LRFD Design Example #2:

Cast-in-Place Flat Slab Bridge Design

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About this Design Example

Description

This document provides guidance for the design of a cast-in-place flat slab bridge.

The example includes the following component designs:

- Solid c.i.p. slab design
- Edge Beam design
- Expansion Joint design
- Intermediate bent cap design

The following assumptions have been incorporated in the example:

- Three span continuous @ 35'-0" each for a total of 105'-0" bridge length
- 30 degree skew
- No phased construction.
- Two traffic railing barriers and one median barrier.
- No sidewalks.
- Permit vehicles are not considered.
- Load rating is not addressed.

Since this example is presented in a **Mathcad** document, a user can alter assumptions, constants, or equations to create a customized application.

Standards

The example utilizes the following design standards:

- Florida Department of Transportation Standard Specifications for Road and Bridge Construction (2000 edition) and applicable modifications.
- AASHTO LRFD Bridge Design Specifications, 2nd Edition, 2002 Interims.
- Florida Department of Transportation Structures LRFD Design Guidelines, January 2003 Edition.
- Florida Department of Transportation Structures Detailing Manual for LRFD, 1999 Edition.

Defined Units

All calculations in this electronic book use U.S. customary units. The user can take advantage of Mathcad's unit conversion capabilities to solve problems in MKS or CGS units. Although Mathcad has several built-in units, some common structural engineering units must be defined. For example, a lbf is a built-in Mathcad unit, but a kip or ton is not. Therefore, a kip and ton are globally defined as:

$kip \equiv 1000 \cdot lbf$ $\quad$ $ton \equiv 2000 \cdot lbf$

Definitions for some common structural engineering units:

$N \equiv newton$ $\quad$ $kN \equiv 1000 \cdot newton$

$plf \equiv \frac{lbf}{ft}$ $\quad$ $psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$ $\quad$ $psi \equiv \frac{lbf}{in^2}$

$klf \equiv \frac{kip}{ft}$ $\quad$ $ksf \equiv \frac{kip}{ft^2}$

$ksi \equiv \frac{kip}{in^2}$ $\quad$ $°F \equiv 1deg$

$MPa \equiv 1 \cdot 10^6 \cdot Pa$ $\quad$ $GPa \equiv 1 \cdot 10^9 \cdot Pa$

Acknowlegements

The Tampa office of HDR Engineering, Inc. prepared this document for the Florida Department of Transportation.

Notice

General Notes

Design Method................ Load and Resistance Factor Design (LRFD) except that Prestressed Piles have been designed for Service Load.

Design Loading................. HL-93 Truck

Future Wearing Surface... Design provides allowance for 15 psf

Earthquake........................ Seismic provisions for minimum bridge support length only **[SDG 2.3.1]**.

Concrete............................

Class	Minimum 28-day Compressive Strength (psi)	Location
II	f`c = 3400	Traffic Barriers
II (Bridge Deck)	f`c = 4500	CIP Flat Slab
IV	f`c = 5500	CIP Substructure
V (Special)	f`c = 6000	Concrete Piling

Environment...................... The superstructure is classified as slightly aggressive.
The substructure is classfied as moderately aggressive.

Reinforcing Steel............. ASTM A615, Grade 60

Concrete Cover................

Superstructure	
Top deck surfaces	2" (Short bridge)
All other surfaces	2"
Substructure	
External surfaces exposed	3"
External surfaces cast against earth	4"
Prestressed Piling	3"

Concrete cover does not include reinforcement placement or fabrication tolerances, unless shown as "minimum cover". See FDOT Standard Specifications for allowable reinforcement placement tolerances.

Dimensions....................... All dimensions are in feet or inches, except as noted.

Design Parameters

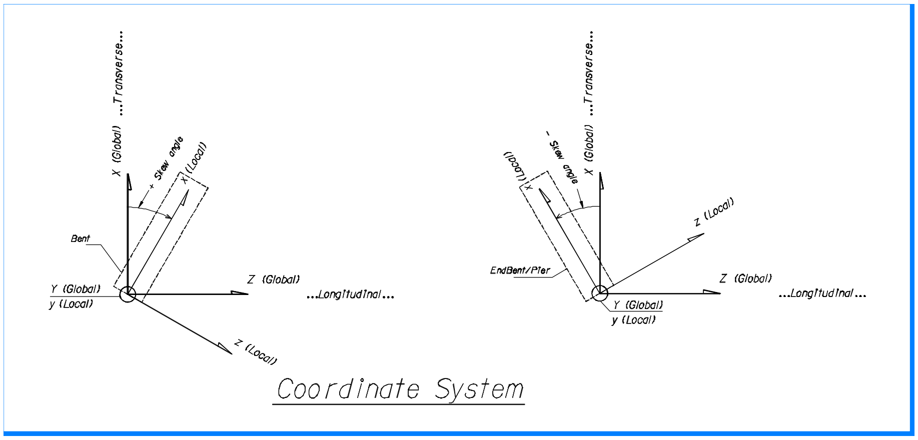
Description

This section provides the design input parameters necessary for the superstructure and substructure design.

Page	Contents

A. General Criteria

This section provides the general layout and input parameters for the bridge example.



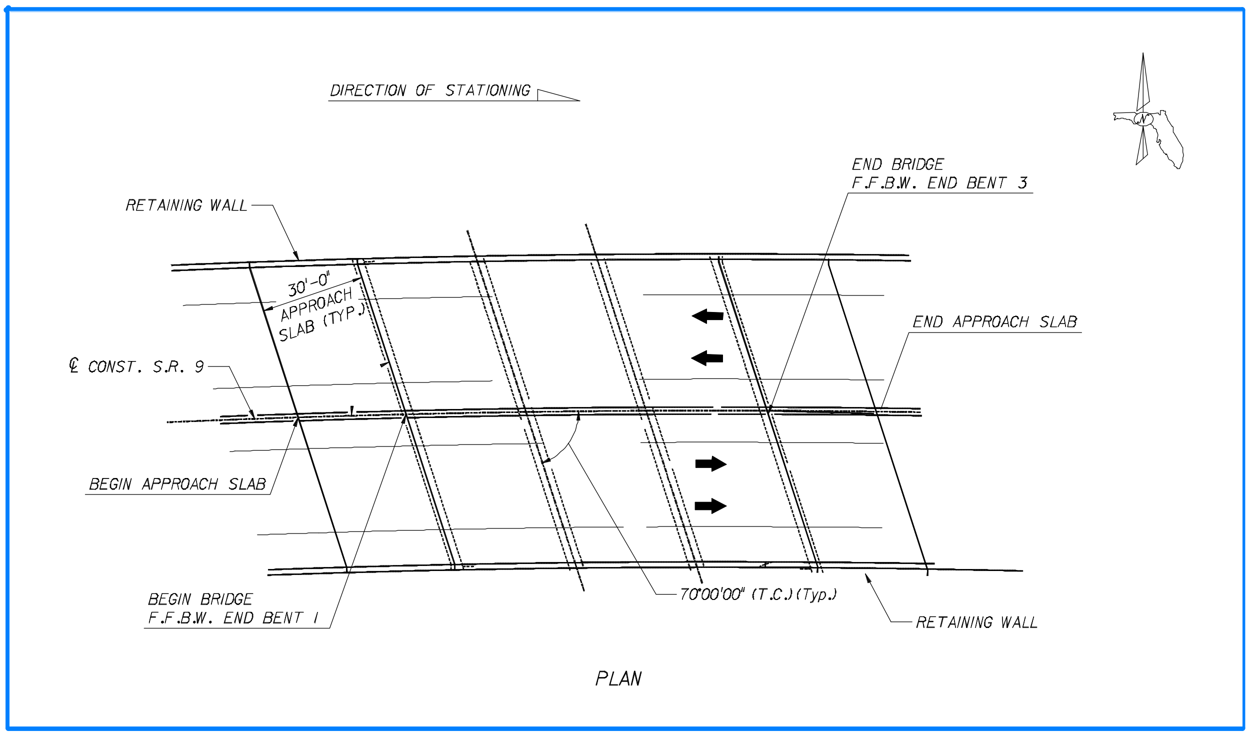
In addition, the bridge is also on a skew which is defined as:

Skew Angle......................... $Skew := -30deg$

A1. Bridge Geometry

Horizontal Profile

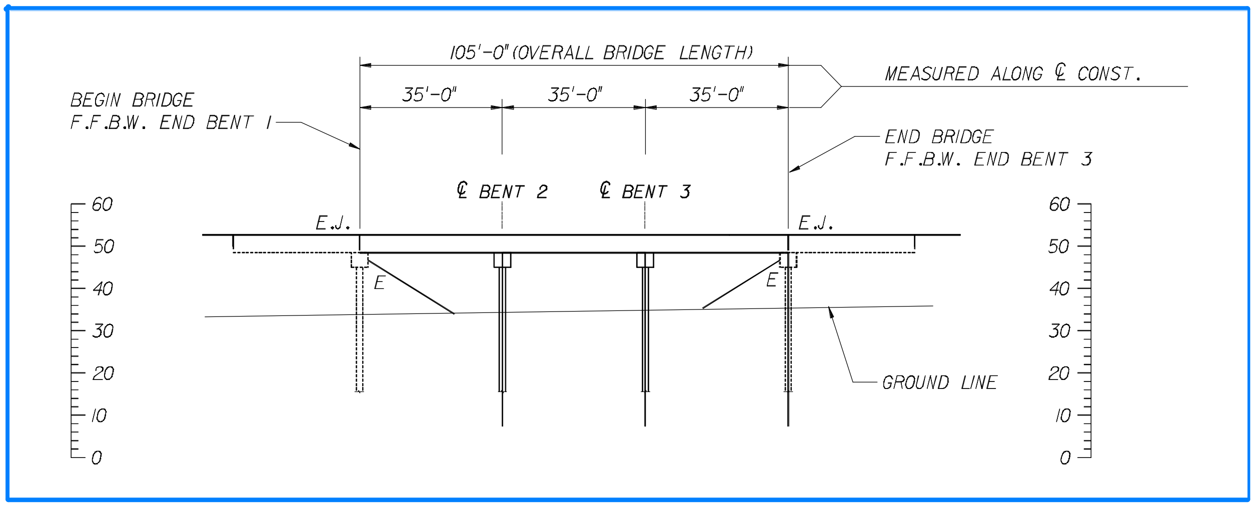
A slight horizontal curvature is shown in the plan view. For all component designs, the horizontal curvature will be taken as zero.



