cu} = (\varepsilon_{cp} \geq 0.0035) = 0.0035$

The stress-strain relationship for UHPC in compression is thus defined through these three points (strain, stress): (0, 0), (ε_{cp}, $\alpha_u f'_c$), and (ε_{cu}, $\alpha_u f'_c$). The compressive stress-strain relationship for the UHPC used in this example problem is shown in Figure 5.



Source: FHWA.

Figure 5. Graph. UHPC compressive stress-strain relationship.

UHPC Tension Behavior (UHPC Guide Article 1.4.2.5)

The tension behavior of the UHPC is idealized as elastic-plastic or bilinear response depending on the ratio of the localization stress to the cracking stress. If UHPC localization stress is less than 120 percent of effective cracking strength, the UHPC localization stress equals the effective cracking strength, $f_{t,cr}$ (UHPC Guide Article 1.4.2.5.2). The localization stress is equal to the cracking stress in this example, so an elastic-plastic model is used.

The cracking stress, localization stress, and localization strain are multiplied by a factor (γ_u) to account for potential reduced tensile resistance within fabricated structural elements as compared to qualification test specimens. This factor shall not be taken as larger than 1.0 (UHPC Guide Article 1.4.2.5.4). A value of 1.0 is selected for this example.

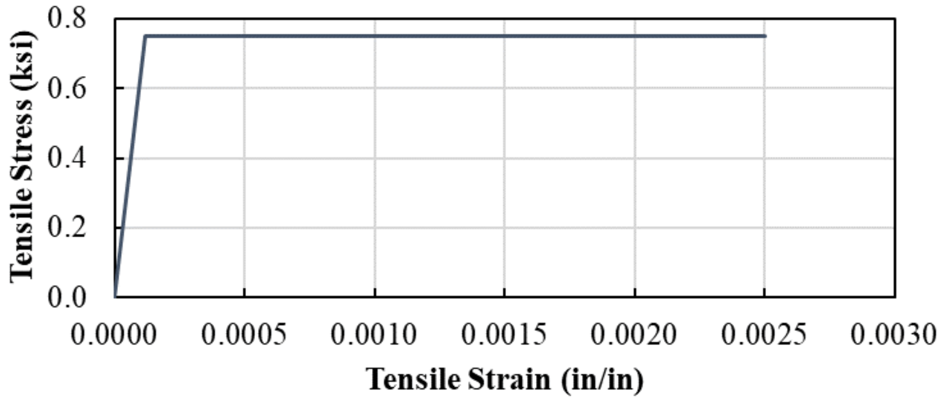
Factor on tensile mechanical properties: $\gamma_u = 1.0$

The UHPC is assumed to be linear elastic with a slope equal to the elastic modulus up to the elastic tensile strain limit ($\varepsilon_{t,cr}$), which is found using UHPC Guide Eq. 1.4.2.5.4-1:

$$\varepsilon_{t,cr} = (\gamma_u \times f_{t,cr}) / E_c = 0.000116$$

The assumed tensile response then remains plastic until the crack localization strain ($\varepsilon_{t,loc}$) is reached. The crack localization strain ($\varepsilon_{t,loc}$ = 0.0025) and stress ($f_{t,loc}$ = 0.75 ksi) were set at the start of this design example.

The stress-strain relationship for UHPC in tension is defined with these three points (strain, stress): (0, 0), ($\varepsilon_{t,cr}$, $\gamma_u f_{t,cr}$), and ($\gamma_u \varepsilon_{t,loc}$, $\gamma_u f_{t,loc}$). The tensile stress-strain relationship for the UHPC used in this example problem is shown in Figure 6.



Source: FHWA.

Figure 6. Graph. UHPC tensile stress-strain relationship.

Conventional Concrete (Piers)

Basic mechanical properties for the concrete used in the piers is needed for the calculation of the longitudinal pier stiffness. The concrete strength and elastic modulus used in this example are provided below.

Concrete compressive strength: f'_c = 4 ksi

Concrete modulus of elasticity (AASHTO LRFD BDS Article C5.4.2.4-3): $E_c = 1{,}820\sqrt{f'_c}$ = 3,640 ksi

CHAPTER 3. LINK SLAB DESIGN (LOCAL BEHAVIOR)

INTRODUCTION

Calculations for the detailed design of a UHPC link slab for an existing four-span, steel simple-span composite structure are provided in this chapter. The calculations in this chapter follow the general procedure for link slab design outlined in Chapter 1.

GIRDER END ROTATION

The maximum live load deflection (Δ_{LL}) was found from a 2D grillage model using HL-93 loading on each span, as defined in AASHTO LRFD BDS Article 3.6.1. This deflection is the same for each span because the spans are equal length with the same superstructure components. The maximum deflection from the four girders is used in the design.

Maximum live load deflection on each span: Δ_{LL} = 0.695 inch

As previously discussed in Chapter 1, girder end rotation is found based on the relationship between end rotation and maximum deflection for simply supported beams found using Bernoulli beam theory for simply supported beams with a distributed load.

Girder end rotation due to live load: $\theta_{LL} = (16 \times \Delta_{LL}) / (5 \times Span) = 0.0022$ rad

The relationship for a distributed load is used in this example because it represents the most severe case, engaging the HL-93 live load model with a uniform lane load and point loads for the design truck axles. Alternatively, the girder end rotation could be extracted from the structural analysis model in lieu of the simplified assumption above.

The midspan deflection due to the future wearing surface was from a 2D grillage model by applying the distributed load from the wearing surface based on its thickness (t_{ws} = 2 inch) and density (0.140 kcf for bituminous wearing surfaces from AASHTO LRFD BDS Table 3.5.1.1). This deflection is the same for each span because the spans are equal length with the same superstructure components.

Midspan deflection from wearing surface: Δ_{DW} = 0.188 inch

The rotation can again be calculated based on the midspan deflection, just as with the live load.

Girder end rotation due to wearing surface: $\theta_{DW} = (16 \times \Delta_{DW}) / (5 \times Span) = 0.0006$ rad

The application of the fatigue load, as defined in AASHTO LRFD BDS Article 3.6.1.4, results in the following midspan deflection as determined through a 2D grillage model.

Fatigue load deflection: Δ_{fat} = 0.356 inch

The girder end rotation caused by the application of the fatigue load is calculated as follows.

Girder end rotation due to fatigue load: $\theta_{fat} = (16 \times \Delta_{fat}) / (5 \times Span) = 0.0011$ rad

The appropriate load factors from AASHTO LRFD BDS Article 3.4.1 are applied to combine these three rotation components for each limit state of interest.

Service Limit State (Service I): $\theta_{SER} = \theta_{LL} + \theta_{DW} = 0.0027$ rad

Strength Limit State (Strength I): $\theta_{STR} = 1.75\theta_{LL} + 1.5\theta_{DW} = 0.0046$ rad

Fatigue Limit State (Fatigue I): $\theta_{FAT} = 1.75\theta_{fat} = 0.0019$ rad

The total rotation for design of the link slab will depend on the limit state being evaluated. The maximum deflections in adjacent spans are considered simultaneously for the Service and Strength Limit States, while only one design truck is applied at a time for the Fatigue Limit State.

Total design rotation for Service Limit State: $2\theta_{SER}$

Total design rotation for Strength Limit State: $2\theta_{STR}$

Total design rotation for Fatigue Limit State: θ_{FAT}

These rotation demands are used in the evaluation of the link slab design.

LINK SLAB DIMENSIONS

The link slab thickness (h), debond zone length (L_{DB}), longitudinal reinforcement density (A_s), and depth to tensile reinforcement (d_t) need to be selected to perform the design check. The initial values selected for this example are the following.

Link slab thickness: $h = 4$ inch

Debond zone length: $L_{DB} = 24$ inch

Longitudinal reinforcement density, #5 at 12 inches: $A_s = 0.31$ inch^2/ft

Depth to tensile reinforcement: $d_t = h - 2$ inch = 2 inch

These parameters can be modified if the design does not meet one of the design checks. The Appendix shows how the debond zone length would need to be increased in certain scenarios.

The typical length of a UHPC link slab is between 2 and 4 feet, as summarized by FHWA-HIF-20-062. The debond zone length is directly related to the span length, i.e., longer spans may require longer debond zone lengths.

SERVICE LIMIT STATE

Determine Stress and Strain in UHPC and Reinforcement for the Service Limit Case

The first series of checks is for the service limit state. The girder end rotation for the Service I load combination and the selected debond zone length are the following.

Design girder end rotation (Service I): $\theta_{SER} = 0.0027$ rad

Debond zone length: $L_{DB} = 24$ inch

The girder end rotation is assumed to be applied uniformly over the debond zone length. The end of each girder is assumed to rotate θ_{SER}, so the total rotation in the link slab is $2\theta_{SER}$. This leads to the following average curvature in the link slab over the debond zone length.

Average curvature in link slab (Service I): $\psi_{SER} = 2\theta_{SER} / L_{DB} = 2.25 \times 10^{-4}$ rad/inch

The neutral axis depth associated with this curvature can be found using the iterative procedure discussed in Chapter 1. A tolerance may need to be defined depending on what solver or programming loop is being used. This solution was originally developed using a "while" loop and the tolerance to exit the while loop was defined as *tol*.

Neutral axis depth error tolerance: $tol = 0.0001$ inch

The calculations for the first iteration for solving for the neutral axis depth are shown here. Additional iterations are performed until tension and compression forces are equal.

Initial assumed neutral axis depth: $c = 0.3h = 1.2$ inch

The strain in the extreme compression fiber can be found by taking the average curvature in the debond zone length times the neutral axis depth.

Strain at extreme compression fiber: $\varepsilon_c = c \times \psi_{SER} = 0.000274$

The resultant compression force can be found based on the strain at the extreme compression fiber and the assumed stress-strain relationship for the UHPC in compression. The resultant compression force (C) per unit width will be equal to the area of the compression stress versus depth diagram. The stress diagram on the compression side of the element will remain linear elastic (i.e., the stress diagram will be triangular) as long as the compression strain at the extreme compression fiber is less than the elastic compressive strain limit (ε_{cp}), as shown in Figure 2. Otherwise, the stress diagram on the compression side will have a trapezoidal shape, varying linearly from zero at the neutral axis to $\alpha_u f'_c$ at the elastic compressive strain limit and then remaining at a constant stress for the rest of the section depth. This calculation can be programmed with an if-then statement.

If $\varepsilon_c \leq \varepsilon_{cp}$, then $C = 0.5E_c \times \varepsilon_c \times c$, else $C = \alpha_u f'_c \times c \times (1 - 0.5 \times (\varepsilon_{cp} / \varepsilon_c))$.

This calculation results in C = 12.79 kip/ft.

No reinforcement is present on the compression side of the link slab, so the compression force in the UHPC will be the only compression force present.

There are two tension forces in the link slab with the assumed neutral axis: (1) tension force in the reinforcement and (2) tension force in the UHPC. Each tension force can be calculated from the strain and stress diagrams for the link slab section. The strain at the level of the reinforcement can be calculated based on the curvature and the distance between the neutral axis and the centroid of the reinforcement.

Strain at reinforcement (first iteration): $\varepsilon_s = (d_t - c) \times \psi_{SER} = 0.000183$

The reinforcement is linear elastic at this strain level ($\varepsilon_s \leq \varepsilon_y$), so the stress can be calculated by taking the strain times the modulus of elasticity of steel.

Reinforcement stress (first iteration): $f_s = E_s \times \varepsilon_s = 5.29$ ksi

The last force component in the link slab section is the tension force in the UHPC. The tension force in UHPC is based on the strain on the cross section and the stress-strain relationship for the UHPC. The strain at the extreme tension fiber is calculated as follows.

Strain at extreme tension fiber (first iteration): $\varepsilon_t = (h - c) \times \psi_{SER} = 0.000639$

This strain is greater than the elastic tensile strain limit ($\varepsilon_{t,cr}$ = 0.000116), but less than the factored crack localization strain ($\gamma_u \varepsilon_{t,loc}$ = 0.0025). In a general case, the stress in the extreme tension fiber can be found by linearly interpolation between the stress-strain pairs at the cracking and localization points as shown in the equation below. In this example, the UHPC tension behavior is elastic-plastic; therefore, the tension stress is equal to both the factored cracking stress and the strain localization stress.

$$f_{t,max} = \gamma_u f_{t,cr} + \gamma_u \left(f_{t,loc} - f_{t,cr}\right) \times \frac{\varepsilon_t - \varepsilon_{t,cr}}{\gamma_u \varepsilon_{t,loc} - \varepsilon_{t,cr}} = 0.75 \text{ ksi}$$

The distance from the neutral axis to the elastic tensile strain limit, is needed to find the area of the triangular and rectangular portions of the tensile stress diagram. This distance is calculated as follows.

$d_{t,cr} = (\varepsilon_{t,cr} / \varepsilon_c) \times c = 0.507$ inches

The resultant tension force is the summation of the reinforcement and UHPC components.