HORIZONTAL CURVE DATA

R = 3,800'

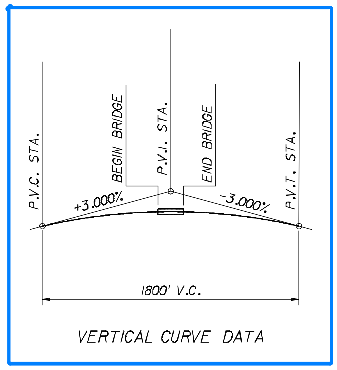
Vertical Profile



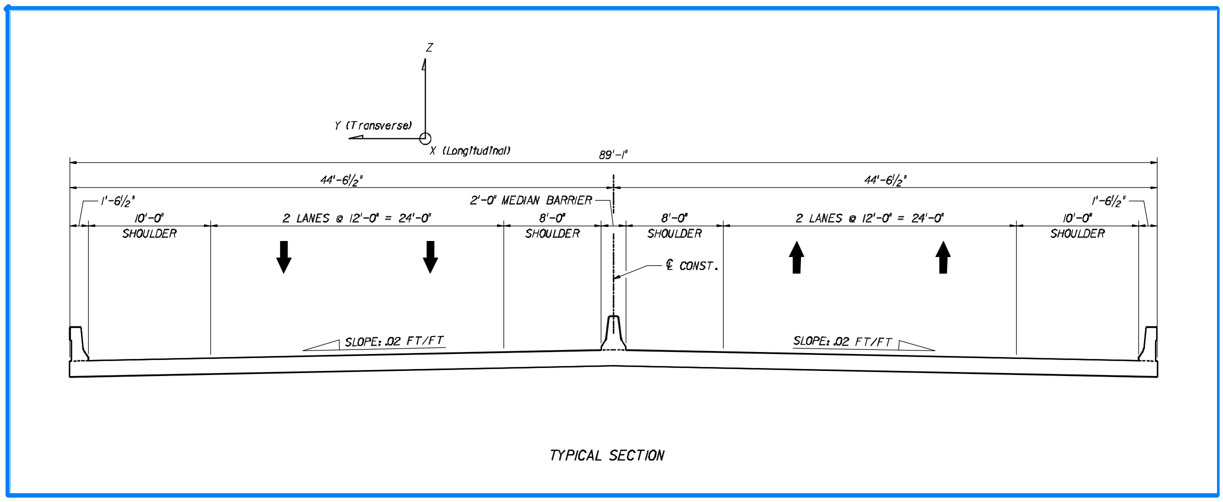
Overall bridge length............. $L_{bridge} \equiv 105 \cdot ft$

Bridge design span length...... $L_{span} := 35 \cdot ft$

*(**Note**: For unsymmetric spans, use average span length)*



Typical Cross-secton



Overall bridge width............ $W_{bridge} := 89.0833 \cdot ft$

A2. Number of Lanes

Design Lanes

Current lane configurations show two striped lanes per roadway with a traffic median barrier separating the roadways. Using the roadway clear width between barriers, $Rdwy_{width}$, the number of design traffic lanes per roadway, N_{lanes}, can be calculated as:

Roadway clear width........... $Rdwy_{width} := 42 \cdot ft$

Number of design traffic lanes per roadway......................... $N_{lanes} := \mathrm{floor}\left(\frac{Rdwy_{width}}{12 \cdot ft}\right)$

$N_{lanes} = 3$

A3. Concrete, Reinforcing and Prestressing Steel Properties

Unit weight of concrete......... $\gamma_{conc} := 150 \cdot pcf$

Modulus of elasticity for reinforcing steel.................. $E_s := 29000 \cdot ksi$

B. LRFD Criteria

The bridge components are designed in accordance with the following LRFD design criteria:

B1. Dynamic Load Allowance [LRFD 3.6.2]

An impact factor will be applied to the static load of the design truck or tandem, except for centrifugal and braking forces.

Impact factor for fatigue and fracture limit states............... $IM_{fatigue} := 1 + \frac{15}{100}$

Impact factor for all other limit states.................................. $IM := 1 + \frac{33}{100}$

B2. Resistance Factors [LRFD 5.5.4.2]

Flexure and tension of reinforced concrete............. $\phi := 0.9$

Shear and torsion of normal weight concrete................... $\phi_v := 0.90$

B3. Limit States [LRFD 1.3.2]

The LRFD defines a limit state as a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed. There are four limit states prescribed by LRFD. These are as follows:

STRENGTH LIMIT STATE

Load combinations which ensures that strength and stability, both local and global, are provided to resist the specified load combinations that a bridge is expected to experience in its design life. Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained.

EXTREME EVENT LIMIT STATES

Load combinations which ensure the structural survival of a bridge during a major earthquake or flood, or when collided by a vessel, vehicle, or ice flow, possibly under scoured conditions. Extreme event limit states are considered to be unique occurrences whose return period may be significantly greater than the design life of the bridge.

SERVICE LIMIT STATE

Load combinations which place restrictions on stress, deformation, and crack width under regular service conditions.

FATIGUE LIMIT STATE

Load combinations which place restrictions on stress range as a result of a single design truck. It is intended to limit crack growth under repetitive loads during the design life of the bridge.

Table 3.4.1-1 - Load Combinations and Load Factors

Load Combination	DC DD DW	LL IM CE	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
Limit State	EH EV ES	BR PL LS								EQ	IC	CT	CV
Strength I	y_p	1.75	1.00	-	-	1.00	0.50/1.20	y_{TG}	y_{SE}	-	-	-	-
Strength II	y_p	1.35	1.00	-	-	1.00	0.50/1.20	y_{TG}	y_{SE}	-	-	-	-
Strength III	y_p	-	1.00	1.40	-	1.00	0.50/1.20	y_{TG}	y_{SE}	-	-	-	-
Strength IV EH, EV, ES, DW, and DC ONLY	y_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
Strength V	y_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	y_{TG}	y_{SE}	-	-	-	-
Extreme Event I	y_p	y_{EQ}	1.00	-	-	1.00	-		-	1.00	-	-	-
Extreme Event II	y_p	0.50	1.00	-	-	1.00	-		-	-	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	y_{TG}	y_{SE}	-	-	-	-
Service II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
Service III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	y_{TG}	y_{SE}	-	-	-	-
Fatigue	-	0.75	-	-	-	-	-	-	-	-	-	-	-

Table 3.4.1-2 - Load factors for permanent loads, y_p

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EL: Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure		
• Overall Stability	1.00	N/A
• Retaining Walls and Abutments	1.30	0.90
• Rigid Buried Structure	1.35	0.90
• Rigid Frames	1.95	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.50	0.90
• Flexible Metal Box Culverts		
ES: Earth Surcharge	1.50	0.75

B4. Span-to-Depth Ratios in LRFD [2.5.2.6.3]

For continuous reinforced slabs with main reinforcement parallel to traffic

$$t_{min} = \frac{S + 10}{30} \geq 0.54 \cdot ft$$

Minimum slab thickness

$$t_{min} := \max\left(\frac{L_{span} + 10 \cdot ft}{30}, 0.54 \cdot ft\right) \qquad t_{min} = 18\,in$$

Thickness of flat slab chosen.. $t_{slab} := 18 \cdot in$

C. FDOT Criteria

C1. Chapter 1 - General Requirements

General [SDG 1.1]

- The design life for bridge structures is 75 years.
- Approach slabs are considered superstructure component.
- Class II Concrete (Bridge Deck) will be used for all environmental classifications.

Criteria for Deflection only [SDG 1.2]

This provision for deflection only is not applicable, since no pedestrian loading is applied in this bridge design example.

Concrete and Environment [SDG 1.3]

The concrete cover for the slab is based on either the environmental classification **[SDG 1.4]** or the type of bridge **[SDG 4.2.1]**.

Concrete cover for the slab..

$$cover_{slab} := \begin{cases} 2 \cdot in & \text{if } L_{bridge} < 300ft \\ 2.5 \cdot in & \text{otherwise} \end{cases} \qquad cover_{slab} = 2\,in$$

Concrete cover for substructure not in contact with water

$$cover_{sub} := \begin{cases} 4 \cdot in & \text{if } Environment_{sub} = \text{"Extremely"} \\ 3 \cdot in & \text{otherwise} \end{cases} \qquad cover_{sub} = 3\,in$$

Minimum 28-day compressive strength of concrete components

Class		Location
II (Bridge Deck)	CIP Bridge Deck	$f_{c.slab} := 4.5 \cdot ksi$
IV	CIP Substructure	$f_{c.sub} := 5.5 \cdot ksi$
V (Special)	Concrete Piling	$f_{c.pile} := 6.0 \cdot ksi$

Environmental Classifications [SDG 1.4]

The environment can be classified as either "Slightly", "Moderately" or "Extremely" agressive.

Environmental classification for superstructure.................. $Environment_{super} \equiv$ "Slightly"

Environmental classification for substructure..................... $Environment_{sub} \equiv$ "Moderately"

C2. Chapter 2 - Loads and Load Factors

Dead loads [SDG 2.2]

Weight of future wearing surface

$$\rho_{fws} := \begin{cases} 15 \cdot psf & \text{if } L_{bridge} < 300ft \\ 0 \cdot psf & \text{otherwise} \end{cases} \qquad \rho_{fws} = 15\,psf$$

Weight of sacrificial milling surface, using $t_{mill} = 0\,in$

$$\rho_{mill} := t_{mill} \cdot \gamma_{conc} \qquad \rho_{mill} = 0\,psf$$

*(**Note**: See Sect. C3 [SDG 4.2] for calculation of t_{mill}).*

Seismic Provisions [SDG 2.3]

Seismic provisions for minimum bridge support length only.

Miscellaneous Loads [SDG 2.5]

ITEM	UNIT	LOAD
Traffic Railing Barrier (32 " F-Shape)	Lb / ft	421
Traffic Railing Median Barrier, (32" F- Shape)	Lb / ft	486
Traffic Railing Barrier (42 " Vertical Shape)	Lb / ft	587
Traffic Railing Barrier (32 " Vertical Shape)	Lb / ft	385
Traffic Railing Barrier (42 " F-Shape)	Lb / ft	624
Traffic Railing Barrier / Soundwall (Bridge)	Lb / ft	1008
Concrete, Structural	Lb / ft^3	150
Future Wearing Surface	Lb / ft^2	15 *
Soil, Compacted	Lb / ft^3	115
Stay-in-Place Metal Forms	Lb / ft^2	20 **

* The Future Wearing Surface allowance applies only to minor widenings or short bridges as defined in SDG Chapter 7.
** Unit load of metal forms and concrete required to fill the form flutes to be applied over the projected plan area of the metal forms

Weight of traffic railing barrier.. $w_{barrier} := 421 \cdot plf$

Weight of traffic railing median barrier............................. $w_{median.bar} := 486 \cdot plf$

Barrier / Railing Distribution for Beam-Slab Bridges [SDG 2.8]

The traffic railing barriers and median barriers will be distributed equally over the full bridge cross-section.

C3. Chapter 4 - Superstructure Concrete

General [SDG 4.1]

Correction factor for Florida limerock coarse aggregate $\phi_{limerock} := 0.9$

Unit Weight of Florida limerock concrete $w_{c.limerock} := 145 \cdot pcf$

Yield strength of reinforcing steel $f_y := 60 \cdot ksi$

Note: *Epoxy coated reinforcing not allowed on FDOT projects.*

Modulus of elasticity for slab

$$E_{c.slab} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.slab} \cdot ksi}\right) \qquad E_{c.slab} = 3475\,ksi$$

Modulus of elasticity for substructure

$$E_{c.sub} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.sub} \cdot ksi}\right) \qquad E_{c.sub} = 3841\,ksi$$

Modulus of elasticity for piles

$$E_{c.pile} := \phi_{limerock} \cdot \left(1820 \cdot \sqrt{f_{c.pile} \cdot ksi}\right) \qquad E_{c.pile} = 4012\,ksi$$

Concrete Deck Slabs [SDG 4.2]

Bridge length definition

$$BridgeType := \begin{cases} \text{"Short"} & \text{if } L_{bridge} < 300ft \\ \text{"Long"} & \text{otherwise} \end{cases} \qquad BridgeType = \text{"Short"}$$

Thickness of sacrificial milling surface

$$t_{mill} \equiv \left(\begin{cases} 0 \cdot in & \text{if } L_{bridge} < 300ft \\ 0.5 \cdot in & \text{otherwise} \end{cases} \right) \qquad t_{mill} = 0\,in$$

Slab thickness $t_{slab} = 18\,in$

C4. Chapter 6 - Superstructure Components

Temperature Movement [SDG 6.3]

Structural Material of Superstructure	Temperature (Degrees Fahrenheit) Mean	High	Low	Range
Concrete Only	70	95	45	50
Concrete Deck on Steel Girder	70	110	30	80
Steel Only	70	120	30	90

The temperature values for "Concrete Only" in the preceding table apply to this example.

Temperature mean.. $t_{mean} := 70 \cdot °F$

Temperature high.. $t_{high} := 95 \cdot °F$

Temperature low.. $t_{low} := 45 \cdot °F$

Temperature rise

$\Delta t_{rise} := t_{high} - t_{mean}$ $\Delta t_{rise} = 25\,°F$

Temperature fall

$\Delta t_{fall} := t_{mean} - t_{low}$ $\Delta t_{fall} = 25\,°F$

Coefficient of thermal expansion **[LRFD 5.4.2.2]** for normal weight concrete... $\alpha_t := \dfrac{6 \cdot 10^{-6}}{°F}$

Expansion Joints [SDG 6.4]

Joint Type	Maximum Joint Width *
Poured Rubber	¾"
Silicone Seal	2"
Strip Seal	3"
Modular Joint	Unlimited
Finger Joint	Unlimited

*Joints in sidewalks must meet all requirements of Americans with Disabilities Act.

For new construction, use only the joint types listed in the preceding table. A typical joint for C.I.P. flat slab bridges is the silicone seal.

Maximum joint width.. $W_{max} := 2 \cdot in$

Minimum joint width at 70º F.................................... $W_{min} := \dfrac{5}{8} \cdot in$

Proposed joint width at 70º F.................................... $W := 1 \cdot in$

Movement [6.4.2]

For concrete structures, the movement is based on the greater of the following combinations:

Movement from the combination of temperature fall, creep, and shrinkage...

$\Delta x_{fall} = \Delta x_{temperature.fall} + \Delta x_{creep.shrinkage}$

*(**Note**: A temperature rise with creep and shrinkage is not investigated since they have opposite effects).*

Movement from factored effects of temperature..........

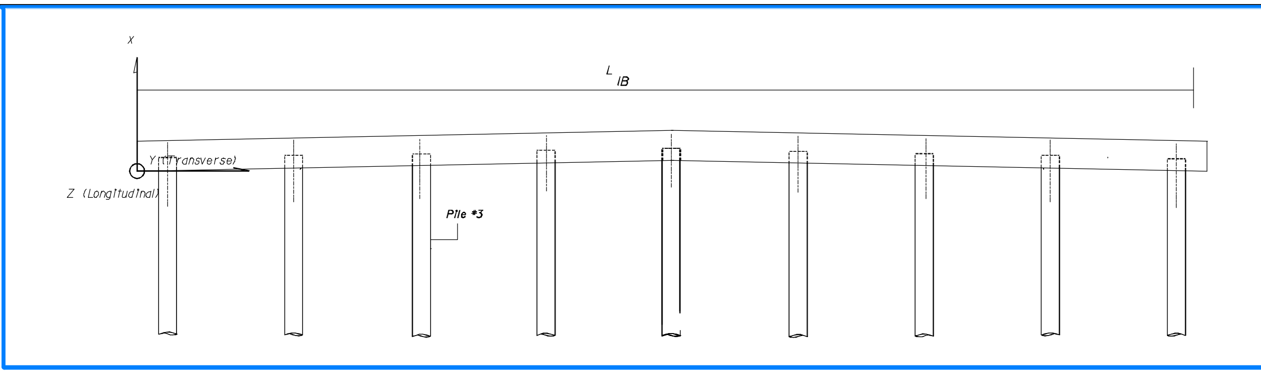
$\Delta x_{rise} = 1.15 \cdot \Delta x_{temperature.rise}$

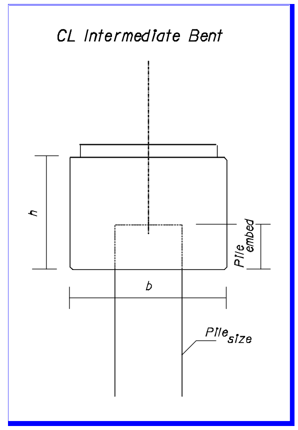
$\Delta x_{fall} = 1.15 \cdot \Delta x_{temperature.fall}$

*(**Note**: For concrete structures, the temperature rise and fall ranges are the same.*

D. Substructure

D1. Bent 2 Geometry (Bent 3 similar)





Depth of intermediate bent cap..... $h := 2.5 \cdot ft$

Width of intermediate bent cap..... $b := 3.5 \cdot ft$

Length of intermediate bent cap.... $L := 102.86 \cdot ft$

Pile Embedment Depth................ $Pile_{embed} := 12 \cdot in$

Pile Size................................... $Pile_{size} := 18 \cdot in$

*(**Note**: For this design example, only the intermediate bent will be evaluated).*

▶ Defined Units

SUPERSTRUCTURE DESIGN

Design Loads

References

Reference:G:\computer_support\StructuresSoftware\StructuresManual\HDRDesignExamples\Ex2_FlatSlab\103DsnPar.mcd(R)

Description

This section provides the design loads for the flat slab superstructure

Page Contents

LRFD Criteria

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

$WA = 0$ For superstructure design, water load and stream pressure are not applicable.

$FR = 0$ No friction forces.

$$Strength1 = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot LL + 0.50 \cdot (TU + CR + SH)$$

STRENGTH II - Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

"Permit vehicles are not evaluated in this design example"

SERVICE I - Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

$BR, WL = 0$ For superstructure design, braking forces and wind on live load are not applicable.

$CR, SH = 0$ Creep and shrinkage is not evaluated in this design example.

$$Service1 = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot LL$$

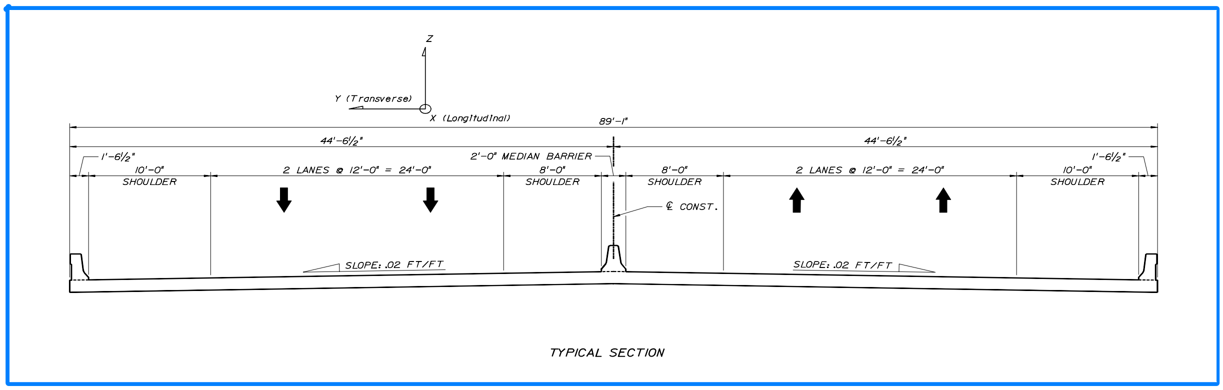
FATIGUE - Fatigue load combination relating to repetitive gravitational vehicular live load under a single design truck.

$$Fatigue = 0.75 \cdot LL$$

Note:

- **LRFD** Commentary **C4.6.2.1.6** states that "past practice has been not to check shear in typical decks... It is not the intent to check shear in every deck." In addition, **LRFD 5.14.4.1** states that for cast-in-place slab superstructures designed for moment in conformance with **LRFD 4.6.2.3**, may be considered satisfactory for shear.
- For this design example, shear will not be investigated. From previous past experience, if the slab thickness is chosen according to satisfy LRFD minimum thickness requirements as per the slab to depth ratios and designed utilizing the distribution strips, shear will not control. If special vehicles are used in the design, shear may need to be investigated.

A. Input Variables



Bridge design span length........................ $L_{span} = 35\,ft$

Thickness of superstructure slab.............. $t_{slab} = 18\,in$

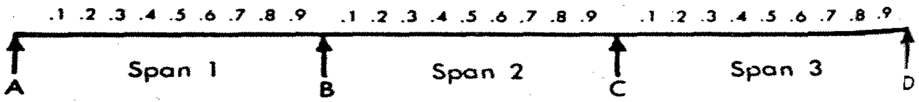
Milling surface thickness......................... $t_{mill} = 0\,in$

Dynamic Load Allowance........................ $IM = 1.33$

Bridge skew.. $Skew = -30\,deg$

B. Dead Load Analysis

There are numerous programs and charts that can be used to calculate the dead load moments on this type of structure. For the dead load calculation, the influence line coordinates for a uniform load applied on the structure is utilized. The influence coordinates are based on AISC's Moments, Shears and Reactions for Continuous Highway Bridges, published 1966.



Bridge Length =	105	ft	
Bridge Width =	89.0833	ft	
# of Traffic Barriers =	2	each	
# of Median Barriers =	1	each	
No. of spans =	3	each	
End Span Lengths =	35.000	ft	
Interior Span Lengths =	35.000		
Concrete Weight (DC) =	0.150	kcf	
Traffic Railing Barrier (DC) =	0.418	klf	
Median Barrier (DC) =	0.483	klf	
Wearing Surface and/or fws (DW) =	0.015	ksf	
Barriers & Median (DC) =	0.0148	ksf	= [(2 x 0.418 klf) + (1 x 0.483 klf)] / 89.0833 ft = 0.0148 ksf
18 in = Thickness Bridge Slab (DC) =	0.225	ksf	= 18 in. / 12) x 0.15 kcf = 0.225 ksf
Additional Misc Loads (DC)	0.000		
Components & Attachments (DC) =	0.240	ksf	= 0.0148 ksf + 0.225 ksf+ 0 ksf = 0.24 ksf

span ratio = 1.00

Use tables 1.0 and 1.1

(From "Moments, Shears and Reactions for Continuous Highway Bridges" published by AISC, 1966)

Pt.	AISC Table	Influence Line Coordinates 1.0	1.1	DC MOMENTS (FT-KIP/FT)	DW MOMENTS (FT-KIP/FT)	DC SHEARS (KIP/FT)	DW SHEARS (KIP/FT)
0	A	0.0000	0.0000	0.0	0.0	3.4	0.2
1	0.1	0.0350	0.0340	10.3	0.6	2.5	0.2
2	0.2	0.0600	0.0580	17.6	1.1	1.7	0.1
3	0.3	0.0750	0.0720	22.0	1.4	0.8	0.1
4	0.4	0.0800	0.0760	23.5	1.5	0.0	0.0
5	0.5	0.0750	0.0700	22.0	1.4	-0.8	-0.1
6	0.6	0.0600	0.0540	17.6	1.1	-1.7	-0.1
7	0.7	0.0350	0.0280	10.3	0.6	-2.5	-0.2
8	0.8	0.0000	-0.0080	0.0	0.0	-3.4	-0.2
9	0.9	-0.0450	-0.0540	-13.2	-0.8	-4.2	-0.3
10	B	-0.1000	-0.1100	-29.4	-1.8	-5.0	-0.3
	B	-0.1000	-0.1100	-29.4	-1.8	4.2	0.3
11	1.1	-0.0550	-0.0555	-16.2	-1.0	3.1	0.2
12	1.2	-0.0200	-0.0132	-5.9	-0.4	2.1	0.1
13	1.3	0.0050	0.0171	1.5	0.1	1.0	0.1
14	1.4	0.0200	0.0352	5.9	0.4	0.0	0.0
15	1.5	0.0250	0.0413	7.3	0.5	0.0	0.0
16	1.6	0.0200	0.0352	5.9	0.4	-0.7	0.0
17	1.7	0.0050	0.0171	1.5	0.1	-1.4	-0.1
18	1.8	-0.0200	-0.0132	-5.9	-0.4	-2.1	-0.1
19	1.9	-0.0550	-0.0555	-16.2	-1.0	-2.8	-0.2
	C	-0.1000	-0.1100	-29.4	-1.8	-4.2	-0.3
20	C	-0.1000	-0.1100	-29.4	-1.8	5.0	0.3
21	2.1	-0.0450	-0.0540	-13.2	-0.8	4.2	0.3
22	2.2	0.0000	-0.0080	0.0	0.0	3.4	0.2
23	2.3	0.0350	0.0280	10.3	0.6	2.5	0.2
24	2.4	0.0600	0.0540	17.6	1.1	1.7	0.1
25	2.5	0.0750	0.0700	22.0	1.4	0.8	0.1
26	2.6	0.0800	0.0760	23.5	1.5	0.0	0.0
27	2.7	0.0750	0.0720	22.0	1.4	-0.8	-0.1
28	2.8	0.0600	0.0580	17.6	1.1	-1.7	-0.1
29	2.9	0.0350	0.0340	10.3	0.6	-2.5	-0.2
30	D	0.0000	0.0000	0.0	0.0	-3.4	-0.2

C. Approximate Methods of Analysis - Decks [LRFD 4.6.2]

C1. Equivalent Strip Widths for Slab-type Bridges [LRFD 4.6.2.3]

The superstructure is designed on a per foot basis longitudinally. However, in order to distribute the live loads, equivalent strips of flat slab deck widths are calculated. The moment and shear effects of a single HL-93 vehicle or multiple vehicles are divided by the appropriate equivalent strip width. The equivalent strips account for the transverse distribution of LRFD wheel loads. This section is only applicable for spans greater than **15 feet**.