Resultant tension force (first iteration):

$T = A_s f_s + 0.5d_{t,cr} \times \gamma_u f_{t,cr} + (h - c - d_{t,cr}) \gamma_u f_{t,cr} + 0.5(h - c - d_{t,cr}) \gamma_u (f_{t,max} - f_{t,cr}) =$ 24.56 kip/ft

The last term of this force equation is equal to 0 as $f_{t,max}$ is equal to $f_{t,cr}$ in this example.

The resultant tension force is not equal to the resultant compression force for this assumed neutral axis depth.

Equilibrium check (first iteration): absolute value of $(T - C) = 11.77$ kip/ft $\neq 0$ kip/ft

A new assumed neutral axis depth can be calculated based on this equation.

$c_{it} = 0.5(c + c \times (T + C) / (2C)) = 1.48$ inches

The error between the initially assumed neutral axis depth and the new assumed neutral axis depth can be found and compared to the predetermined tolerance. These calculations will be repeated until the neutral axis depth error is less than the tolerance. The next iteration will be performed using the new assumed neutral axis depth (c_{it}) from the previous iteration.

Through this iterative process, the neutral axis depth was determined to be the following value.

Neutral axis depth: $c_{SER} = c = 1.53$ inch

The strain, stress, and force components can be calculated using this neutral axis to verify the analysis and calculate the strain and stress values required for the Service Limit State design checks.

Strain at extreme compression fiber (Service I): $\varepsilon_{c,SER} = c_{SER} \times \psi_{SER} = 0.00035$

Stress at extreme compression fiber: $f_{c,SER} = \min(E_c \times \varepsilon_{c,SER}, \alpha_u f'_c) = 2.27$ ksi

The resultant compression force is then calculated to be $C_{SER} = 20.88$ kip/ft.

The strain and stress in the reinforcement under Service I loading can be calculated using the correct neutral axis depth.

Strain at reinforcement (Service I): $\varepsilon_{s,SER} = (d_t - c_{SER}) \times \psi_{SER} = 0.000107$

Reinforcement stress (Service I): $f_{s,SER} = E_s \times \varepsilon_{s,SER} = 3.09$ ksi

The strain in the extreme tension fiber of the UHPC can be calculated as follows.

Strain at extreme tension fiber (Service I): $\varepsilon_{t,SER} = (h - c_{SER}) \times \psi_{SER} = 0.000563$

At this strain, the stress in the extreme tensile fiber of the UHPC is 0.75 ksi.

The distance to elastic tensile strain limit from the neutral axis is calculated as follows.

$d_{t,cr,SER} = (\varepsilon_{t,cr} / \varepsilon_{c,SER}) \times c_{SER} = 0.51$ inches

The resultant tension force is the summation of the reinforcement and UHPC components.

Resultant tension force (Service I):

$$T_{SER} = A_s f_s + 0.5d_{t,cr,SER} \times \gamma_u f_{t,cr} + (h - c_{SER} - d_{t,cr,SER}) \gamma_u f_{t,cr} + 0.5(h - c_{SER} - d_{t,cr,SER}) \gamma_u (f_{t,max} - f_{t,cr}) = 20.88 \text{ kip/ft}$$

The compression resultant force equals the tension resultant force, so the calculation results have been verified to provide equilibrium for this neutral axis and curvature.

Check equilibrium (Service I): absolute value of ($T_{SER} - C_{SER}$) = 0 kip/ft

Compare Stresses and Strains to Limits, Service Limit State (UHPC Guide Article 1.5.2)

The strains and stresses calculated in the preceding analysis are compared to allowable stress and strains in the UHPC and reinforcement under Service I loading. There are three Service Limit State design checks for UHPC link slabs. The hollow column reduction factor is found in AASHTO LRFD BDS Article 5.6.4.7.2c equal to 1.0 for this solid link slab (ϕ_w = 1.0).

Allowable UHPC tension strain: $\varepsilon_{ta,SER} = (0.25\varepsilon_{t,loc} \leq 0.001) = 0.000625$

Allowable UHPC compression stress due to permanent and transient loads:
$f_{ca,SER} = 0.60\phi_w \times f'_c = 10.8$ ksi

Allowable reinforcing steel stress: $f_{sa,SER} = 0.8f_y = 48$ ksi

The last design check is the allowable tensile stress in UHPC components subjected to cyclic stresses.

Allowable tensile stress in UHPC for components subjected to cyclic stresses:
$f_{ta,SER} = 0.95\gamma_u f_{t,cr} = 0.7125$ ksi

A summary of the calculated strains and stresses in the UHPC and reinforcement in comparison to the allowable design values is provided in Table 1.

Table 1. Actual versus allowable strains and stresses in link slab under Service I loading.

Design Check	Allowable Value	Service I Value	Check?
UHPC tension strain	$\varepsilon_{ta,SER}$ = 0.000625	$\varepsilon_{t,SER}$ = 0.000563	OK
UHPC compression stress	$f_{ca,SER}$ = 10.8 ksi	$f_{c,SER}$ = 2.27 ksi	OK
Reinforcing steel stress	$f_{sa,SER}$ = 48 ksi	$f_{s,SER}$ = 3.09 ksi	OK
UHPC tension stress (cyclic)	$f_{ta,SER}$ = 0.7125 ksi	$f_{t,max,SER}$ = 0.75 ksi	No Good

As described in Chapter 1, the intent of the UHPC Guide for the cyclic tensile stress check is to ensure no cracking under cyclic stress. However, this concept is not necessarily applicable to a component such as a link slab which is intended to deform to alleviate other specific deformations in the structure; thus, this stress check is recommended to be ignored. Calculations provided in the Appendix show the effect if the UHPC Guide is strictly followed; the cyclic stress tension limit would control and the debonded length would have to increase to over six times longer.

The strains and stresses in the cross section can also be plotted across the section depth and compared to strain and stress limits graphically. The points needed for plotting stress and strain over the section depth depend on if the UHPC remains elastic in the compression region. If the UHPC in compression remains elastic, i.e., if $\varepsilon_c \leq \varepsilon_{cp}$, then the stress, strain, and deflection under service loads can be plotted using the equations in Table 2.

Table 2. Comparing service limit case stress and strain if $\varepsilon_c \leq \varepsilon_{cp}$.

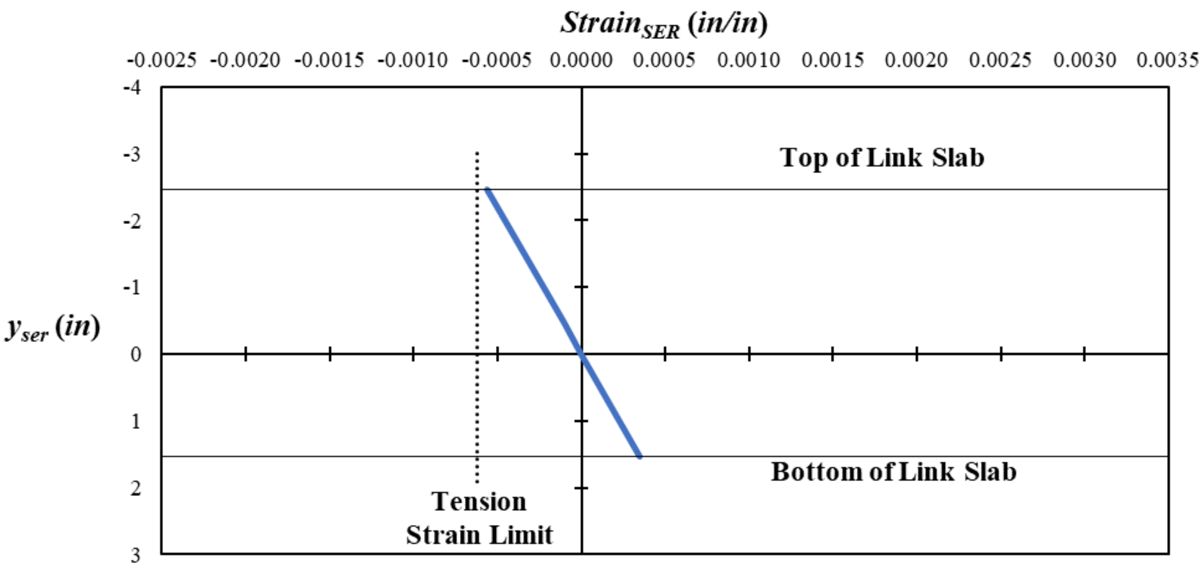
Point	$Stress_{SER}$ (x-axis)	$Strain_{SER}$ (x-axis)	y_{SER} (y-axis)
1	$E_c \times \varepsilon_{c,SER}$	$\varepsilon_{c,SER}$	c_{SER}
2	0	0	0
3	$-f_{t,cr}$	$-\varepsilon_{t,cr}$	$-d_{t,cr,SER}$
4	$-f_{t,cr}$	$-\varepsilon_{t,SER}$	$c_{SER} - h$

Otherwise, if $\varepsilon_c > \varepsilon_{cp}$, then the stress, strain, and deflection under service loads can be plotted using the equations in Table 3.

Table 3. Comparing service limit case stress and strain if $\varepsilon_c > \varepsilon_{cp}$.

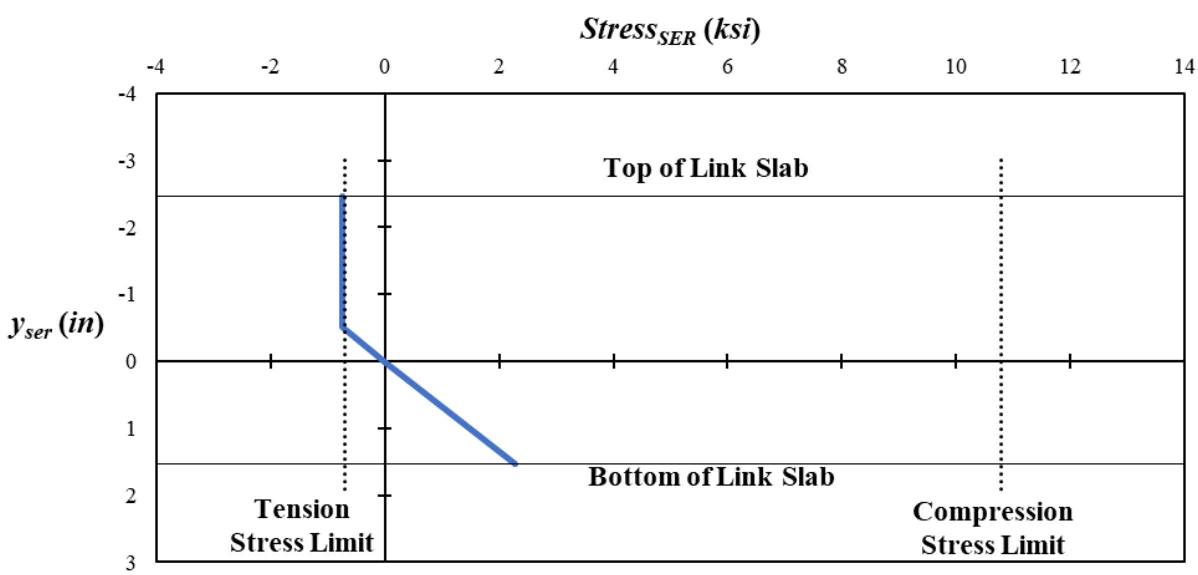
Point	$Stress_{SER}$ (x-axis)	$Strain_{SER}$ (x-axis)	y_{SER} (y-axis)
1	$E_c \times \varepsilon_{c,SER}$	$\varepsilon_{c,SER}$	c_{SER}
2	$E_c \times \varepsilon_{c,SER}$	ε_{cp}	$c_{SER} \times \varepsilon_{cp} / \varepsilon_{c,SER}$
3	0	0	0
4	$-f_{t,cr}$	$-\varepsilon_{t,cr}$	$-d_{t,cr,SER}$
5	$-f_{t,cr}$	$-\varepsilon_{t,SER}$	$c_{SER} - h$

The actual strain and actual stress in the UHPC link slab under Service I loading are plotted across the depth of the section in Figure 7 and Figure 8, respectively. The allowable limits on tensile and compressive strain and the allowable compressive stress are shown.



Source: FHWA.

Figure 7. Graph. Beam strain at service limit state.



Source: FHWA.

Figure 8. Graph. Beam stress at service limit state.

STRENGTH LIMIT STATE

Determine Stress and Strain in UHPC and Reinforcement, Strength Limit Case

The process for finding the strains and stresses across the depth of the UHPC link slab under Strength I loading is the same as the process followed above for Service I loading. The design girder end rotation for the Strength I load combination and the selected debond zone length are the following.

Design girder end rotation (Strength I): $\theta_{STR} = 0.0046$ rad

Debond zone length (same as above): $L_{DB} = 24$ inch

Like before, the girder end rotation is assumed to be applied uniformly over the debond zone length. The end of each girder is assumed to rotate θ_{STR}, so the total rotation in the link slab is $2\theta_{STR}$. This calculation leads to the following average curvature in the link slab over the debond zone length.