One design lane

The equivalent width of longitudinal strips per lane for both shear and moment with one lane loaded:

$$E = 10 + 5.0\sqrt{L_1 \cdot W_1}$$

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet.................................... $L_1 := \min(L_{span}, 60.0 \cdot ft)$

$L_1 = 35\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 30 feet for single lane loading.. $W_1 := \min(W_{bridge}, 30.0 \cdot ft)$

$W_1 = 30\,ft$

The equivalent distribution width for one lane loaded is given as... $E_{OneLane} := \left(10 + 5.0\sqrt{\frac{L_1}{ft} \cdot \frac{W_1}{ft}}\right) \cdot in$

$E_{OneLane} = 172.0\,in$ or $E_{OneLane} = 14.3\,ft$

Two or more design lanes

The equivalent width of longitudinal strips per lane for both shear and moment with more than one lane loaded:

$$E = 84 + 1.44\sqrt{L_1 \cdot W_1} \le \frac{12.0W}{N_L}$$

where

L_1, modified span length.. $L_1 = 35\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 60 feet for multilane loading.. $W_1 := \min(W_{bridge}, 60.0 \cdot ft)$

$W_1 = 60\,ft$

Since the bridge is crowned and the full width of the bridge is used in the equivalent distribution width equation, the number of design lanes should include both roadways. Therefore, number of design lanes..... $N_L := 2 \cdot N_{lanes}$

$N_L = 6$

The equivalent distribution width for more than one lane loaded is given as.................................

$$E_{TwoLane} := \min\left[\left(84 + 1.44\sqrt{\frac{L_1}{ft}\frac{W_1}{ft}}, \frac{12.0\left(\frac{W_{bridge}}{ft}\right)}{N_L}\right)\right]\cdot in$$

$E_{TwoLane} = 150.0\ in$ or $E_{TwoLane} = 12.5\ ft$

The design strip width to use would be the one that causes the maximum effects. In this case, it would be the minimum value of the two..

$$E := \min\left(E_{OneLane}, E_{TwoLane}\right)$$

$E = 150.0\ in$ or $E = 12.5\ ft$

Skew modification

For skewed bridges, the longitudinal force effects (moments only) **may** be reduced by a factor r...............

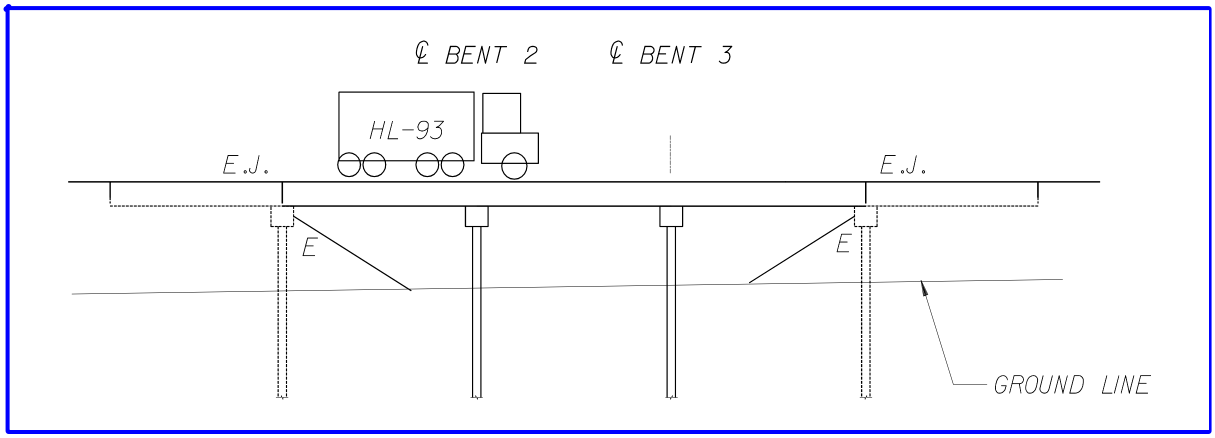
$$r := \min(1.05 - 0.25\cdot\tan(|Skew|), 1.00)$$

$r = 0.91$

*(**Note**: For this design example, the skew modification will not be applied in order to design for more conservative moment values)*

C2. Live Load Analysis

Determine the live load moments and shears due to one HL-93 vehicle on the continuous flat slab structure. The design live loads will consists of the HL-93 vehicle moments, divided by the appropriate equivalent strip widths. This will result in a design live load per foot width of flat slab.



In order to calculate the live load moments and shears, the FDOT MathCad program "LRFD Live Load Generator, English, v2.1".

Read Live Load results from files generated by FDOT Program

The files generated by the program are as follows: ("service1.txt" "fatigue.txt"). These files are output files that can be used to transfer information from one file to another via read and write commands in MathCad.

The files can be view by clicking on the following icons:

service1.txt fatigue.txt

To data is read from the file created by FDOT MathCad program "LRFD Live Load Generator" program.

The values for Strength I can be obtained by multiplying by the appropriate load case factor. The values of Live Load for the HL-93 loads are as follows:

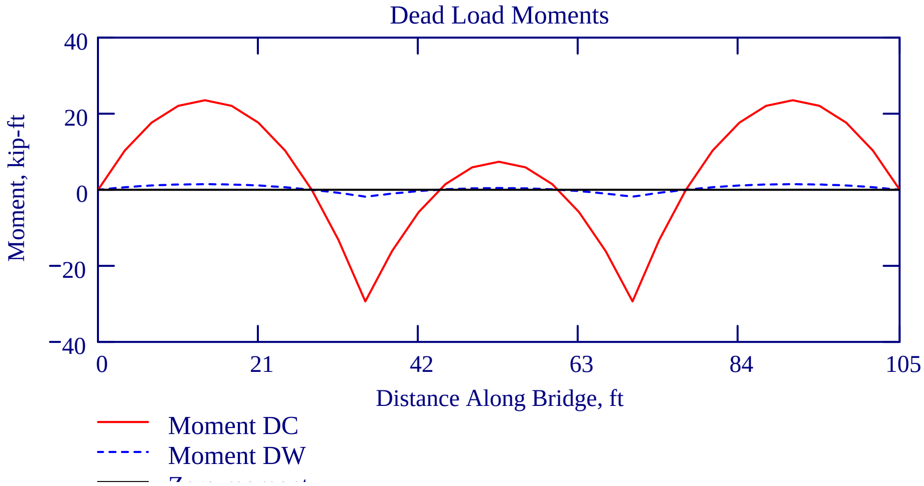
HL-93 Live Load Envelopes

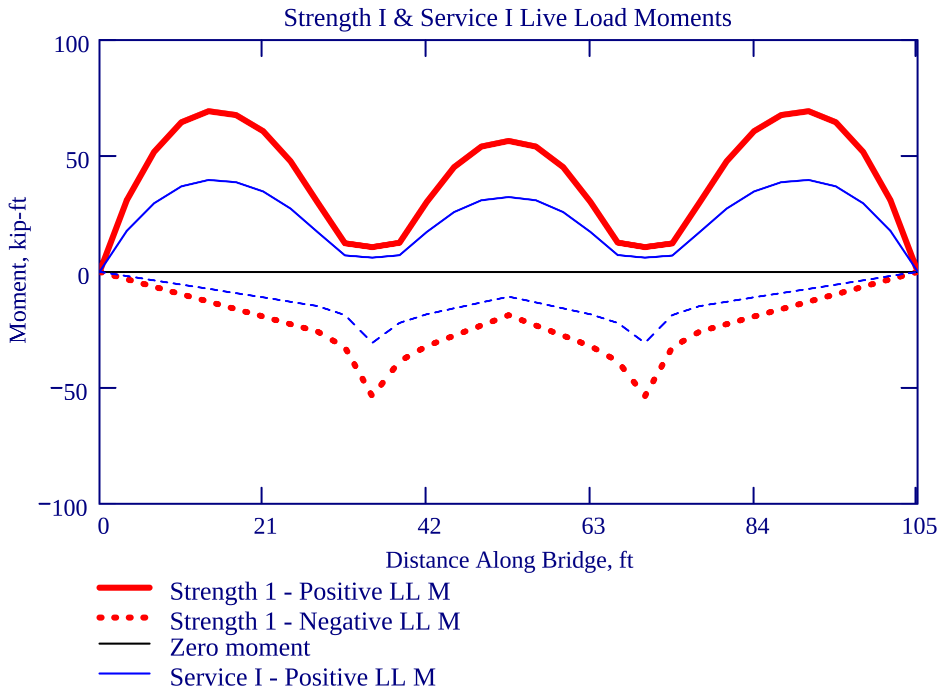
	(10th points)	Service I		Strength I		Fatigue		
Pt.	"X" distance	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5
2	7	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5
3	10.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8
4	14	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3
5	17.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3
6	21	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8
7	24.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5
8	28	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0
9	31.5	88.1	-232.9	154.2	-407.6	39.8	-117.9	157.7
10	35	76.1	-383.5	133.2	-671.1	27.0	-186.9	213.8
11	38.5	89.5	-275.7	156.7	-482.5	48.7	-122.2	170.8
12	42	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8
13	45.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8
14	49	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5
15	52.5	403.4	-133.9	706.0	-234.3	134.4	-40.5	174.9
16	56	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5
17	59.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8
18	63	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8
19	66.5	90.1	-275.7	157.6	-482.5	48.7	-122.2	170.8
20	70	76.1	-383.0	133.2	-670.3	27.0	-186.9	213.8
21	73.5	87.5	-232.9	153.1	-407.6	39.8	-117.9	157.7
22	77	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0
23	80.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5
24	84	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8
25	87.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3
26	91	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3
27	94.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8
28	98	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5
29	101.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0

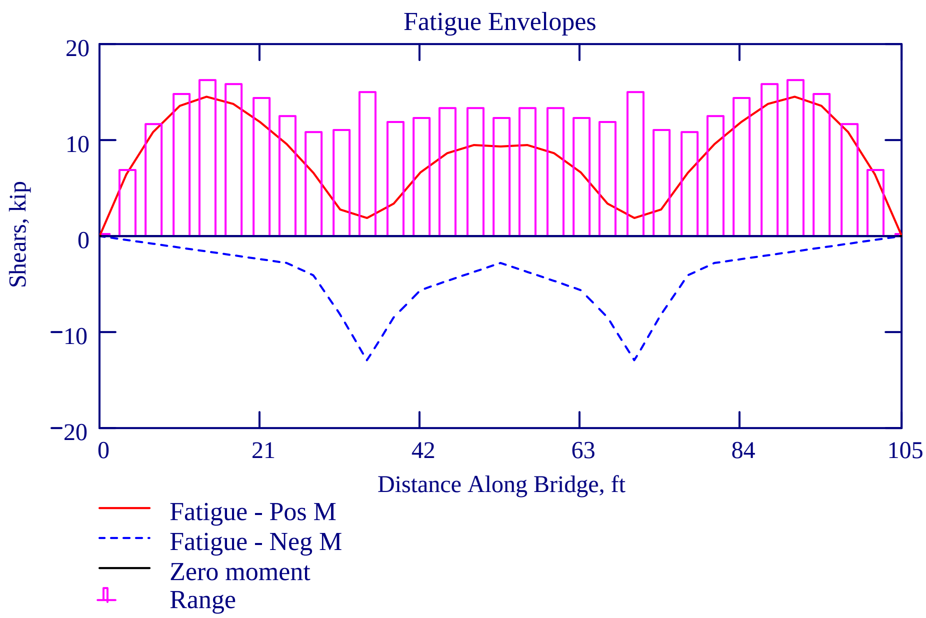
The design values can be obtained by dividing the moments by the distribution width, $E = 12.5ft$; for fatigue, $E_{OneLane} = 14.3ft$

Design Live Load Envelopes									
						E =	12.5 ft		
						$E_{fatigue}$ =	14.3 ft		
	(10th points)	Service I		Strength I		Fatigue			
Joint	"X" distance	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	17.7	-1.8	30.9	-3.2	6.4	-0.4	6.8
2	7	29.6	-3.7	51.7	-6.4	10.8	-0.8	11.6
3	10.5	36.9	-5.5	64.5	-9.7	13.6	-1.2	14.8
4	14	39.6	-7.4	69.3	-12.9	14.5	-1.6	16.1
5	17.5	38.6	-9.2	67.6	-16.1	13.8	-2.0	15.8
6	21	34.6	-11.0	60.6	-19.3	11.9	-2.4	14.3
7	24.5	27.2	-12.9	47.7	-22.6	9.6	-2.8	12.4
8	28	17.1	-14.8	29.9	-25.8	6.6	-4.1	10.7
9	31.5	7.1	-18.6	12.3	-32.6	2.8	-8.2	10.9
10	35	6.1	-30.7	10.7	-53.7	1.9	-13.0	14.8
11	38.5	7.2	-22.1	12.5	-38.6	3.4	-8.5	11.8
12	42	17.2	-18.3	30.1	-32.0	6.6	-5.6	12.3
13	45.5	25.8	-15.7	45.1	-27.5	8.6	-4.7	13.3
14	49	30.9	-13.2	54.1	-23.2	9.5	-3.7	13.2
15	52.5	32.3	-10.7	56.5	-18.7	9.3	-2.8	12.1
16	56	30.9	-13.2	54.1	-23.2	9.5	-3.7	13.2
17	59.5	25.8	-15.7	45.1	-27.5	8.6	-4.7	13.3
18	63	17.2	-18.3	30.1	-32.0	6.6	-5.6	12.3
19	66.5	7.2	-22.1	12.6	-38.6	3.4	-8.5	11.8
20	70	6.1	-30.6	10.7	-53.6	1.9	-13.0	14.8
21	73.5	7.0	-18.6	12.2	-32.6	2.8	-8.2	10.9
22	77	17.1	-14.8	29.9	-25.8	6.6	-4.1	10.7
23	80.5	27.2	-12.9	47.7	-22.6	9.6	-2.8	12.4
24	84	34.6	-11.0	60.6	-19.3	11.9	-2.4	14.3
25	87.5	38.6	-9.2	67.6	-16.1	13.8	-2.0	15.8
26	91	39.6	-7.4	69.3	-12.9	14.5	-1.6	16.1
27	94.5	36.9	-5.5	64.5	-9.7	13.6	-1.2	14.8
28	98	29.6	-3.7	51.7	-6.4	10.8	-0.8	11.6
29	101.5	17.7	-1.8	30.9	-3.2	6.4	-0.4	6.8
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0

$i := 0 .. rows(X) - 1$


Dead Load Moments
40
20
0
−20
−40
Moment, kip-ft
0
21
42
63
84
105
Distance Along Bridge, ft
Moment DC
Moment DW


Strength I & Service I Live Load Moments
100
50
0
−50
−100
Moment, kip-ft
0
21
42
63
84
105
Distance Along Bridge, ft
Strength 1 - Positive LL M
Strength 1 - Negative LL M
Zero moment
Service I - Positive LL M


Fatigue Envelopes
20
10
0
−10
−20
Shears, kip
0
21
42
63
84
105
Distance Along Bridge, ft
Fatigue - Pos M
Fatigue - Neg M
Zero moment
Range

C3. Limit State Moments and Shears

The service and strength limit states used to design the section are calculated as follows:

Limit State Design Loads

Pt.	"X" dist (10th points)	Service I 1.0DC + 1.0DW + 1.0LL +M	Service I -M	Strength I 1.25DC + 1.50DW + 1.75LL +M	Strength I -M	Fatigue 1.0DC + 1.0DW + 1.5LL +M	Fatigue -M	MRange = 0.75LL; M_{Range}	$-M_{min}$ = 0.75LL
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	28.6	9.1	44.7	10.6	20.6	10.3	6.8	-0.4
2	7	48.3	15.1	75.4	17.3	35.0	17.5	11.6	-0.8
3	10.5	60.3	17.9	94.1	20.0	43.8	21.6	14.8	-1.2
4	14	64.6	17.6	100.9	18.7	46.7	22.6	16.1	-1.6
5	17.5	62.0	14.2	97.2	13.5	44.1	20.4	15.8	-2.0
6	21	53.4	7.7	84.3	4.4	36.5	15.1	14.3	-2.4
7	24.5	38.2	-2.0	61.5	-8.8	25.3	6.7	12.4	-2.8
8	28	17.1	-14.8	29.9	-25.8	9.9	-6.1	10.7	-4.1
9	31.5	-7.0	-32.7	-5.4	-50.4	-9.9	-26.3	10.9	-8.2
10	35	-25.1	-61.9	-28.8	-93.2	-28.4	-50.7	14.8	-13.0
11	38.5	-10.0	-39.2	-9.2	-60.3	-12.1	-29.9	11.8	-8.5
12	42	11.0	-24.5	22.2	-39.9	3.7	-14.7	12.3	-5.6
13	45.5	27.4	-14.2	47.1	-25.6	14.5	-5.5	13.3	-4.7
14	49	37.1	-7.0	61.9	-15.3	20.5	0.6	13.2	-3.7
15	52.5	40.1	-2.9	66.3	-8.9	21.8	3.6	12.1	-2.8
16	56	37.1	-7.0	61.9	-15.3	20.5	0.6	13.2	-3.7
17	59.5	27.4	-14.2	47.1	-25.6	14.5	-5.5	13.3	-4.7
18	63	11.0	-24.5	22.2	-39.9	3.7	-14.7	12.3	-5.6
19	66.5	-10.0	-39.2	-9.1	-60.3	-12.1	-29.9	11.8	-8.5
20	70	-25.1	-61.9	-28.8	-93.1	-28.4	-50.7	14.8	-13.0
21	73.5	-7.0	-32.7	-5.5	-50.4	-9.9	-26.3	10.9	-8.2
22	77	17.1	-14.8	29.9	-25.8	9.9	-6.1	10.7	-4.1
23	80.5	38.2	-2.0	61.5	-8.8	25.3	6.7	12.4	-2.8
24	84	53.4	7.7	84.3	4.4	36.5	15.1	14.3	-2.4
25	87.5	62.0	14.2	97.2	13.5	44.1	20.4	15.8	-2.0
26	91	64.6	17.6	100.9	18.7	46.7	22.6	16.1	-1.6
27	94.5	60.3	17.9	94.1	20.0	43.8	21.6	14.8	-1.2
28	98	48.3	15.1	75.4	17.3	35.0	17.5	11.6	-0.8
29	101.5	28.6	9.1	44.7	10.6	20.6	10.3	6.8	-0.4
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Maximum negative Moments =			-61.9		-93.2	*46.7*		*16.1*	*-1.6*
Maximum positive Moments =		64.6		100.9			*-50.7*	*14.8*	*-13.0*

<−Maximum positive moment and corresponding fatigue values

<−Maximum negative moment and corresponding fatigue values

Defined Units

Flat Slab Design

References

Reference:G:\computer_support\StructuresSoftware\StructuresManual\HDRDesignExamples\Ex2_FlatSlab\201DesignLds.mc

Description

This section provides the design for the flat slab superstructure.

Page Contents

A. Input Variables

	Service	Strength	Fatigue
Maximum positive moment and corresponding fatigue values	$M_{pos} = 64.6\,ft\cdot kip$	$M_{r.pos} = 100.9\,ft\cdot kip$	$M_{fatigue.pos} = 46.7\,ft\cdot kip$ $M_{range.pos} = 16.1\,ft\cdot kip$ $M_{min.pos} = -1.6\,ft\cdot kip$
Maximum negative moment and corresponding fatigue values	$M_{neg} = -61.9\,ft\cdot kip$	$M_{r.neg} = -93.2\,ft\cdot kip$	$M_{fatigue.neg} = -50.7\,ft\cdot kip$ $M_{range.neg} = 14.8\,ft\cdot kip$ $M_{min.neg} = -13\,ft\cdot kip$

B. Moment Design

The design procedure consists of calculating the reinforcement required to satisfy the design moment, then checking this reinforcement against criteria for crack control, minimum reinforcement, maximum reinforcement, shrinkage and temperature reinforcement, and distribution of reinforcement. The procedure is the same for both positive and negative moment regions.

B1. Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Factored resistance $M_r = \phi \cdot M_n$

Nominal flexural resistance

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) - A'_s \cdot f'_y \cdot \left(d'_s - \frac{a}{2}\right) + 0.85 \cdot f'_c \cdot (b - b_w) \cdot \beta_1 \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Simplifying the nominal flexural resistance

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \text{where} \qquad a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Using variables defined in this example..... $M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

where

$M_{r.pos} = 100.9\ \text{ft} \cdot \text{kip}$

$f_{c.slab} = 4.5\ \text{ksi}$

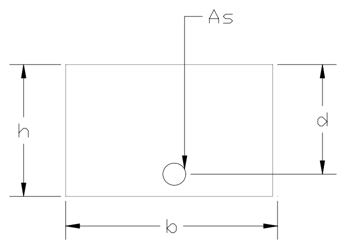
$f_y = 60\ \text{ksi}$

$\phi = 0.9$

$t_{slab} = 18\ \text{in}$

$b := 1 \cdot \text{ft}$

$t_{slab} = h$



$b = 12\ \text{in}$

Initial assumption for area of steel required

Size of bar...................................... bar := "8"

Proposed bar spacing........................ $spacing_{pos} := 6 \cdot \text{in}$

Bar area.. $A_{bar} = 0.790\ in^2$

Bar diameter.................................... $dia = 1.000\ in$

Area of steel provided per foot of slab....... $A_{s.pos} := \dfrac{A_{bar} \cdot 1ft}{spacing_{pos}}$

$A_{s.pos} = 1.58\ in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel.................... $d_{s.pos} := t_{slab} - cover_{slab} - \dfrac{dia}{2}$

$d_{s.pos} = 15.5\ in$

Solve the quadratic equation for the area of steel required

Given $\quad M_{r.pos} = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_{s.pos} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

Reinforcing steel required....................... $A_{s.reqd} := Find(A_{s.pos})$

$A_{s.reqd} = 1.55\ in^2$

The area of steel provided, $A_{s.pos} = 1.58\ in^2$, should be greater than the area of steel required, $A_{s.reqd} = 1.55\ in^2$. If not, decrease the spacing of the reinforcement. Once $A_{s.pos}$ is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the design moments.

Moment capacity provided...................... $M_{r.positive.prov} := \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_{s.pos} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

$M_{r.positive.prov} = 102.9\ ft \cdot kip$

B2. Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Variables:

$|M_{r.neg}| = 93.167\ ft \cdot kip$

$f_{c.slab} = 4.5\ ksi$

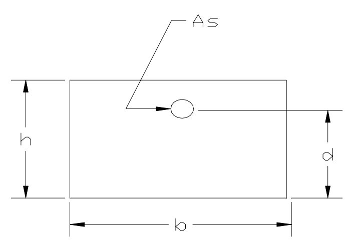
$f_y = 60\ ksi$

$\phi = 0.9$

$t_{slab} = 18\ in$

$b = 1\ ft$

$t_{slab} = h$



$b = 12\ in$

Initial assumption for area of steel required

Size of bar..................................... $bar_{neg} := "8"$

Proposed bar spacing...................... $spacing_{neg} := 6 \cdot in$

Bar area.. $A_{bar.neg} = 0.790\ in^2$

Bar diameter................................... $dia_{neg} = 1.000\ in$

Area of steel provided per foot of slab...... $A_{s.neg} := \dfrac{A_{bar.neg} \cdot 1ft}{spacing_{neg}}$

$A_{s.neg} = 1.58\ in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel................ $d_{s.neg} := \left(-t_{slab} + cover_{slab} + \dfrac{dia_{neg}}{2}\right)$

$d_{s.neg} = -15.5\ in$

Solve the quadratic equation for the area of steel required

Given $$M_{r.neg} = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_{s.neg} + \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$$

Reinforcing steel required....................... $A_{s.reqd} := Find(A_{s.neg})$

$A_{s.reqd} = 1.42\ in^2$

The area of steel provided, $A_{s.neg} = 1.58\ in^2$, should be greater than the area of steel required, $A_{s.reqd} = 1.42\ in^2$. If not, decrease the spacing of the reinforcement. Once $A_{s.neg}$ is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the design moments.