Average curvature in link slab (Strength I): $\psi_{STR} = 2\theta_{STR} / L_{DB} = 3.83 \times 10^{-4}$ rad/inch

The neutral axis depth, strains, and stresses in the UHPC link slab associated with this curvature can be determined through the same iterative procedure used for the Service Limit State. After a series of iterations, the following neutral axis depth was determined.

Neutral axis depth: $c_{STR} = 1.3$ inch

This neutral axis can be used to find the strains, stresses, and force components in the UHPC link slab under Strength I loading.

Strain at extreme compression fiber (Strength I): $\varepsilon_{c,STR} = c_{STR} \times \psi_{STR} = 0.000503$

Stress at extreme compression fiber: $f_{c,STR} = (E_c \times \varepsilon_{c,STR} \leq \alpha_u f'_c) = 3.26$ ksi

The resultant compression force can then be calculated as $C_{STR} = 25.41$ kip/ft.

The strain and stress in the reinforcement under Strength I loading can subsequently be calculated.

Strain at reinforcement (Strength I): $\varepsilon_{s,STR} = (d_t - c_{STR}) \times \psi_{STR} = 0.000272$

Reinforcement stress (Strength I): $f_{s,STR} = E_s \times \varepsilon_{s,STR} = 7.88$ ksi

The strain in the extreme tension fiber of the UHPC can then be calculated.

Strain at extreme tension fiber (Strength I): $\varepsilon_{t,STR} = (h - c_{STR}) \times \psi_{STR} = 0.00105$

This again falls between the elastic tensile strain limit ($\varepsilon_{t,cr} = 0.000116$) and the factored crack localization strain ($\gamma_u \varepsilon_{t,loc} = 0.0025$), so the stress in the extreme tension fiber is 0.75 ksi.

The distance from the neutral axis to the depth in the cross section exhibiting the elastic tensile strain limit is then calculated.

$d_{t,cr,STR} = (\varepsilon_{t,cr} / \varepsilon_{c,STR}) \times c_{STR} = 0.3$ inches

The resultant tension force is the summation of the reinforcement and UHPC components.

Resultant tension force (Strength I):

$$T_{STR} = A_s \times f_{s,STR} + 0.5d_{t,cr,STR} \times \gamma_u f_{t,cr} + (h - c_{STR} - d_{t,cr,STR})\gamma_u f_{t,cr} + 0.5(h - c_{STR} - d_{t,cr,STR})\gamma_u (f_{t,max,STR} - f_{t,cr})$$

$$= 25.41 \text{ kip/ft}$$

The compression resultant force equals the tension resultant force, so the calculation results have been verified to provide equilibrium for this neutral axis and curvature.

Compare Stresses and Strains to Limits, Strength Limit State (UHPC Guide Article 1.6.3 and Recommendations from FHWA-HRT-22-065)

The strains and stresses determined through the preceding analysis are compared to allowable stress and strains in the UHPC and reinforcement under Strength I loading. There are three Strength Limit State design checks for UHPC link slabs.

Allowable tension strain in UHPC Guide Article 1.6.3.2.2: $\varepsilon_{ta,STR} = \gamma_u \varepsilon_{t,loc} = 0.0025$

Allowable compression strain in UHPC (based on recommendations from FHWA-HRT-22-065): $\varepsilon_{ca,STR} = \varepsilon_{cp} = 0.00236$

Allowable reinforcing steel strain (based on recommendations from FHWA-HRT-22-065): $\varepsilon_{sa,STR} = \varepsilon_y = f_y / E_s = 0.00207$

The calculated strains and stresses in the UHPC and reinforcement are compared to the allowable design values in Table 4.

Table 4. Actual versus allowable strains and stresses in link slab under Strength I loading.

Design Check	Allowable Value	Strength I Value	Check?
UHPC tension strain	$\varepsilon_{ta,STR} = 0.002500$	$\varepsilon_{t,STR} = 0.000105$	OK
UHPC compression strain	$\varepsilon_{ca,STR} = 0.002360$	$\varepsilon_{c,STR} = 0.000503$	OK
Reinforcing steel strain	$\varepsilon_{sa,STR} = 0.002070$	$\varepsilon_{s,STR} = 0.000272$	OK

The strains and stresses in the section can also be plotted across the section depth and compared to strain and stress limits graphically. The points needed for plotting stress and strain over the section depth depend on if the UHPC remains elastic in the compression region. If the UHPC in compression remains elastic, i.e., if $\varepsilon_c \leq \varepsilon_{cp}$, then the stress, strain, and deflection under service loads can be plotted using the equations in Table 5.

Table 5. Comparing strength limit case stress and strain if $\varepsilon_c \leq \varepsilon_{cp}$.

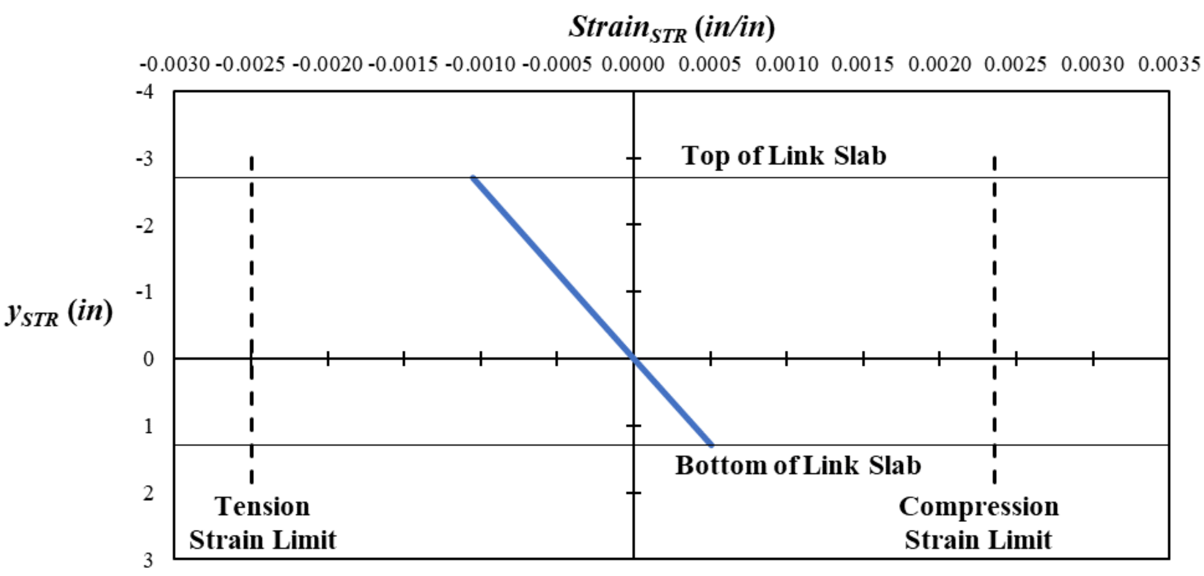
Point	***Stress$_{STR}$*** **(x-axis)**	***Strain$_{STR}$*** **(x-axis)**	***y_{STR}*** **(y-axis)**
1	$E_c \times \varepsilon_{c,STR}$	$\varepsilon_{c,STR}$	c_{STR}
2	0	0	0
3	$-f_{t,cr}$	$-\varepsilon_{t,cr}$	$-d_{t,cr,STR}$
4	$-f_{t,cr}$	$-\varepsilon_{t,STR}$	$c_{STR} - h$

Otherwise, if $\varepsilon_c > \varepsilon_{cp}$, then the stress, strain, and deflection under service loads can be plotted using the equations in Table 6.

Table 6. Comparing strength limit case stress and strain if $\varepsilon_c > \varepsilon_{cp}$.

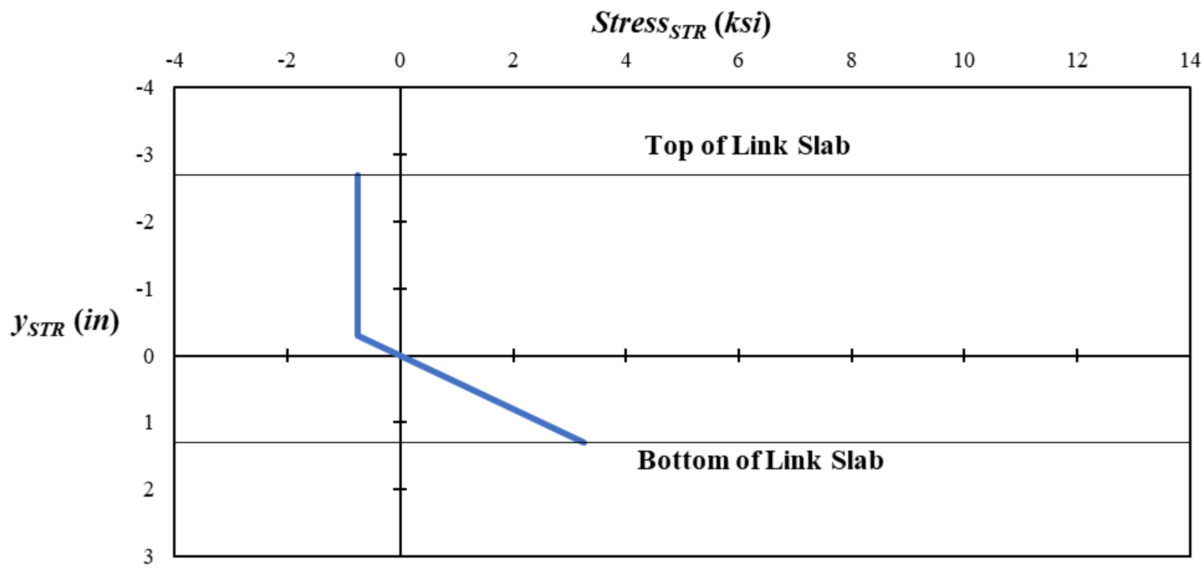
Point	***Stress$_{STR}$*** **(x-axis)**	***Strain$_{STR}$*** **(x-axis)**	***y_{STR}*** **(y-axis)**
1	$E_c \times \varepsilon_{c,STR}$	$\varepsilon_{c,STR}$	c_{STR}
2	$E_c \times \varepsilon_{c,STR}$	ε_{cp}	$c_{STR} \times \varepsilon_{cp} / \varepsilon_{c,STR}$
3	0	0	0
4	$-f_{t,cr}$	$-\varepsilon_{t,cr}$	$-d_{t,cr,STR}$
5	$-f_{t,cr}$	$-\varepsilon_{t,STR}$	$c_{STR} - h$

The actual strain and stress in the UHPC link slab under Strength I loading are plotted across the depth of the section in Figure 9 and Figure 10. The allowable limits on tensile and compressive strain are shown.



Source: FHWA.

Figure 9. Graph. Beam strain at strength limit state.



Source: FHWA.

Figure 10. Graph. Beam stress at strength limit state.

FATIGUE LIMIT STATE

Determine Stress and Strain in UHPC and Reinforcement for the Fatigue Limit Case

The process for finding the strains and stresses across the depth of the UHPC link slab under Fatigue I loading (UHPC Guide Article 1.5.3, AASHTO LRFD BDS Article 5.5.3) is the same as the process followed above for Service I and Strength I loading. The design girder end rotation for the Fatigue I load combination and the selected debond zone length are the following.

Design girder end rotation (Fatigue I): $\theta_{FAT} = 0.0019$ rad

Debond zone length (same as above): $L_{DB} = 24$ inch

Like before, the girder end rotation is assumed to be applied uniformly over the debond zone length. Only one design truck is applied at a time for the Fatigue Limit State, so the average curvature in the link slab over the debond zone length is the following.

Average curvature in link slab (Fatigue I): $\psi_{FAT} = \theta_{FAT} / L_{DB} = 7.92 \times 10^{-5}$ rad/inch

The neutral axis depth, strains, and stresses in the UHPC link slab associated with this curvature can be determined through the same iterative process used for the Service and Strength Limit States. After a series of iterations, the following neutral axis depth was determined.

Neutral axis depth: $c_{FAT} = c = 1.95$ inch

This neutral axis can be used to find the strains, stresses, and force components in the UHPC link slab under Fatigue I loading.

Strain at extreme compression fiber (Fatigue I): $\varepsilon_{c,FAT} = c_{FAT} \times \psi_{FAT} = 0.000157$

Stress at extreme compression fiber: $f_{c,FAT} = (E_c \times \varepsilon_{c,FAT} \leq \alpha_u f'_c) = 1.02$ ksi

The resultant compression force is then determined to be $C_{FAT} = 11.98$ kip/ft.

The strain and stress in the reinforcement under Fatigue I loading can be calculated like before using the correct neutral axis depth.