Moment capacity provided...................... $M_{r.negative.prov} := \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_{s.neg} + \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

$M_{r.negative.prov} = -102.9\ ft \cdot kip$

B3. Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

Stress in the mild steel reinforcement at the service limit state.................................. $$f_{sa} = \frac{z}{(d_c \cdot A)^{\frac{1}{3}}} \le 0.6 \cdot f_y$$

Crack width parameter........................... $$z = \begin{pmatrix} \text{"moderate exposure"} & 170 \\ \text{"severe exposure"} & 130 \\ \text{"buried structures"} & 100 \end{pmatrix} \cdot \frac{kip}{in}$$

The environmental classifications for Florida designs do not match the classifications to select the crack width parameter. For this example, a "Slightly" or "Moderately" aggressive environment corresponds to "moderate exposure" and an "Extremely" aggressive environment corresponds to "severe exposure".

$Environment_{super} = \text{"Slightly"}$ aggressive environment

$$z := 170 \cdot \frac{kip}{in}$$

Positive Moment

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)...................................

$$d_c := \min\left(t_{slab} - d_{s.pos}, 2 \cdot in + \frac{dia}{2}\right)$$

$d_c = 2.5\,in$

Number of bars per design width of slab...

$$n_{bar} := \frac{b}{spacing_{pos}}$$

$n_{bar} = 2$

Effective tension area of concrete surrounding the flexural tension reinforcement.......................................

$$A := \frac{(b) \cdot (2 \cdot d_c)}{n_{bar}}$$

$A = 30.0\,in^2$

Service limit state stress in reinforcement..

$$f_{sa} := \min\left[\frac{z}{(d_c \cdot A)^{\frac{1}{3}}}, 0.6 \cdot f_y\right]$$

$f_{sa} = 36.0\,ksi$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 4.8 \cdot in$

Given $$\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.slab}} \cdot A_{s.pos} \cdot (d_{s.pos} - x)$$

$x_{na.pos} := \text{Find}(x)$

$x_{na.pos} = 4.8\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 4.8\,in$ to equal $x_{na.pos} = 4.8\,in$.

Tensile force in the reinforcing steel due to service limit state moment......................

$$T_s := \frac{M_{pos}}{d_{s.pos} - \frac{x_{na.pos}}{3}}$$

$T_s = 55.8\,kip$

Actual stress in the reinforcing steel due to service limit state moment......................

$$f_{s.actual} := \frac{T_s}{A_{s.pos}}$$

$f_{s.actual} = 35.3\,ksi$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$$LRFD_{5.7.3.3.4a} := \begin{cases} \text{"OK, crack control for +M is satisfied"} & \text{if } f_{s.actual} \le f_{sa} \\ \text{"NG, crack control for +M not satisfied, provide more reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.4a} = \text{"OK, crack control for +M is satisfied"}$

Negative Moment

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)..................................

$$d_c := \min\left(t_{slab} + d_{s.neg}, 2 \cdot in + \frac{dia_{neg}}{2}\right)$$

$d_c = 2.5\,in$

Number of bars per design width of slab..

$$n_{bar} := \frac{b}{spacing_{neg}}$$

$n_{bar} = 2$

Effective tension area of concrete surrounding the flexural tension reinforcement......................................

$$A := \frac{(b) \cdot (2 \cdot d_c)}{n_{bar}}$$

$A = 30.0\,in^2$

Service limit state stress in reinforcement..

$$f_{sa} := \min\left[\frac{z}{(d_c \cdot A)^{\frac{1}{3}}}, 0.6 \cdot f_y\right]$$

$f_{sa} = 36.0\,ksi$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 4.8 \cdot in$

Given $$\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.slab}} \cdot A_{s.neg} \cdot (-d_{s.neg} - x)$$

$x_{na.neg} := Find(x)$

$x_{na.neg} = 4.84\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 4.8\,in$ to equal $x_{na.neg} = 4.8\,in$.

Tensile force in the reinforcing steel due to service limit state moment........................

$T_s = 53.5\,kip$

$$T_s := \frac{M_{neg}}{d_{s.neg} + \frac{x_{na.neg}}{3}}$$

Actual stress in the reinforcing steel due to service limit state moment.....................

$f_{s.actual} = 33.9\,ksi$

$$f_{s.actual} := \frac{T_s}{A_{s.neg}}$$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$LRFD_{5.7.3.3.4b} :=$ | "OK, crack control for -M is satisfied" if $f_{s.actual} \le f_{sa}$
"NG, crack control for -M not satisfied, provide more reinforcement" otherwise

$LRFD_{5.7.3.3.4b} =$ "OK, crack control for -M is satisfied"

B4. Limits for Reinforcement [LRFD 5.7.3.3]

Maximum Reinforcement

The maximum reinforcement requirements ensure the section has sufficient ductility and is not overreinforced. The greater reinforcement from the positive and negative moment sections is checked.

Area of steel provided............................

$A_{s.pos} = 1.58\,in^2$

$A_{s.neg} = 1.58\,in^2$

Stress block factor................................

$$\beta_1 := \max\left[0.85 - 0.05\cdot\left(\frac{f_{c.slab} - 4000\cdot psi}{1000\cdot psi}\right), 0.65\right]$$

$\beta_1 = 0.825$

Distance from extreme compression fiber to the neutral axis of section......................

$$c_{pos} := \frac{A_{s.pos}\cdot f_y}{0.85\cdot f_{c.slab}\cdot\beta_1\cdot b} \quad \text{and} \quad c_{neg} := \frac{A_{s.neg}\cdot f_y}{0.85\cdot f_{c.slab}\cdot\beta_1\cdot b}$$

$c_{pos} = 2.5\,in$

$c_{neg} = 2.5\,in$

Effective depth from extreme compression fiber to centroid of the tensile reinforcement.

$$d_e = \frac{A_s\cdot f_{ps}\cdot d_p + A_s\cdot f_y\cdot d_s}{A_{ps}\cdot f_{ps} + A_s\cdot f_y}$$

Simplifying for this example.............

$$d_{e.pos} := d_{s.pos} \quad \text{and} \quad d_{e.neg} := -d_{s.neg}$$

$d_{e.pos} = 15.5\,in$

$d_{e.neg} = 15.5\,in$

The $\frac{c_{pos}}{d_{e.pos}} = 0.162$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$$LRFD_{5.7.3.3.1} := \begin{cases} \text{"OK, maximum reinforcement in +M region"} & \text{if } \frac{c_{pos}}{d_{e.pos}} \le 0.42 \\ \text{"NG, section is over-reinforced in +M region, see LRFD eq. C5.7.3.3.1-1"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement in +M region"

The $\frac{c_{neg}}{d_{e.neg}} = 0.162$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$$LRFD_{5.7.3.3.1} := \begin{cases} \text{"OK, maximum reinforcement in -M region"} & \text{if } \frac{c_{neg}}{d_{e.neg}} \le 0.42 \\ \text{"NG, section is over-reinforced in -M region, see LRFD eq. C5.7.3.3.1-1"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement in -M region"

Minimum Reinforcement

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture............................... $f_r := 0.24 \cdot \sqrt{f_{c.slab} \cdot ksi}$

$f_r = 509.1\ psi$

Section modulus.................................... $S := \frac{b \cdot t_{slab}^{2}}{6}$

$S = 648.0\ in^3$

Cracking moment.................................. $M_{cr} := f_r \cdot S$

$M_{cr} = 27.5\ kip \cdot ft$

Required flexural resistance (+M)............ $M_{r.reqd} := \min\left(1.2 \cdot M_{cr}, 133 \cdot \% \cdot M_{r.pos}\right)$

$M_{r.reqd} = 33.0\ ft \cdot kip$

Check that the capacity provided, $M_{r.positive.prov} = 102.9 ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 33 ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{cases} \text{"OK, minimum reinforcement for positive moment is satisfied"} & \text{if } M_{r.positive.prov} \ge M_{r.reqd} \\ \text{"NG, reinforcement for positive moment is less than minimum"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for positive moment is satisfied"

Required flexural resistance (-M).............. $M_{r.reqd} := \min\left(1.2 \cdot M_{cr}, 133 \cdot \% \cdot \left|M_{r.neg}\right|\right)$

$M_{r.reqd} = 33.0\,ft \cdot kip$

Check that the capacity provided, $M_{r.negative.prov} = -102.9 ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 33 ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{cases} \text{"OK, minimum reinforcement for negative moment is satisfied"} & \text{if } M_{r.positive.prov} \geq M_{r.reqd} \\ \text{"NG, reinforcement for negative moment is less than minimum"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for negative moment is satisfied"

B5. Shrinkage and Temperature Reinforcement [LRFD 5.10.8.2]

Shrinkage and temperature reinforcement provided

Size of bar ("4" "5" "6")................... $bar_{st} :=$ "5"

Bar spacing.. $bar_{spa.st} := 9 \cdot in$

Bar area.. $A_{bar} = 0.31\,in^2$

Bar diameter................................... $dia = 0.625\,in$

Gross area of section............................ $A_g := b \cdot t_{slab}$

$A_g = 216.0\,in^2$

Minimum area of shrinkage and temperature reinforcement....................................... $A_{ST} := \dfrac{0.11 \cdot ksi \cdot A_g}{f_y}$

$A_{ST} = 0.40\,in^2$

Maximum spacing for shrinkage and temperature reinforcement...................... $spacing_{ST} := \min\left(\dfrac{b}{\dfrac{A_{ST}}{A_{bar}}}, 3 \cdot t_{slab}, 18 \cdot in\right)$

$spacing_{ST} = 9.4\,in$

The bar spacing should be less than the maximum spacing for shrinkage and temperature reinforcement

$$LRFD_{5.7.10.8} := \begin{cases} \text{"OK, minimum shrinkage and temperature requirements"} & \text{if } bar_{spa.st} \leq spacing_{ST} \\ \text{"NG, minimum shrinkage and temperature requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.10.8} =$ "OK, minimum shrinkage and temperature requirements"

B6. Distribution of Reinforcement [LRFD 5.14.4]

Transverse distribution reinforcement shall be placed in the bottom of the slab. The amount to place is based on a percentage of the longitudinal main steel.

Distribution reinforcement provided

Size of bar ("4" "5" "6")................... $bar_{dist} := "5"$

Bar spacing.................................... $bar_{spa.dist} := 12 \cdot in$

Bar area... $A_{bar} = 0.31\,in^2$

Bar diameter.................................... $dia = 0.625\,in$

The area for secondary reinforcement should not exceed 50% of the area for primary reinforcement............................

$$\%A_{steel} := \min\left(\frac{100}{\sqrt{\frac{L_{span}}{ft}}}\%, 50\%\right)$$

$\%A_{steel} = 0.17$

Required area for secondary reinforcement

$$A_{s.DistR} := A_{s.pos} \cdot \%A_{steel}$$

$A_{s.DistR} = 0.27\,in^2$

Maximum spacing for secondary reinforcement.......................................

$$MaxSpacing_{DistR} := \frac{b}{\left(\frac{A_{s.DistR}}{A_{bar}}\right)}$$

$MaxSpacing_{DistR} = 13.9\,in$

The bar spacing should not exceed the maximum spacing for secondary reinforcement

$$LRFD_{5.14.4} := \begin{cases} \text{"OK, distribution reinforcement requirements"} & \text{if } bar_{spa.dist} \le MaxSpacing_{DistR} \\ \text{"NG, distribution reinforcement requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{5.14.4} =$ "OK, distribution reinforcement requirements"

B7. Fatigue Limit State [LRFD 5.5.3]

The section properties for fatigue shall be based on cracked sections where the sum of stresses due to unfactored permanent loads and 1.5 times the fatigue load is tensile and exceeds $0.095\sqrt{f_c}$.

Allowable tensile stress for fatigue........... $f_{tensile} := 0.095\sqrt{f_{c.slab} \cdot ksi}$

$f_{tensile} = 0.202\ ksi$

Positive Moment Region

Stress due to positive moment................. $f_{fatigue.pos} := \dfrac{M_{fatigue.pos}}{S}$

$f_{fatigue.pos} = 0.866\ ksi$

$$Fatigue_{section} := \begin{cases} \text{"Use Cracked section"} & \text{if } f_{fatigue.pos} > f_{tensile} \\ \text{"Use Uncracked section"} & \text{otherwise} \end{cases}$$

$Fatigue_{section} =$ "Use Cracked section"

Minimum stress in reinforcement due to minimum live load.................................. $f_{min} := \dfrac{M_{min.pos}}{A_{s.pos} \cdot \left(d_{s.pos} - \dfrac{x_{na.pos}}{3}\right)}$

$f_{min} = -0.878\ ksi$

Ratio of r/h is taken as $r\_h := 0.3$, therefore the allowable stress range is given by....... $f_{t.allow} := (21 \cdot ksi - 0.33 \cdot f_{min}) + 8 \cdot ksi \cdot (r\_h)$

$f_{t.allow} = 23.69\ ksi$

Actual stress range................................ $f_t := \dfrac{M_{range.pos}}{A_{s.pos} \cdot \left(d_{s.pos} - \dfrac{x_{na.pos}}{3}\right)}$

$f_t = 8.813\ ksi$

$$LRFD_{5.5.3.2} := \begin{cases} \text{"OK, fatigue stress range requirement for +M region"} & \text{if } f_t \le f_{t.allow} \\ \text{"NG, fatigue stress range requirements for +M region"} & \text{otherwise} \end{cases}$$

$LRFD_{5.5.3.2} =$ "OK, fatigue stress range requirement for +M region"

Negative Moment Region

Stress due to negative moment.................

$$f_{fatigue.neg} := \frac{\left|M_{fatigue.neg}\right|}{S}$$

$f_{fatigue.neg} = 0.938\ ksi$

$$Fatigue_{section} := \begin{cases} \text{"Use Cracked section"} & \text{if } f_{fatigue.neg} > f_{tensile} \\ \text{"Use Uncracked section"} & \text{otherwise} \end{cases}$$

$Fatigue_{section} = \text{"Use Cracked section"}$

Minimum stress in reinforcement due to minimum live load.................................

$$f_{min} := \frac{M_{min.neg}}{A_{s.neg} \cdot \left(d_{s.neg} + \frac{x_{na.neg}}{3}\right)}$$

$f_{min} = 7.09\ ksi$

Ratio of r/h is taken as 0.3, therefore the allowable stress range is given by............

$$f_{t.allow} := \left(21 \cdot ksi - 0.33 \cdot f_{min}\right) + 8 \cdot ksi \cdot (r\_h)$$

$f_{t.allow} = 21.06\ ksi$

Actual stress range................................

$$f_t := \left|\frac{M_{range.neg}}{A_{s.neg} \cdot \left(d_{s.neg} + \frac{x_{na.neg}}{3}\right)}\right|$$

$f_t = 8.111\ ksi$

$$LRFD_{5.5.3.2} := \begin{cases} \text{"OK, fatigue stress range requirement for -M region"} & \text{if } f_t \le f_{t.allow} \\ \text{"NG, fatigue stress range requirements for -M region"} & \text{otherwise} \end{cases}$$

$LRFD_{5.5.3.2} = \text{"OK, fatigue stress range requirement for -M region"}$

B8. Summary of Reinforcement Provided

Main reinforcing

Top bar size (-M) $bar_{neg} = \text{"8"}$

Top spacing $spacing_{neg} = 6.0\,in$

Bottom bar size (+M) $bar = \text{"8"}$

Bottom spacing $spacing_{pos} = 6.0\,in$

Shrinkage and temperature reinforcing

Bar size $bar_{st} = \text{"5"}$

Bottom spacing $bar_{spa.st} = 9.0\,in$

$LRFD_{5.7.10.8}$ = "OK, minimum shrinkage and temperature requirements"

Longitudinal Distribution reinforcing

Bar size $bar_{dist} = \text{"5"}$

Bottom spacing $bar_{spa.dist} = 12.0\,in$

$LRFD_{5.14.4}$ = "OK, distribution reinforcement requirements"

Defined Units

Edge Beam Design Loads

References

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\202FlatSlab.mcd(R)

Description

This section provides the design loads for the flat slab edge beam superstructure.

Page Contents

LRFD Criteria

STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.

$WA = 0$ For superstructure design, water load and stream pressure are not applicable.

$FR = 0$ No friction forces.

$$Strength1 = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot LL + 0.50 \cdot (TU + CR + SH)$$

STRENGTH II - Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

"Permit vehicles are not evaluated in this design example"

SERVICE I - Load combination relating to the normal operational use of the bridge with a 55 MPH wind and all loads taken at their nominal values.

$BR, WL = 0$ For superstructure design, braking forces and wind on live load are not applicable.

$CR, SH = 0$ Creep and shrinkage is not evaluated in this design example.

$$Service1 = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot LL$$

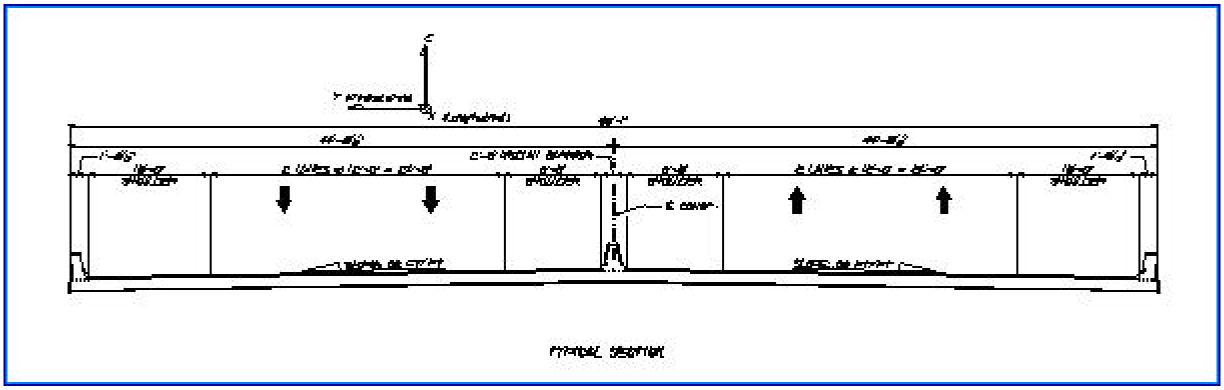
FATIGUE - Fatigue load combination relating to repetitive gravitational vehicular live load under a single design truck.

$$Fatigue = 0.75 \cdot LL$$

Note:

- **LRFD** Commentary **C4.6.2.1.6** states that "past practice has been not to check shear in typical decks... It is not the intent to check shear in every deck." In addition, **LRFD 5.14.4.1** states that for cast-in-place slab superstructures designed for moment in conformance with **LRFD 4.6.2.3**, may be considered satisfactory for shear.
- For this design example, shear will not be investigated. From previous past experience, if the slab thickness is chosen according to satisfy LRFD minimum thickness requirements as per the slab to depth ratios and designed utilizing the distribution strips, shear will not control. If special vehicles are used in the design, shear may need to be investigated.

A. Input Variables



Bridge design span length........................ $L_{span} = 35\,ft$

Thickness of superstructure slab.............. $t_{slab} = 18\,in$

Milling surface thickness......................... $t_{mill} = 0\,in$

Dynamic Load Allowance........................ $IM = 1.33$

Bridge skew... $Skew = -30\,deg$

B. Dead Load Analysis

For the dead load calculation, the influence line coordinates for a uniform load applied on the structure is utilized. The influence coordinates are based on AISC's Moments, Shears and Reactions for Continuous Highway Bridges, published 1966.

Unfactored Dead Loads

	(10th points)	Moments	
Pt.	"X" distance	DC	DW
0	0	0.0	0.0
1	3.5	10.3	0.6
2	7	17.6	1.1
3	10.5	22.0	1.4
4	14	23.5	1.5
5	17.5	22.0	1.4
6	21	17.6	1.1
7	24.5	10.3	0.6
8	28	0.0	0.0
9	31.5	-13.2	-0.8
10	35	-29.4	-1.8
11	38.5	-16.2	-1.0
12	42	-5.9	-0.4
13	45.5	1.5	0.1
14	49	5.9	0.4
15	52.5	7.3	0.5
16	56	5.9	0.4
17	59.5	1.5	0.1
18	63	-5.9	-0.4
19	66.5	-16.2	-1.0
20	70	-29.4	-1.8
21	73.5	-13.2	-0.8
22	77	0.0	0.0
23	80.5	10.3	0.6
24	84	17.6	1.1
25	87.5	22.0	1.4
26	91	23.5	1.5
27	94.5	22.0	1.4
28	98	17.6	1.1
29	101.5	10.3	0.6
30	105	0.0	0.0

*(**Note**: For input values, see Section 2.01 - Design Loads)*

C. Approximate Methods of Analysis - Decks [LRFD 4.6.2]

C1. Equivalent Strip Widths for Slab-type Bridges [LRFD 4.6.2.3]

The superstructure is designed on a per foot basis longitudinally. However, in order to distribute the live loads, equivalent strips of flat slab deck widths are calculated. The moment and shear effects of a single HL-93 vehicle or multiple vehicles are divided by the appropriate equivalent strip width. The equivalent strips account for the transverse distribution of LRFD wheel loads. This section is only applicable for spans greater than **15 feet**.

One design lane

The equivalent width of longitudinal strips per lane for both shear and moment with one lane loaded for the edge beam is given as:

$$E_{EB} = \frac{E_{OneLane}}{2} + b_{barrier} + 12 \cdot in \le E_{OneLane} \le 72 \cdot in$$

where

$E_{OneLane} = 172\,in$ The equivalent distribution width for one lane loaded

$b_{barrier} := 1.5417 \cdot ft$ Edge of deck to inside face of barrier

The equivalent distribution width for the edge beam is given as......................

$$E_{EdgeBm} := \frac{E_{OneLane}}{2} + b_{barrier} + 12 \cdot in$$

$E_{EdgeBm} = 116.5\,in$

Applying the restraint conditions, the equivalent distribution width is given as

$$E_{EB} := \mathrm{minval}\left(E_{EdgeBm}, E_{OneLane}, 72 \cdot in\right)$$

$E_{EB} = 72\,in$ or $E_{EB} = 6\,ft$

Skew modification

For skewed bridges, the longitudinal force effects (moments only) **may** be reduced by a factor r...

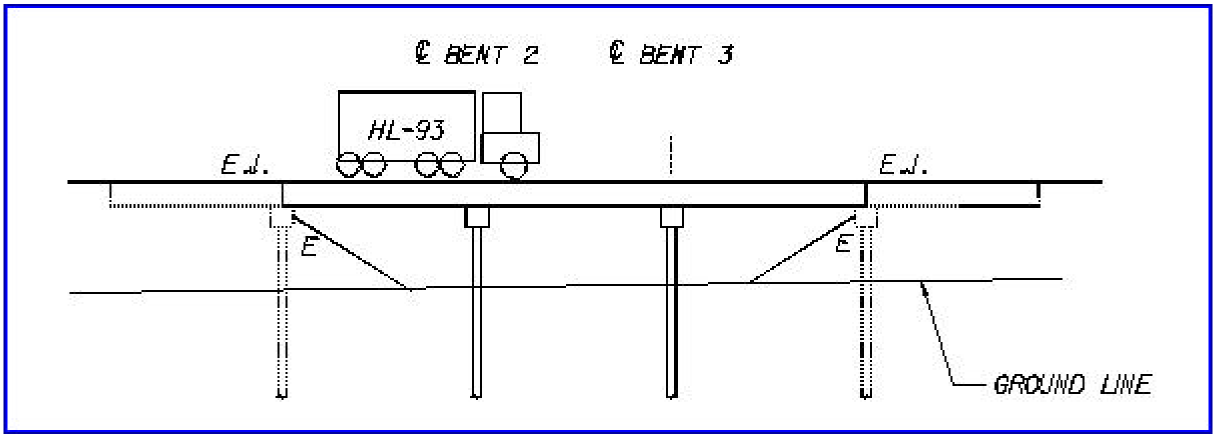
$$r := \min(1.05 - 0.25 \cdot \tan(|Skew|), 1.00)$$

$r = 0.91$

*(**Note**: For this design example, the skew modification will not be applied in order to design for more conservative moment values)*

C2. Live Load Analysis

Determine the live load moments and shears due to one HL-93 vehicle on the continuous flat slab structure. The design live loads will consists of the HL-93 vehicle moments, divided by the appropriate equivalent strip widths. This will result in a design live load per foot width of flat slab.