Strain at reinforcement (Fatigue I): $\varepsilon_{s,FAT} = (d_t - c_{FAT}) \times \psi_{FAT} = 0.00000363$

Reinforcement stress (Fatigue I): $f_{s,FAT} = E_s \times \varepsilon_{s,FAT} = 0.11$ ksi

The strain in the extreme tension fiber of the UHPC is then calculated.

Strain at extreme tension fiber (Fatigue I): $\varepsilon_{t,FAT} = (h - c_{FAT}) \times \psi_{FAT} = 0.000165$

This value again falls between the elastic tensile strain limit ($\varepsilon_{t,cr} = 0.000116$) and the factored crack localization strain ($\gamma_u \varepsilon_{t,loc} = 0.0025$), so the stress in the extreme tension fiber is 0.75 ksi.

The distance from the neutral axis to the depth in the cross section exhibiting the elastic tensile strain limit is then calculated.

$d_{t,cr,FAT} = (\varepsilon_{t,cr} / \varepsilon_{c,FAT}) \times c_{FAT} = 1.44$ inches

The resultant tension force is the summation of the reinforcement and UHPC components.

Resultant tension force (Fatigue I):

$$T_{FAT} = A_s f_{s,FAT} + 0.5 d_{t,cr,FAT} \times \gamma_u f_{t,cr} + (h - c_{FAT} - d_{t,cr,FAT}) \gamma_u f_{t,cr} + 0.5(h - c_{FAT} - d_{t,cr,FAT}) \gamma_u (f_{t,max,FAT} - f_{t,cr})$$

$$= 11.98 \text{ kip/ft}$$

The compression resultant force equals the tension resultant force, so the calculation results have been verified to provide equilibrium for this neutral axis and curvature.

Compare Stresses and Strains to Limits, Fatigue Limit State

The strains and stresses determined through the preceding analysis are compared to allowable stress and strains in the UHPC and reinforcement under Fatigue I loading.

There are two Fatigue Limit State design checks for UHPC link slabs. The minimum stress in the reinforcement in the fatigue stress range (f_{min}) includes stress from shrinkage strains. The shrinkage strain in the UHPC is found in the Appendix.

Strain due to shrinkage: $\varepsilon_{SH} = 0.6 \times 10^{-3} \times k_s \times k_{hs} \times k_f \times k_{td} \times K_4 = 0.000360$

Stress due to shrinkage: $f_{SH} = \varepsilon_{SH} \times E_c = 2.34$ ksi

Minimum stress in reinforcement in fatigue stress range: $f_{min} = f_{SH} = 2.34$ ksi

Allowable tensile stress in reinforcement, defined as the constant amplitude fatigue threshold ΔF_{TH} in AASHTO LRFD BDS Article 5.5.3.2: $f_{sa,FAT} = \Delta F_{TH} = 26 - 22 \times (f_{min} / f_y) = 25.1$ ksi

Allowable compression stress in UHPC Guide Article 1.5.3: $f_{ca,FAT} = 0.40\, f'_c = 7.2$ ksi

The calculated strains and stresses in the UHPC and reinforcement are compared to the allowable design values in Table 7.

Table 7. Actual versus allowable strains and stresses in link slab under Fatigue I loading.

Design Check	Allowable Value	Fatigue I Value	Check?
UHPC compression stress	$f_{ca,FAT} = 7.2$ ksi	$f_{c,FAT} = 1.02$ ksi	OK
Reinforcing steel stress	$f_{sa,FAT} = 25.1$ ksi	$f_{s,FAT} = 0.11$ ksi	OK

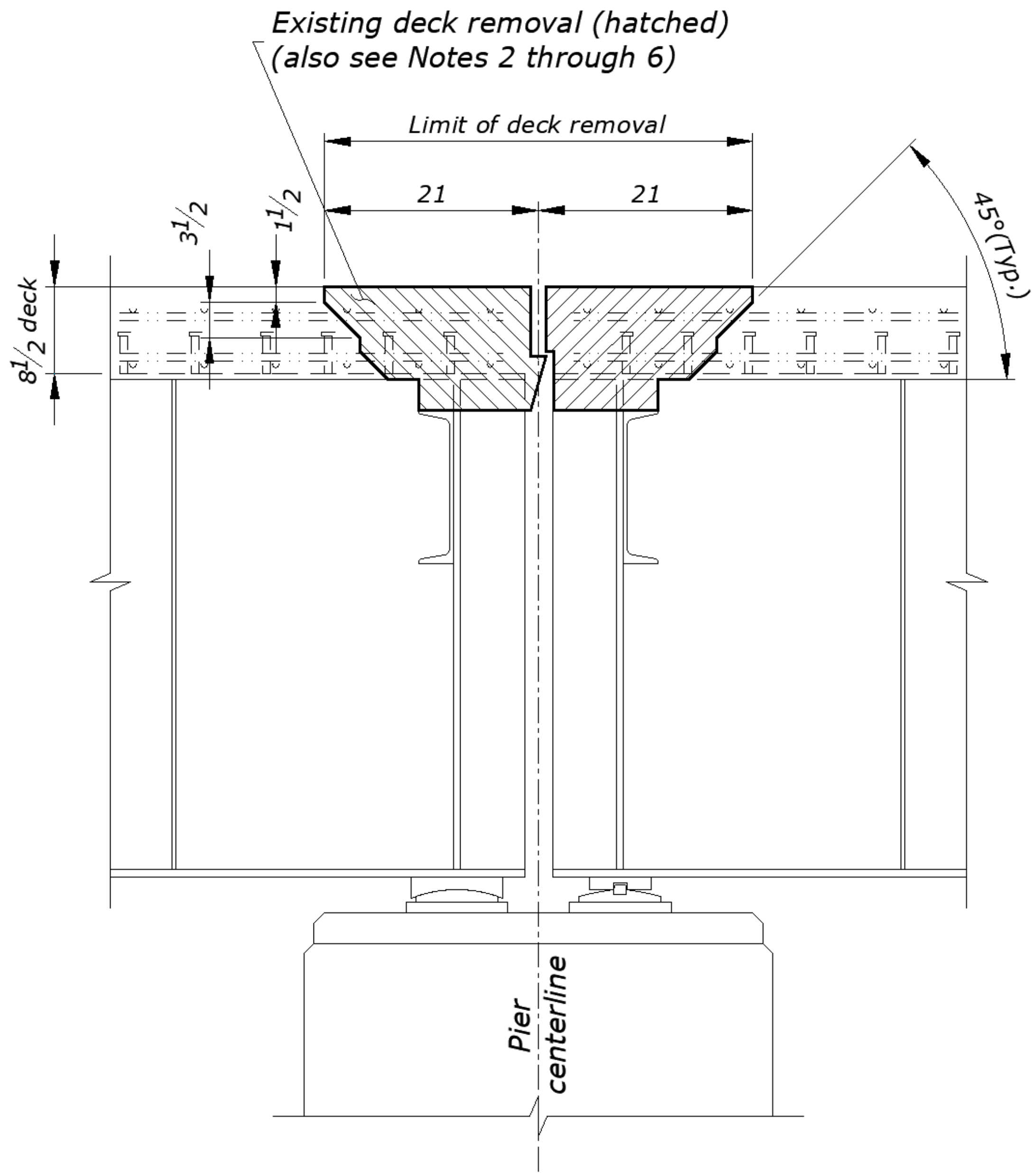
The stresses under Fatigue I loading are less than the specified limits.

FINAL DESIGN DETAILS FOR LINK SLAB

An illustration of the existing expansion joint section over an interior support is shown in Figure 11. The amount of deck that is to be removed as part of the UHPC link slab construction process is shown as 1 foot 9 inches from the end of each girder. The transverse deck reinforcement is removed from the deck removal region.

The longitudinal bars extending from the deck are left in place after the deck removal; these bars have a 7-inch center-to-center spacing. Where existing bars are found to be damaged removal of existing deck concrete, Number 5 bars should be installed by drilling and grouting the bars centered between the existing deck bars. Non-shrink grout conforming to ASTM C1107 should be used.

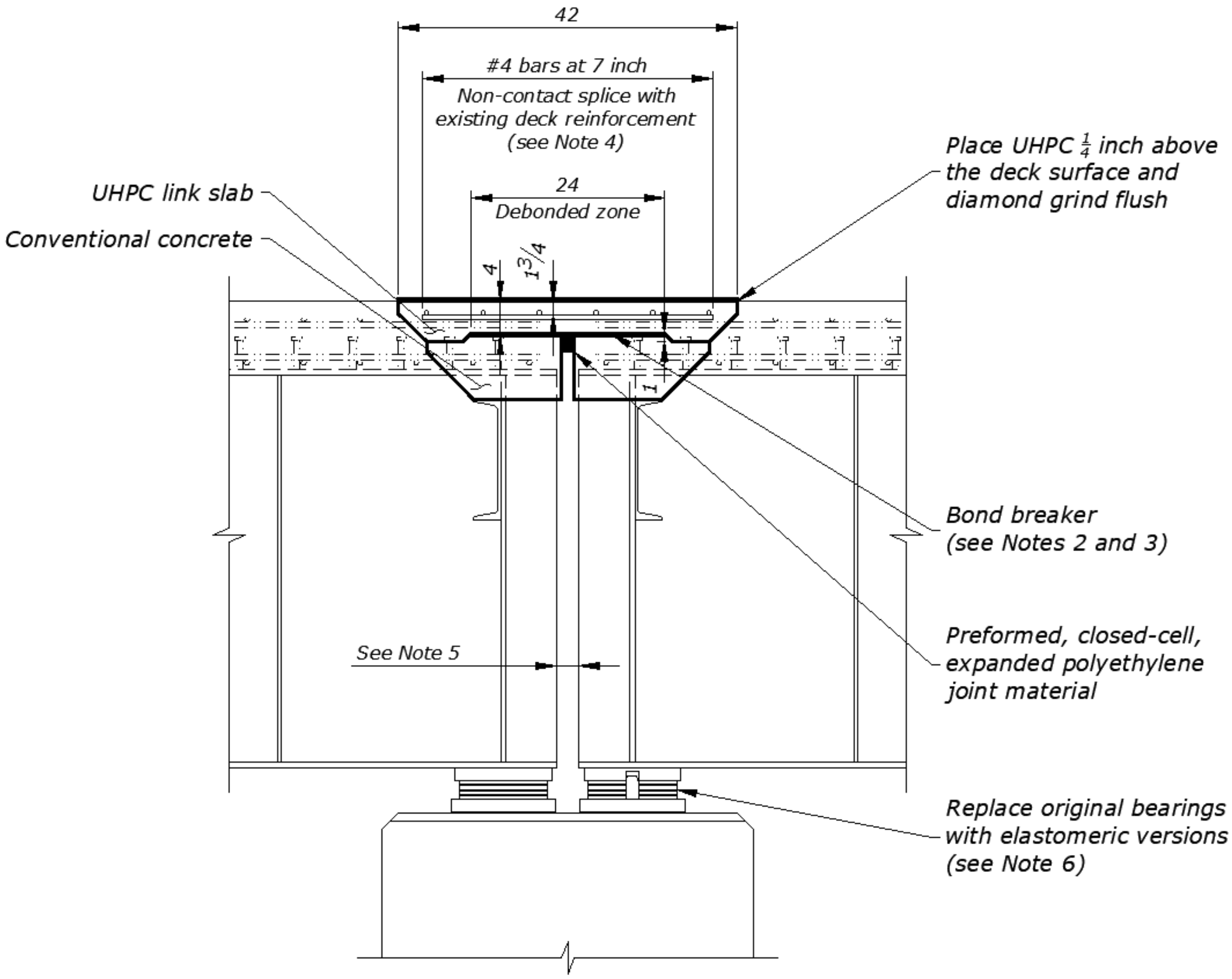
Note 2 in Figure 11 refers to the removal and installation of shear studs. Existing shear studs that interfere with the debonded zone of the UHPC link slab should be removed. Additional shear stud connectors are installed outside the debonded zone underneath the link slab such that the maximum spacing between each shear stud connector does not exceed 5 inches. Shear stud connectors are installed to match the transverse spacing of the existing connectors.



Source: FHWA.
Note 1: Units are in inches.
Note 2: Remove existing joint material.
Note 3: Remove transverse bars within the limit of deck removal.
Note 4: Retain longitudinal bars in the limit of deck removal.
Note 5: Remove and replace shear connectors with shorter versions in the limit of deck removal if they interfere with the future link slab.
Note 6: Retain rough surface of approximate $^{1}/_{8}$ to $^{1}/_{4}$ inch amplitude at interface between existing concrete and future link slab.

Figure 11. Illustration. UHPC link slab bridge deck joint and support pier, existing section.

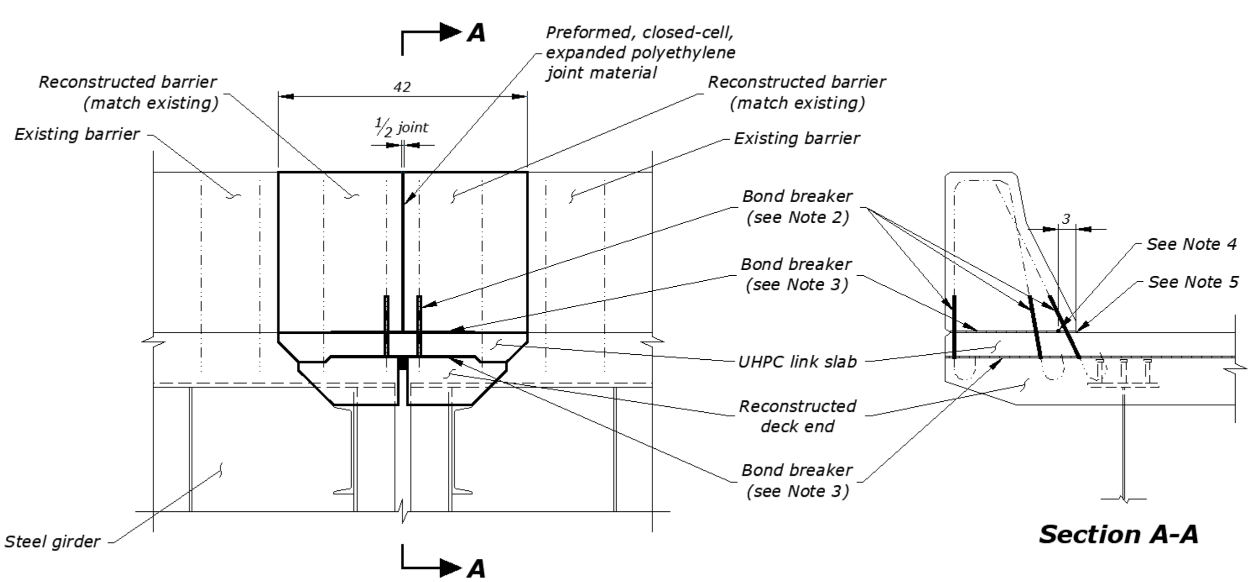
Details for the proposed UHPC link slab section are shown in Figure 12. The UHPC portion of the link slab has a 5-inch thickness at the ends of the link slab and a 4-inch thickness through the debonded zone in the middle. The UHPC for the link slab is cast on top of a reconstructed portion of the conventional concrete deck.



Source: FHWA.
Note 1: Units are in inches.
Note 2: Bond breaker can be 1/16-inch-thick compressed synthetic sheet gasket treated on both sides with a parting agent or use 15-pound felt paper conforming to ASTM D226 Type 1.
Note 3: Trowel finish the conventional concrete in the debonded zone.
Note 4: Do not splice longitudinal reinforcement in the debonded zone. Provide a clear spacing between bars in non-contact splices of at least 1.25 inches, but no greater than 4 inches.
Note 5: Provide a minimum end gap 1.5 inches between adjacent spans. Verify the end gap width before pouring the link slab.
Note 6: Before installation of the proposed bearings, install temporary blocking to ensure global stability of the entire superstructure system. Submit the temporary blocking procedure for approval before removing existing bearings.

Figure 12. Illustration. UHPC link slab bridge deck joint and support pier, proposed section.

A relief joint is provided in the barrier over the link slab, as shown in Figure 13. Barrier anchorage reinforcement that extends out of the UHPC link slab within the debonded zone should be debonded (as discussed in Note 7 in Figure 13); this reinforcement should be debonded 6 inches above the top of the UHPC link slab and down to the bottom of the link slab. A bond breaker should also be installed between the bottom of the concrete barrier and top of UHPC link slab within the debonded zone.



Source: FHWA.
Note 1: Units are in inches.
Note 2: Debond all barrier anchorage reinforcement that extends out of the UHPC link slab within the debonded zone. Debond the reinforcement 6 inches above and throughout the UHPC link slab.
Note 3: Bond breaker can be 1/16-inch-thick compressed synthetic sheet gasket treated on both sides with a parting agent or use 15-pound felt paper conforming to ASTM D226 Type 1.
Note 4: Hydrophilic caulk or strip. Placed prior to bond breaker.
Note 5: Apply sealant after construction is complete. Sealant shall conform to ASTM C920, Type S, Grade NS, Class 50.

Figure 13. Illustration. Crash barrier detailing to decouple from link slab.

CHAPTER 4. GLOBAL BEHAVIOR OF BRIDGE WITH LINK SLABS

INTRODUCTION

The addition of link slabs may affect the global behavior of the bridge because link slabs create connectivity across the superstructure that would not have been present with conventional expansion joints. The horizontal or transverse forces acting on the existing substructure and foundation elements after the installation of link slabs are found through a structural analysis like that introduced in this chapter. This analysis includes:

- Redistribution of braking,
- Wind and seismic loads, and
- Potential changes in thermal restraint.

A simplified analysis is provided in this chapter. More sophisticated analyses, such as finite element modeling of the structure and bearings to analyze the demands within the structure, may be warranted in some cases.

The simplifying assumptions used in this example are as follows:

- All three piers are identical and have equal stiffness.
- The superstructure is substantially stiffer than the piers.
- The abutments are rigid.

Due to the above assumptions, horizontal loading is distributed equally among impacted piers, leading to a simplified case that can be analyzed without sophisticated modeling.

The as-built drawings used for this simplified analysis are shown in Chapter 2.

Pier 2 and Pier 3 are assumed to be fixed in this example.

Number of fixed piers: $n_{fix} = 2$

The "thermal center" is assumed to be at midspan between Pier 2 and Pier 3.

PIER GEOMETRY

The amount of thermal load that is transferred into the piers is dependent on the pier stiffness. The basic properties for the piers are summarized in this section.

Pier height: $L_{col} = 37$ ft

Column diameter: $D_{col} = 3$ ft + 6 inch

The moment of inertia for an individual column with a circular cross section is calculated as follows. This calculation would be different for different column shapes.

$I_{col} = (\pi / 4) \times (D_{col} / 2)^4 = 7.37\ \text{ft}^4$

There are two columns per pier.

Number of columns: $n_{col} = 2$

The total stiffness for the pier can be found based on the number of columns per pier, moment of inertia of each column, length of the columns, and modulus of elasticity for the concrete, as follows.

$$K_{pier} = n_{col} \times \frac{3E_c\, I_{col}}{L_{col}^{\ 3}} = 457.4\ \frac{\text{kip}}{\text{ft}}$$

DESIGN LOAD DETERMINATION

There are three load components that need to be considered: (1) wind loads, (2) braking loads, and (3) thermal loads. Calculations are provided for the existing structure (prior to installation of the link slabs) and for the structure after link slab installation.

Wind Load on Structure Calculations (AASHTO LRFD BDS Article 3.8.1.2)

The design wind speed is found using AASHTO LRFD BDS Figure 3.8.1.1.2-1. Strength III wind speeds are shown in the figure. Design 3-second gust wind speeds for other load combinations are provided in Table 3.8.1.1.2-1. The design 3-second gust wind speed for this example from northcentral North Carolina are summarized in Figure 14.

$$\text{Design 3-second gust wind speed:}\ \ V = \begin{bmatrix} 115 \\ 80 \\ 70 \\ 86.25 \end{bmatrix} \text{mph} \quad \begin{matrix} \text{Strength III} \\ \text{Strength V} \\ \text{Service I} \\ \text{Service IV} \end{matrix}$$

Figure 14. Design 3-second gust wind speed.

The exposed area height for wind load calculations is found from the height of the superstructure and bridge railings.

Exposed area height: H_w = 8 ft + 7 inch

The approximate structure height is found based on AASHTO LRFD BDS Article 3.8.1.2.1, where “in no case shall the structure height, Z, used in calculating K_Z be taken as less than 33.0 ft." The minimum value is used in these calculations.

Approximate structure height (min 33 ft): Z = 33 ft

The pressure exposure and elevation coefficient is found using AASHTO LRFD BDS Eqn. 3.8.1.2.1-3, assuming wind exposure Category C for this example.

$$K_z = \frac{\left(2.5 \times \ln\left(\frac{Z}{0.0984}\right) + 7.35\right)^2}{478.4} = 1$$

The gust effect factor is found in AASHTO LRFD BDS Table 3.8.1.2.1-1.

Gust effect factor: $G = 1$

The drag coefficient is found in AASHTO LRFD BDS Table 3.8.1.2.1-2 for windward side of I-girder superstructure.

Drag coefficient: $C_D = 1.3$

The wind pressure is found using the above values in AASHTO LRFD BDS Eqn. (3.8.1.2.1-1).

Wind pressure: $P_z = 2.56 \times 10^{-6} \times V^2 \times K_z \times G \times C_d$

The wind pressures for each of the four load combinations considered are summarized in Figure 15.

$$P_z = \begin{bmatrix} 44.08 \\ 21.33 \\ 16.33 \\ 24.79 \end{bmatrix} \text{psf} \quad \begin{array}{c} \text{Strength III} \\ \text{Strength V} \\ \text{Service I} \\ \text{Service IV} \end{array}$$

Figure 15. Wind load values.

The wind load is distributed per pier in the structure before the link slab installation.

Area exposed to wind load, existing (per pier, prior to link slab installation):
$A_{WLe} = H_w \times Span = 738.2 \text{ ft}^2$

This area exposed to the wind load can be used to determine the longitudinal wind load (60-degree) for each of the piers prior to link slab installation, as shown in Figure 16.

$$WS_{60Le} = 0.380 A_{WLe} \times P_z = \begin{bmatrix} 12.36 \\ 5.98 \\ 4.58 \\ 6.95 \end{bmatrix} \text{kip} \quad \begin{array}{c} \text{Strength III} \\ \text{Strength V} \\ \text{Service I} \\ \text{Service IV} \end{array}$$

Figure 16. Longitudinal wind load, existing (per pier).

The wind load of the entire bridge length is distributed to the fixed piers after the installation of the link slabs.

Area exposed to wind load, proposed (per fixed pier, after link slab installation):
$A_{WLp} = 1 / n_{fix} \times H_w \times Length = 1476.3 \text{ ft}^2$

This area exposed to the wind load can be used to determine the longitudinal wind load (60-degree) for each of the fixed piers after link slab installation, as shown in Figure 17.

$$WS_{60Lp} = 0.380 A_{WLp} \times P_z = \begin{bmatrix} 24.73 \\ 11.97 \\ 9.16 \\ 13.91 \end{bmatrix} \text{kip} \quad \begin{matrix} \text{Strength III} \\ \text{Strength V} \\ \text{Service I} \\ \text{Service IV} \end{matrix}$$

Figure 17. Longitudinal wind load, proposed (per fixed pier).

The longitudinal wind load increased on the fixed piers after the installation of the link slabs.

Wind Load on Live Load Calculation (AASHTO LRFD BDS Article 3.8.1.3)

The installation of the link slabs will also increase the wind load on live load demand, found in AASHTO LRFD BDS Article 3.8.1.3. This load will be distributed to the substructure through the fixed piers after the installation of the link slabs. The loads before and after link slab installation are summarized below.

Existing wind load per pier: $WL_e = 0.038 \text{ klf} \times Span = 3.27 \text{ kip}$

Proposed wind load per fixed pier: $WL_p = (1 / n_{fix}) \times 0.038 \text{ klf} \times Length = 6.54 \text{ kip}$

Braking Load Calculation (AASHTO LRFD BDS Article 3.6.4)

The braking force specified in AASHTO LRFD BDS Article 3.6.4 is taken as the greater of:

- 25 percent of the axle weights of the design truck or tandem, or
- Five percent of the design truck plus lane load or five percent of the design tandem plus lane load.

The structure has one lane in each direction, so there is only one design truck headed in the same direction when calculating these loads.

The braking load per pier before link slab installation is shown below:

25 percent of axle weights: $BR_{e1} = 0.25 \times 72 \text{ kip} = 18 \text{ kip}$

5 percent of lane loading: $BR_{e2} = 0.05 \times (640 \text{ plf} \times Span + 72 \text{ kip}) = 6.35 \text{ kip}$

Controlling: BR_e = maximum of BR_{e1} and $BR_{e2} = BR_{e1} = 18 \text{ kip}$

The braking load per fixed pier after link slab installation is shown below.

25 percent of axle weights: $BR_{p1} = (1 / n_{fix}) \times 0.25 \times 72 \text{ kip} = 9 \text{ kip}$

5 percent of lane loading: $BR_{p2} = (1 / n_{fix}) \times 0.05 \times (640 \text{ plf} \times Length + 72 \text{ kip}) = 7.3 \text{ kip}$

Controlling: BR_p = maximum of BR_{p1} and $BR_{p2} = BR_{p1} = 9$ kip

The installation of the link slabs increases the demand from the lane load but allows for distribution of the braking force between the fixed piers. The combination of these factors led to a smaller braking force after the installation of the link slabs.

Thermal Load Calculation (AASHTO LRFD BDS Article 3.12.2)

There is no thermal load present prior to the installation of the link slabs. After the installation of the link slabs, the thermal expansion will lead to a load on the fixed piers. The loads can be found using AASHTO LRFD BDS Article 3.12.2.

The assumed temperature range for steel girder bridges with concrete decks was found from the contour maps in AASHTO LRFD BDS Figure 3.12.2.2-3 and Figure 3.12.2.2-4. AASHTO LRFD BDS Article C14.7.5.3.2 notes that the installation temperature of bearings is usually within 15 percent of the average of the maximum and minimum design temperatures and therefore, 65 percent of the thermal movement range can be used for design purposes. This 65 percent reduction is applied to the thermal range here.

Assumed thermal range: $\Delta T = 0.65 \times 110\ °\text{F} = 71.5\ °\text{F}$

The design thermal movement range can be found based on AASHTO LRFD BDS Eqn. 3.12.2.3-1. The assumed coefficient of thermal expansion for the normal weight concrete deck is used here ($\alpha = 6.0 \times 10^{-6}/°\text{F}$ from AASHTO LRFD BDS Article 5.4.2.2). The expansion length in this case is half the distance between the fixed piers, which is half of the span length for this example.

Thermal expansion between fixed piers: $\Delta_{TU} = 0.000006 \times (Span / 2) \times \Delta T = 0.22$ inch

The thermal load on the piers is the thermal expansion times the pier stiffness calculated earlier in this chapter.

Thermal load on piers: $TU = k_{pier} \times \Delta_{TU} = 8.44$ kip

FACTORED LONGITUDINAL LOAD

The two load combinations of interest for the longitudinal load are Strength III and Strength V.

- Strength III – Load combination relating to the bridge exposed to the design wind speed at the location of the bridge.
- Strength V – Load combination relating to normal vehicular use of the bridge with wind of 80 mph velocity

The longitudinal load used for design would be the maximum of these two load combinations.

The loads at Pier 2 prior to the installation of the link slabs are as follows:

Strength III: $H_{e,STRIII} = WS_{60Le} = 12.36$ kip

Strength V: $H_{e,STRV} = WS_{60Le} + 1.35BR_e + WL_e = 33.55$ kip

Controlling: H_e = maximum of $H_{e,STRIII}$ and $H_{e,STRV} = H_{e,STRV} = 33.55$ kip

The loads at Pier 2 after the installation of the link slabs are as follows. Per AASHTO LRFD BDS Article 3.4.1, for simplified analysis of concrete substructures in the strength limit state, $\gamma_{TU} = 0.5$ may be used when calculating force effects, taken in conjunction with the gross moment of inertia in the columns or piers.

Strength III: $H_{p,STRIII} = WS_{60Lp} + 0.5TU = 28.95$ kip

Strength V: $H_{p,STRV} = WS_{60Lp} + 1.35BR_p + WL_p + 0.5TU = 34.87$ kip

Controlling: H_p = maximum of $H_{p,STRIII}$ and $H_{p,STRV} = H_{p,STRV} = 34.87$ kip

The difference between the longitudinal force before and after the installation of the link slabs is calculated below.

$\Delta_{value} = H_p - H_e = 1.32$ kip

$\Delta_{percent} = (H_p - H_e) / H_e = 3.93$ percent

The longitudinal loads increased slightly by 3.93 percent. The increase is minor and is not likely to negatively impact the substructure. Per engineering judgment, a refined analysis does not need to be performed.

CHAPTER 5. BEARING DESIGN CALCULATIONS

INTRODUCTION

This chapter provides design calculations for laminated elastomeric bearings for the installation of UHPC link slabs in the four-span, steel simple-span composite structure. Elastomeric bearings are preferred for link slabs as they are able to better withstand repetitive horizontal movements from girder translation and rotation.

The elastomeric bearing is designed using Method A specified in AASHTO LRFD BDS Article 14.7.6. This bearing design will be used at all piers and at Abutment B. The thermal movement for design will be found based on the larger distance from the thermal center.

Pier 2 and Pier 3 are each assumed to have one fixed and one expansion bearing for this design. Thus, the "thermal center" is assumed to be at midspan between Pier 2 and Pier 3. The analysis of bearings in this chapter only considers bearings at the piers. This example will not consider changes needed for bearings and expansion joints at the two abutments.

DEMAND ON BEARING

The loads and deflections used in the design of the bearing were found through a structural analysis of the superstructure.

Bearing Loads

Loads represent the maximum from the four girder reactions at Pier 1 output from a 2D grillage model. The three load components needed for the bearing design are:

Component dead load: $DC = 123.91$ kip

Wearing surface dead load: $DW = 11.23$ kip

Live load: $LL = 132.67$ kip

Service I and Strength I load combinations will be used for the design of the bearings.

Service I load: $P_{SER\ I} = DC + DW + LL = 267.8$ kip

Strength I load: $P_{STR\ I} = 1.25DC + 1.50DW + 1.75LL = 403.9$ kip

Deflections

The midspan girder deflections due to camber, dead loads, and live loads were found from a 2D grillage model of the superstructure, as shown below.

Due to camber: $\delta_c = -1.625$ inch

Due to dead load: δ_{DL} = 2.075 inch

Due to wearing surface: δ_{DW} = 0.188 inch

Due to live load: δ_{LL} = 0.695 inch

Shear Deformations (Δ_s)

The shear deformations that will occur in the bearing are a combination of temperature deformations (Δ_t) and translation due to the end rotation of girder (Δ_b).

Total shear deformations: $\Delta_s = \Delta_t + \Delta_b$

The maximum temperature range and design temperature range are used to find temperature deflections. AASHTO LRFD BDS Article C14.7.5.3.2 notes that the installation temperature of bearings is usually within 15 percent of the average of the maximum and minimum design temperatures and therefore, 65 percent of the thermal movement range can be used for design purposes. This 65 percent reduction is applied to the design temperature range.

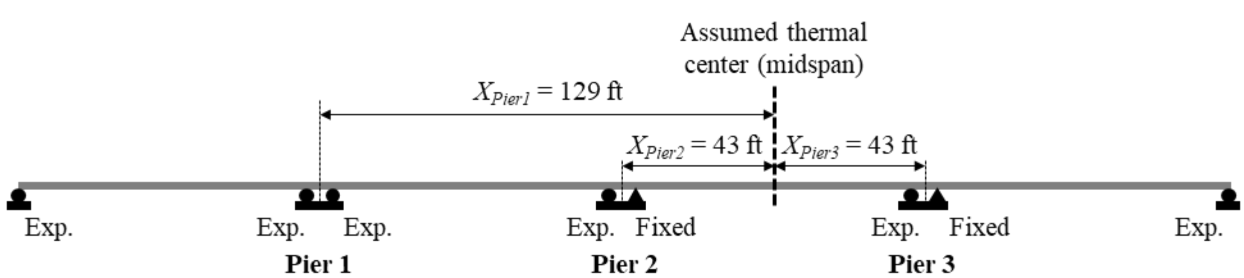
Maximum temperature range: ΔT_{max} = 110 °F

Design temperature range (65 percent): $T_d = 0.65\Delta T_{max}$ = 71.5 °F

The temperature deformation is found based on this design temperature range (T_d), coefficient of thermal expansion (α_s), and the distance from the thermal center to bearings (X). The temperature deformation is factored by 1.2 for the Service I load combination; the larger load factor is used for deformations.

Temperature deformation: $\Delta_t = 1.2\alpha_s \times T_d \times X$

The distance from the thermal center to bearings will depend on the pier being investigated. The distances for the three piers are highlighted in Figure 18.



Source: FHWA.

Figure 18. Illustration. Bearing fixity for bridge with thermal center and distance from thermal center to Pier 1.

As discussed in Chapter 1, the center of rotation is assumed to be located in the link slab. The end rotation will result in a translation in the roller or bearing, which can be found based on the end rotation and depth of the girder and deck.

Horizontal displacement at top of bearing: $\Delta_b = \theta \times h_r$

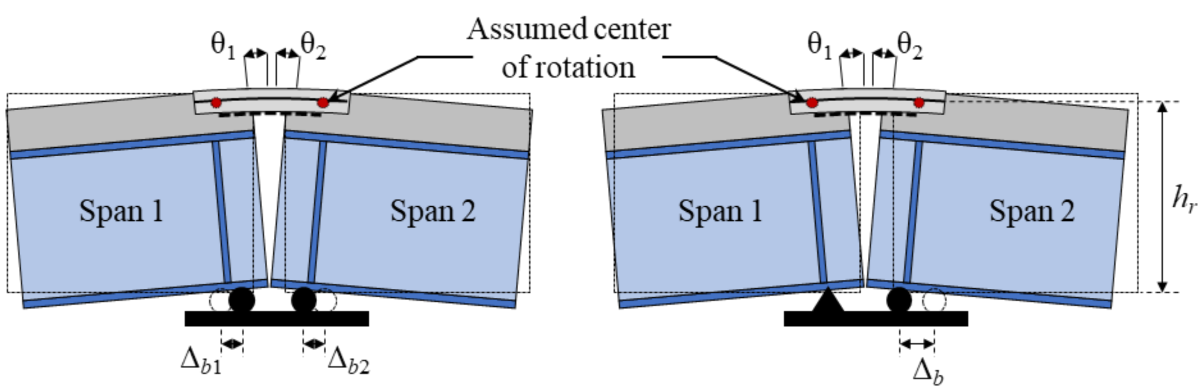
The girder end rotations were found in Chapter 2.

Design girder end rotation (Service I): θ_{SER} = 0.0027 rad

The girder height (h_g) is 49.375 inches, haunch thickness (h_h) is 3 inches, total deck thickness (h_d) is 8.5 inches, and link slab thickness (h) is 4 inches. These dimensions are used to find the distanced from the center of rotation to the top of the support.

Distance from center of rotation to top of support: $h_r = h_g + h_h + h_d - 0.5h$ = 58.875 inches

The rotation used to find the associated horizontal translation will depend on if the supports under the link slab are both expansion bearing or if one is fixed and the other expansion, as shown in Figure 19. If there is a fixed bearing present, all horizontal translation from the total girder rotation from both spans goes to the expansion bearing. If there are two expansion bearings, there is an assumed horizontal translation at each bearing proportional to the girder end rotation for each span.



Source: FHWA.

Figure 19. Illustration. Horizontal translation at bearing due to end rotation depending on support conditions.

There is often an expansion joint at the end of the span, so there are no horizontal translation deflections due to girder end rotation at the abutments. The total shear deformations will depend on the support location, as summarized in Table 8. Recognize that whether thermal is expansion or contraction, on one side of the pier the thermal translation and translation from girder rotation will be additive.

Table 8. Components of shear deformations at different supports (Service I).

Support	X (ft)	Δ_t (in)	θ (rad)	Δ_b (in)	Δ_s (in)
Left Abutment[a]	215	1.44	--	--	1.44
Pier 1	129	0.86	0.0027	0.16	1.02
Pier 2	43	0.29	0.0054[b]	0.32	0.61
Pier 3	43	0.29	0.0054[b]	0.32	0.61
Right Abutment[a]	129	0.86	--	--	0.86

[a]provided for information only. Design example only focusing on bearings at pier locations.
[b]Piers 2 and 3 have fixed and expansion bearings, therefore twice the Service I rotation is used.

BEARING DESIGN CALCULATIONS

All pier bearings have the same loads and deflections. Pier 1 has the largest shear deformation (of the piers) and will control the design. The same bearing size will be used for all the piers to make fabrication easier. The bearing design calculations are only provided for Pier 1, but the same bearing can also be used for the expansion bearings in Pier 2 and Pier 3.

Material Properties

The material properties for the elastomeric bearing are shown below.

Elastomer hardness: 60 Durometer

Minimum shear modulus: G_{min} = 0.13 ksi

Maximum shear modulus: G_{max} = 0.20 ksi

Creep / initial deflection ratio: a = 0.35

Steel reinforcement yield strength: f_y = 36 ksi

Constant-amplitude fatigue threshold: ΔF_{TH} = 24 ksi

Steel thermal expansion coefficient: $\alpha_s = 6.50 \times 10^{-6}$ (1 ⁄ °F)

Preliminary Sizing

The preliminary sizing can be found based on the maximum service bearing stress and the applied service load on the bearing. The maximum service bearing stress is found using AASHTO LRFD BDS Article 14.7.6.3.2-8 for steel-reinforced elastomeric bearings.

Maximum service bearing stress: $\sigma_{s,max}$ = 1.25 ksi

The minimum bearing area is simply the total service load applied on the bearing ($P_{SER\ I}$) divided by the maximum allowable bearing stress.

Minimum bearing area: $A_{min} = P_{SER\ I} / \sigma_{s,max} = 214.2 \text{ in}^2$

The bearing dimensions can be selected based on this minimum bearing area.

Bearing pad width: $W = 21$ inch

Bearing pad length: $L = 11$ inch

Bearing pad area: $A = L \times W = 231 \text{ inch}^2$

The interior layer thickness and steel layer thickness initially selected for this design are shown below.

Interior layer thickness: $h_{ri} = 0.5$ inch

Steel layer thickness: $h_s = 0.1192$ inch

Bearing Height

The minimum elastomer thickness is found using AASHTO LRFD BDS Eqn. 14.7.6.3.4-1 based on the maximum total shear deformation of the bearing (Δ_s).

Minimum elastomer thickness: $h_{rt,min} = 2\Delta_s = 2.04$ inch

The minimum number of elastomer layers can be found by taking the minimum thickness divided by the interior layer thickness defined above.

Minimum layers, based on deflection: $n_\delta = h_{rt,min} / h_{ri} = 4.09$

The actual number of layers should be this minimum layer number rounded up to the nearest whole number. The number of steel layers is the number of elastomer layers minus one.

Number of elastomer layers: $n_{ri} = 5$

Number of steel layers: $n_s = n_{ri} - 1 = 4$

The total elastomer height and total bearing height can be found based on the number of layers and layer thicknesses.

Total elastomer height: $h_{rt} = n_{ri} \times h_{ri} = 2.5$ inch

Total bearing height: $h_b = h_{rt} + n_s \times h_s = 2.98$ inch

Shape Factor

The shape factor for the bearing is specified in AASHTO LRFD BDS Eqn. 14.7.5.1-1and calculated below. This shape factor is for the expansion bearing without a hole.

$$S_i = \frac{L \times W}{2h_{ri} \times (L + W)} = 7.22$$

A 1.5-inch anchor pin is provided in the elastomeric bearing used for the fixed bearing with a hole diameter of 1.625 inches.

Hole diameter: d_h = 1.625 inch

Shape factors for rectangular bearings with holes are found using AASHTO LRFD BDS Eqn. C14.7.5.1-1, as calculated here

$$S_{i,hole} = \frac{L \times W - \frac{\pi}{4} d_h^{\,2}}{h_{ri} \times (2L + 2W + \pi d_h)} = 6.63$$

Method A may be used for steel-reinforced elastomeric bearings in with $S_i^2/n_{ri} < 22$, as specified in AASHTO LRFD BDS Article 14.7.6.1.

Limit for Method A: $S_i^2/n_{ri} = 10.4 < 22$

This means that Method A may be used in this example.

Bearing Stability (AASHTO LRFD BDS Article 14.7.6.3.6)

The stability of the bearing is ensured by dimensional limits specified in AASHTO LRFD BDS Article 14.7.6.3.6. The total thickness of the bearing pad shall not exceed the least of $L/3$ or $W/3$ (for rectangular bearings).

Total bearing height: h_b = 2.98 inch

Max bearing height, based on length: $L/3$ = 3.67 inch

Max bearing height, based on width: $W/3$ = 7 inch

The total height is less than the maximum bearing height limits, so the bearing stability checks are satisfied.

Bearing Stresses

The bearing stresses are found by taking the total service load divided by the area. The stresses due to dead load, live load, and Service I loading are provided below.

Due to dead load: $\sigma_{DL} = (DC + DW)/A$ = 0.59 ksi

Due to live load: $\sigma_{LL} = LL/A$ = 0.57 ksi

Due to Service I load: $\sigma_{SER\,I} = P_{SER\,I}/A$ = 1.16 ksi

The stress due to the Service I load is compared against the allowable service stress limit provided in AASHTO LRFD BDS Eqn. 14.7.6.3.2-7 and Eqn. 14.7.6.3.2-8.

Allowable service stress: $\sigma_{allow} = \min(1.25,\ 1.25G_{min} \times S_i) = 1.17$ ksi

The stress due to Service I load (1.16 ksi) is less than the allowable service stress limit (1.17 ksi), so this design check is satisfied.

Reinforcement (AASHTO LRFD BDS Article 14.7.6.3.7, which refers to Article 14.7.5.3.5)

The design requirements for the steel reinforcement layers are specified in AASHTO LRFD BDS Article 14.7.5.3.5. The minimum thickness of steel reinforcement layer (h_s) is 0.0625 inch The steel layer thickness must also satisfy service and fatigue limit state requirements provided in AASHTO LRFD BDS Eqn. 14.7.5.3.5-1 and Eqn. 14.7.5.3.5-2, respectively. The thickness provided and the minimum steel thicknesses are shown below.

Steel layer thickness: $h_s = 0.1192$ inch

Minimum steel thickness, service: $h_{min,SER} = (3h_{ri} \times \sigma_{SER\ I}) / f_y = 0.05$ inch

Minimum steel thickness, fatigue: $h_{min,FAT} = (2h_{ri} \times \sigma_{LL}) / \Delta F_{TH} = 0.02$ inch

The provided thickness is greater than the thickness limits, so this design check is satisfied.

Compressive Deflection (AASHTO LRFD BDS Article 14.7.6.3.3, which refers to Article 14.7.5.3.6)

Compression deflections from dead loads and live loads are limited by provisions in AASHTO LRFD BDS Articles 14.7.6.3.3 and 14.7.5.3.6. The shape factor and compressive stresses were found in previous sections.

Shape factor, interior layer: $S_i = 7.22$

Compressive stress, dead load: $\sigma_{DL} = 0.59$ ksi

Compressive stress, live load: $\sigma_{LL} = 0.57$ ksi

Compressive stress, service: $\sigma_{SER\ I} = 1.16$ ksi

Stress in the bearing can be related to strain in the bearing using the calculated shape factor and AASHTO LRFD BDS Figure C14.7.6.3.3-1 for 60 durometer reinforced bearings. The compressive strain due to dead load and live load found from this figure are shown below.

Compressive strain, dead load: $\varepsilon_{DL} = 2.6$ percent

Compressive strain, live load: $\varepsilon_{LL} = 2.7$ percent

These strains can be used with the equations provided in AASHTO LRFD BDS Article 14.7.5.3.6 to find the dead load and live load deflections.

The initial dead load deflection of the elastomeric bearing is specified in AASHTO LRFD BDS Eqn. 14.7.5.3.6-2.

Initial dead load deflection: $\delta_d = \varepsilon_{DL} \times h_{rt} = 0.065$ inch

The long-term dead load deflection of the elastomeric bearing is specified in AASHTO LRFD BDS Eqn. 14.7.5.3.6-3.

Long-term dead load deflection: $\delta_{lt} = \delta_d \times (1 + a) = 0.088$ inch

This deflection is compared against the allowable long-term dead load deflection specified in AASHTO LRFD BDS Article 14.7.6.3.3.

Allowable dead load deflection: $\delta_{da} = 0.09h_{rt} = 0.23$ inch

The instantaneous live load deflection of the elastomeric bearing is specified in AASHTO LRFD BDS Eqn. 14.7.5.3.6-1.

Instantaneous live load deflection: $\delta_L = \varepsilon_{LL} \times h_{rt} = 0.068$ inch

This deflection is compared against the allowable live load deflection specified in AASHTO LRFD BDS Article 14.7.6.3.3.

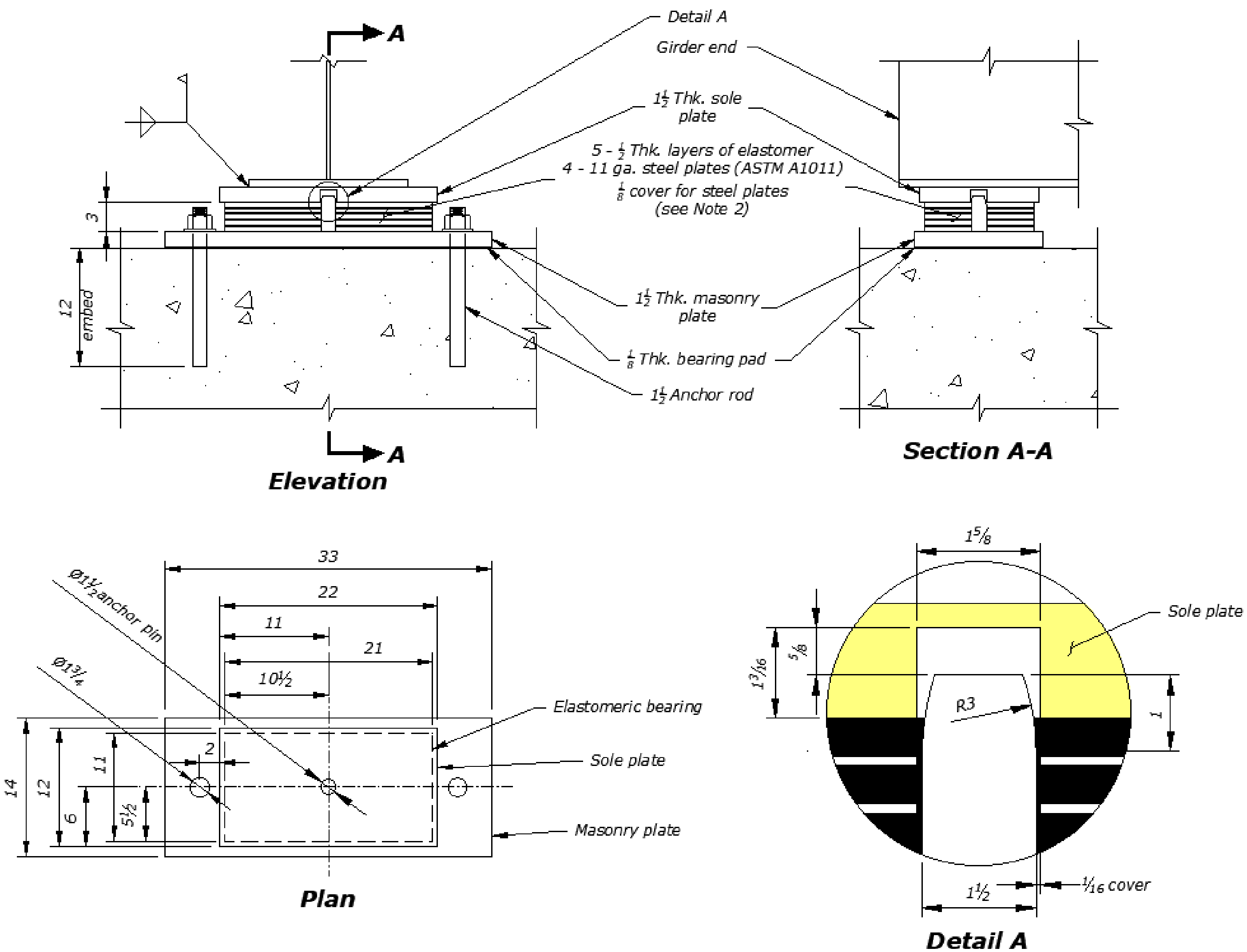
Allowable live load deflection: $\delta_{La} = \min(0.125 \text{ inch}, 0.09h_{rt}) = 0.13$ inch

The calculated deflections are less than the allowable deflections, so this design check is satisfied.

FINAL DETAILS FOR BEARING DESIGN

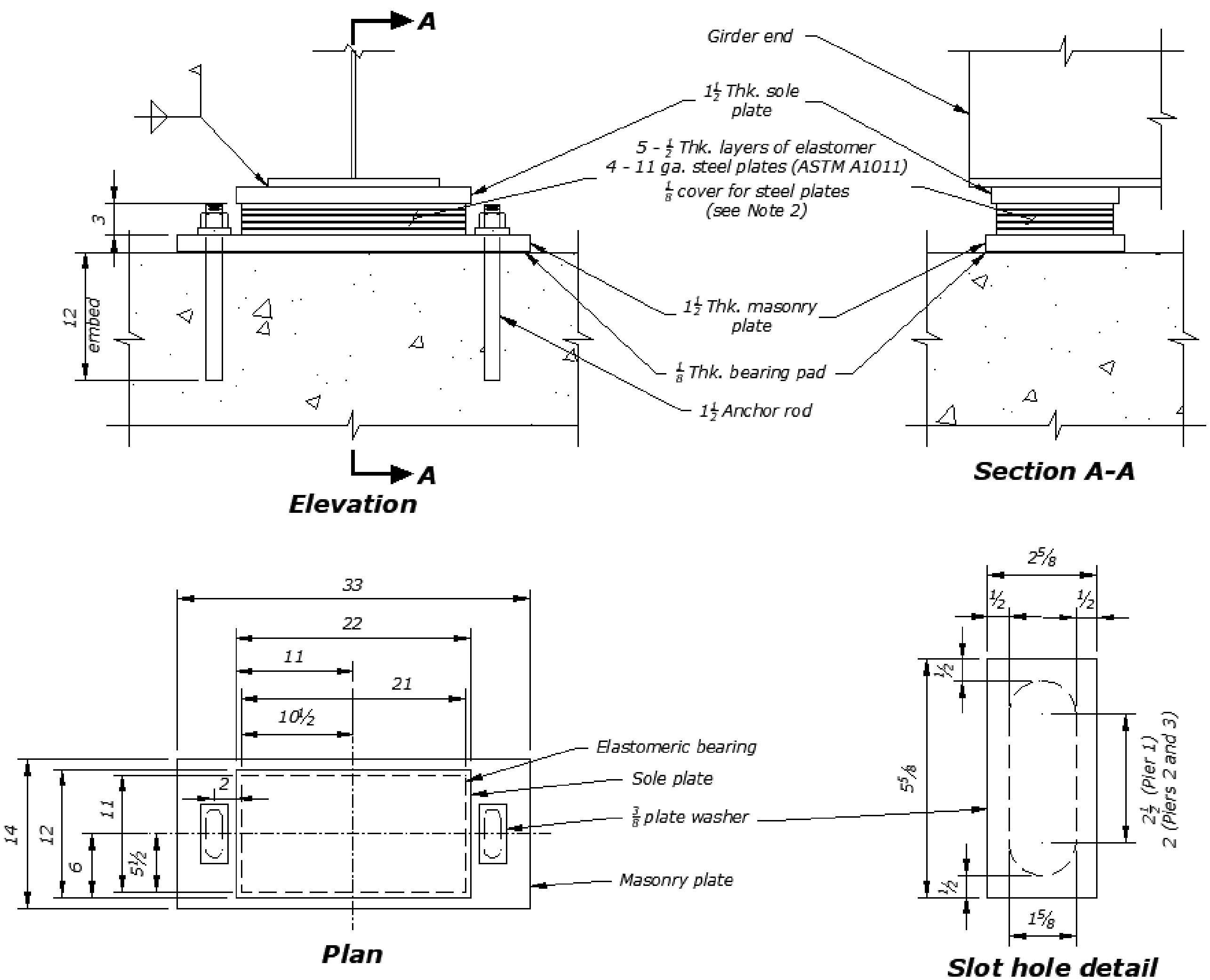
The final elastomeric bearing details for the fixed bearing and expansion bearing are provided in this section.

The elevation details for the fixed and expansion bearing assemblies are shown in Figure 20 and Figure 21, respectively. The bottom flange of the existing steel girder is welded to a 1.5-inch-thick sole plate, which sits on a reinforced elastomeric bearing pad on a 1.5-inch-thick masonry plate. The masonry plate is attached to the pier with 1.5-inch diameter anchor bolts. A 1.5-inch diameter anchor pin attached to the masonry plate extends through the elastomeric bearing pad into a hole in the bottom of the sole plate for the fixed bearing assembly. The 1.5-inch diameter anchor pin is based on the minimum pin diameter allowed by New York State DOT.



Source: FHWA.
Note 1: Units are in inches.
Note 2: Factory vulcanize elastomeric bearing to masonry and sole plates.

Figure 20. Illustration. Fixed bearing assembly details.



Source: FHWA.
Note 1: Units are in inches.
Note 2: Factory vulcanize elastomeric bearing to masonry and sole plates.

Figure 21. Illustration. Expansion bearing assembly details.

CHAPTER 6. REFERENCES

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APPENDIX A: ADDITIONAL DETAILS ON UHPC LINK SLAB DESIGN

Details and sample calculations are provided in this appendix for three additional factors that could be included in the design of a UHPC link slab:

- Checking allowable tensile stress in UHPC for components subjected to cyclic tensile stresses (from UHPC Guide Article 1.5.2.3) – current recommendation is to ignore this check for UHPC link slab design.
- Including the effect of the shrinkage strain in the UHPC link slab in the design.
- Including the effect of the strains due to thermal movement in the UHPC link slab design.

The inclusion of the cyclic stress check or shrinkage strain in the UHPC link slab design will result in debond lengths that are much longer than standard practice.

Allowable UHPC Tensile Stress for Components Subjected to Cyclic Stresses (UHPC Guide 1.5.2.3):

The UHPC Guide requires that the tensile stress in the UHPC due to the Service I load combinations does not exceed $0.95\gamma_u f_{t,cr}$ for components subjected to cyclic stresses (UHPC Guide 1.5.2.3). However, with the low thickness and short span of link slabs, this is typically not achievable without excessively long debonded lengths for the link slabs. The calculations provided in this section show the debond length required for the UHPC link slab to meet this design check.

The allowable tensile stress in the UHPC for components subjected to cyclic stresses (UHPC Guide 1.5.2.3) for this example is the following:

Allowable tensile stress in UHPC for components subjected to cyclic stresses: $f_{ta,SER} = 0.95\gamma_u f_{t,cr} = 0.71$ ksi

This tensile stress limit is associated with a tensile strain of $0.95\varepsilon_{t,cr}$, which is the following for this example.

Strain associated with allowable tensile stress limit: $\varepsilon_{ta,SER} = 0.95\varepsilon_{t,cr} = 0.00011$

The tensile strain and stress at the extreme tension fiber for the UHPC link slab design investigated in Chapter 3 (h = 4 in, L_{DB} = 24 in, #5 at 12 in at mid-height of link slab) were calculated as follows.

Strain at extreme tension fiber (Service I, from Chapter 3): $\varepsilon_{t,SER} = 0.000563$

Stress at extreme tension fiber (Service I, from Chapter 3): $f_{t,max,SER} = 0.75$ ksi

The strain and stress both exceed the allowable strain and stress limits. Given that the UHPC is assumed to exhibit elastic-plastic behavior in tension, the calculated strain relative to the allowable strain limit is the more important comparison. The strain in the extreme tensile fiber is 633 percent larger than the allowable tensile strain.

To meet the allowable tensile stress in UHPC for components subjected to cyclic stresses limit, the UHPC link slab would need either to be thicker or to have a longer debonded length. A thicker UHPC link slab would be less flexible, and therefore not the preferred adjustment to make.

The following calculations are to determine the required unbonded length of the link slab to meet the tensile stress requirement for components subjected to cyclic stresses. The debond length error and tolerance are set like before.

Initialize debond length error: err_L = 1 inch

Initialize debond length tolerance: $tol_L = tol$ = 0.0001 inch

The initially assumed debond length to start the iterative process in this example is selected based on the debond length and service tensile stress found in Chapter 3 and the tensile strain limit for components subjected to cyclic stresses.

$L_{it} = L_{DB} \times (\varepsilon_{t,SER} / (0.95\varepsilon_{t,SER}))$ = 123.0 inches

An iterative process like that discussed in Chapter 3 can be used to calculate the minimum debond length. A neutral axis depth is calculated for each debond length. This neutral axis depth is used to calculate a new debond length, which is then compared to the assumed value at the start of the iteration.

$\varepsilon_t = [2(h - c_{iti})\theta_{SER}] / L_{it}$

$L_{iti} = L_{it} \times (\varepsilon_t / (0.95\varepsilon_{t,SER}))$

err_L = absolute value of $(L_{it} - L_{iti})$

The iterative process is complete when $err_L \leq tol_L$. Through this iterative process, the neutral axis (c_{cyc}), strain at the extreme tension fiber ($\varepsilon_{t,cyc}$), and required debond length (L_{cyc}) were determined to be the following values.

Required debond length to meet cyclic requirements (UHPC Guide 1.5.3): L_{cyc} = 99.81 inches

Associated neutral axis depth: c_{cyc} = 2.0 inches

Associated extreme tension fiber strain: $\varepsilon_{t,cyc} = [2(h - c_{cyc})\theta_{SER}] / L_{cyc}$ = 0.000110

This debonded length is about four times as the debonding length found in Chapter 3.

Debond length ratio: $ratio_L = L_{cyc} / L_{DB}$ = 4.16

This link slab length is significantly larger than the current state of practice for state DOT's currently designing and constructing UHPC link slabs.

Strain Due to Shrinkage (UHPC Guide Article 1.4.2.8.3):

As previously mentioned, the shrinkage strain in the UHPC may affect the behavior of UHPC link slabs. UHPC link slabs are restrained on each end by the existing concrete deck, which will restrain any shrinkage that occurs in the UHPC.

The shrinkage strain in the UHPC can be predicted using UHPC Guide Equation 1.4.2.8.3-1.

Strain due to shrinkage for UHPC: $\varepsilon_{SH} = k_s \times k_{hs} \times k_f \times k_{td} \times K_4 \times 0.6 \times 10^{-3}$

This shrinkage strain can be included in the design calculations by subtracting ε_{SH} from the allowable tensile strain in the UHPC.

The factors required to calculate the shrinkage strain are shown below.

Relative humidity (AASHTO LRFD BDS Figure 5.4.2.3.3-1): $H = 70$

Volume-to-surface ratio factor: $k_s = 1.0$

Humidity factor: $k_{hs} = 1.5 - 0.01H = 0.8$

UHPC strength factor, calculated as follows:

$$k_f = \frac{18}{1.5f'_c - 3} = 0.75$$

Time development factor (approaches 1.0 as time approaches infinity): $k_{td} = 1.0$

Correction factor: $K_4 = 1.0$

The shrinkage strain in the UHPC is thus calculated as follows.

Strain due to shrinkage: $\varepsilon_{SH} = 0.6 \times 10^{-3} \times k_s \times k_{hs} \times k_f \times k_{td} \times K_4 = 0.000360$

The allowable tensile strain from UHPC Guide Article 1.5.2 is shown below (not including the shrinkage strain).

Allowable UHPC tension strain (not including shrinkage strains): $\varepsilon_{ta,SER}$
$= min\ (0.25\varepsilon_{t,loc}, 0.001) = 0.000625$

The allowable tensile strain including the effect of the restrained shrinkage in the UHPC is then calculated.

Allowable UHPC tension strain (including shrinkage strains): $\varepsilon_{ta,SER} - \varepsilon_{SH} = 0.000265$

The strain in the extreme tension fiber due to Service I loading was calculated in Chapter 3.

Strain at extreme tension fiber (Service I): $\varepsilon_{t,SER} = 0.000563$

This strain exceeds the allowable tension strain including the effects of the restrained shrinkage, so the debond length of the link slab would need to be increased.

The debond length to satisfy the allowable tension strain including the effects of the restrained shrinkage (L_{shr}) is calculated using the iterative procedure like that used to determine the required debond length to satisfy the cyclic load requirements (L_{cyc}). The initial debond length error and tolerance to stop iterations are the following values.

Initialize debond length error: err_L = 1 inch

Initialize debond length tolerance: $tol_L = tol$ = 0.0001 inch

The initially assumed debond length for the first iteration is determined using the following equation.

$$L_{it} = L \times [\varepsilon_{t,SER} / (0.95\varepsilon_{t,cr})] = 123.0 \text{ inches}$$

An iteritive process like that discussed in the previous section can be used to calculate the minimum debond length when shrinkage strains are included. The process is the same except that the shrinkage strain is substracted from the allowable tensile strain when finding the debond length in each iteration.

$$L_{iti} = L_{it} \times [\varepsilon_t / (\varepsilon_{ta,SER} - \varepsilon_{SH})]$$

The debond length was calculated through this iterative process and was determined to be the following value.

Debond length to account for UHPC shrinkage: L_{shr} = 45.14 inch

This length is about twice the debond length required when the restrained shrinkage was not considered. The state of practice for State DOT's currently designing and constructing UHPC link slabs does not include the shrinkage strain effect when determining the debond length. There have not been any reported issues with shrinkage cracking to date, but this topic should continue to be monitored. Additionally, owners may consider specifying lower-shrinkage UHPC mixtures for use UHPC link slabs as these types of mixtures are anticipated to exhibit less cracking.

Strain Due to Thermal Movement:

As discussed in Chapter 4, the installation of UHPC link slabs will create connectivity along the length of the superstructure between expansion joints. This will lead to a thermal load in piers that will also be experienced by the superstructure. The thermal load was calculated in Chapter 4.

Thermal load from pier: (from simplified analysis) F_{TU} = 8.44 kip

This thermal load can be assumed to be applied to the entire superstructure. The steel girders can be transformed into an equivalent concrete area and added to the area of the concrete deck to find the total transformed area for the superstructure. The concrete deck area is found by taking the total bridge width (B) times the deck thickness (t_d).

Concrete deck area: $A_c = t_d \times B = 3{,}468 \text{ in}^2$

The modular ratio is the ratio of the modulus of the steel (E_s) to the modulus of the deck concrete (E_c).

Modular ratio: $n = E_s / E_c = 4.47$

The total steel area for the steel girders is calculated by taking the steel area for one girder (65 in^2) times the number of girders (4).

Total steel area: $A_{sb} = 4 \times 65 \text{ in}^2 = 260 \text{ in}^2$

The transformed area is found by taking the modular ratio times the total steel area.

Steel beam area, transformed: $A_{st} = n \times A_{sb} = 1{,}161.96 \text{ in}^2$

The strain due to the thermal load is calculated as follows.

$\varepsilon_{TU} = F_{TU} / [E_c \times (A_c + A_{st})] = 2.81 \times 10^{-7}$

The strain due to thermal movement can be included in the same manner as shrinkage strain, by subtracting ε_{TU} from the allowable tensile strain in the UHPC. For this example, this strain is small and can be neglected.