In order to calculate the live load moments and shears, the FDOT MathCad program "LRFD Live Load Generator, English, v2.1".

Read Live Load results from files generated by FDOT Program

*(**Note**: For input values, see Section 2.01 - Design Loads)*

HL-93 Live Load Envelopes

Pt.	(10th points) "X" distance	Service I +M	Service I -M	Strength I +M	Strength I -M	Fatigue +M	Fatigue -M	Fatigue Range	Unfactored Lane Load +M	Unfactored Lane Load -M
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5	31.4	-3.9
2	7	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5	54.7	-7.8
3	10.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8	70.7	-11.8
4	14	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3	78.2	-15.7
5	17.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3	78.2	-19.6
6	21	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8	70.7	-23.5
7	24.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5	54.7	-27.4
8	28	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0	31.5	-31.5
9	31.5	88.1	-232.9	154.2	-407.6	39.8	-117.9	157.7	15.9	-51.1
10	35	76.1	-383.5	133.2	-671.1	27.0	-186.9	213.8	13.1	-92.0
11	38.5	89.5	-275.7	156.7	-482.5	48.7	-122.2	170.8	11.8	-55.1
12	42	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8	23.5	-39.2
13	45.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8	43.1	-39.2
14	49	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5	54.7	-39.2
15	52.5	403.4	-133.9	706.0	-234.3	134.4	-40.5	174.9	58.7	-39.2
16	56	386.1	-165.5	675.7	-289.6	136.6	-54.0	190.5	54.7	-39.2
17	59.5	322.4	-196.6	564.2	-344.1	124.3	-67.5	191.8	43.1	-39.2
18	63	215.3	-228.7	376.8	-400.2	95.6	-81.2	176.8	23.5	-39.2
19	66.5	90.1	-275.7	157.6	-482.5	48.7	-122.2	170.8	11.8	-55.1
20	70	76.1	-383.0	133.2	-670.3	27.0	-186.9	213.8	13.1	-91.6
21	73.5	87.5	-232.9	153.1	-407.6	39.8	-117.9	157.7	15.9	-51.1
22	77	213.3	-184.5	373.3	-322.9	94.9	-59.1	154.0	31.5	-31.5
23	80.5	340.6	-161.5	596.1	-282.6	138.0	-40.5	178.5	54.7	-27.4
24	84	433.1	-137.7	757.9	-241.0	171.1	-34.7	205.8	70.7	-23.5
25	87.5	482.8	-115.0	844.9	-201.3	198.4	-28.9	227.3	78.2	-19.6
26	91	495.0	-92.1	866.3	-161.1	209.2	-23.1	232.3	78.2	-15.7
27	94.5	460.8	-69.0	806.4	-120.7	195.5	-17.3	212.8	70.7	-11.8
28	98	369.4	-46.0	646.5	-80.4	156.0	-11.6	167.5	54.7	-7.8
29	101.5	220.9	-23.0	386.6	-40.2	92.7	-5.8	98.5	31.4	-3.9
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

As per **LRFD 4.6.2.1.4a**, the edge beams shall be assumed to support one line of wheels and a tributary portion of the design lane load.

The HL-93 live load moment envelopes shown in the above summary include lane loads (except for Fatigue). The lane load and truck moments need to be separated and manipulated separately. Since the unfactored lane load envelopes are given, the separated values for truck and lane can be can be calculated and multiplied by the appropriate factors.

Edge beams shall be assumed to support one line of wheels, therefore multiply the truck moments by

$$Factor_{truck} := 0.5$$

Tributary portion of the design lane load is given by, $Factor_{lane}$, since the maximum width of the edge beam is limited by the LRFD to 72 inches.

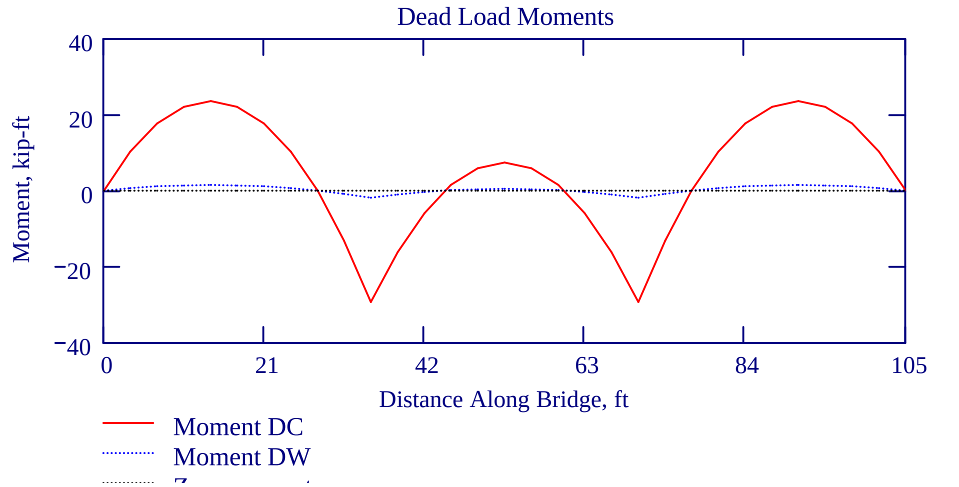
$$Factor_{lane} := \frac{E_{EB} - b_{barrier}}{10 \cdot ft} \qquad Factor_{lane} = 0.446$$

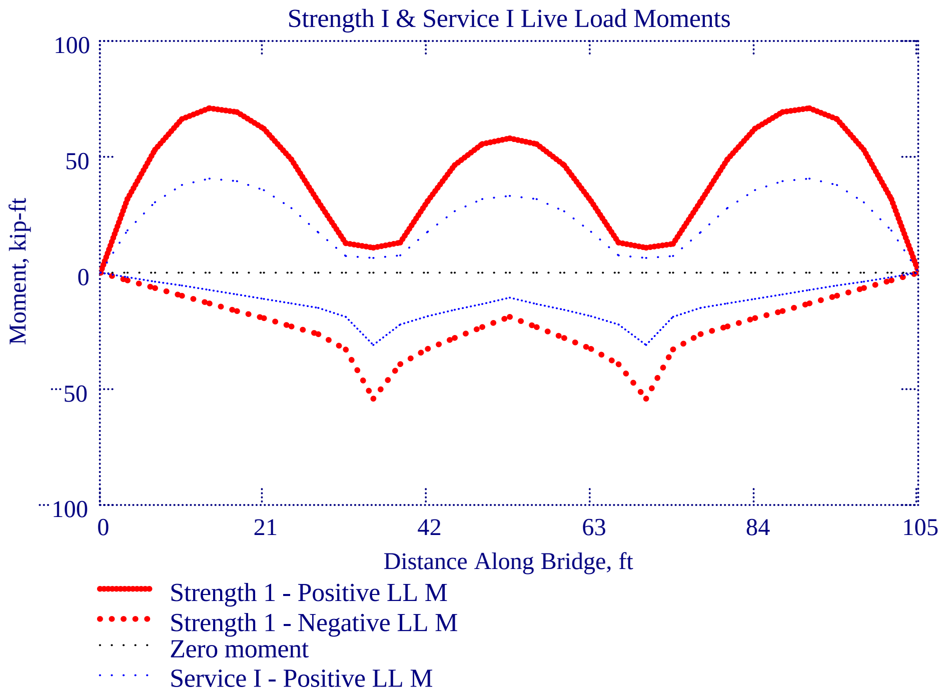
HL-93 Live Load Envelopes

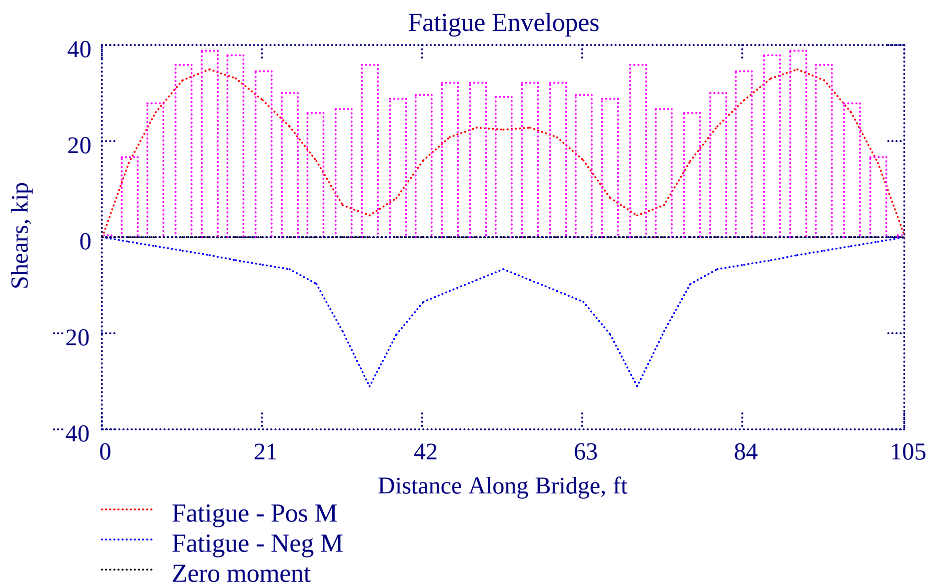
		Service I				Strength I						
	(10th points)	Truck		Lane		Truck		Lane		Fatigue		
Pts.	distance	+M	-M	+M	-M	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	94.8	-9.5	14.0	-1.7	165.8	-16.7	24.5	-3.1	92.7	-5.8	98.5
2	7	157.4	-19.1	24.4	-3.5	275.4	-33.4	42.7	-6.1	156.0	-11.6	167.5
3	10.5	195.1	-28.6	31.5	-5.3	341.4	-50.0	55.1	-9.2	195.5	-17.3	212.8
4	14	208.4	-38.2	34.9	-7.0	364.7	-66.8	61.0	-12.2	209.2	-23.1	232.3
5	17.5	202.3	-47.7	34.9	-8.7	354.0	-83.5	61.0	-15.3	198.4	-28.9	227.3
6	21	181.2	-57.1	31.5	-10.5	317.1	-99.9	55.1	-18.3	171.1	-34.7	205.8
7	24.5	143.0	-67.0	24.4	-12.2	250.2	-117.3	42.7	-21.4	138.0	-40.5	178.5
8	28	90.9	-76.5	14.0	-14.0	159.1	-133.9	24.6	-24.6	94.9	-59.1	154.0
9	31.5	36.1	-90.9	7.1	-22.8	63.2	-159.1	12.4	-39.9	39.8	-117.9	157.7
10	35	31.5	-145.8	5.8	-41.0	55.2	-255.1	10.2	-71.8	27.0	-186.9	213.8
11	38.5	38.9	-110.3	5.3	-24.6	68.0	-193.0	9.2	-43.0	48.7	-122.2	170.8
12	42	95.9	-94.8	10.5	-17.5	167.8	-165.8	18.3	-30.6	95.6	-81.2	176.8
13	45.5	139.6	-78.7	19.2	-17.5	244.4	-137.7	33.6	-30.6	124.3	-67.5	191.8
14	49	165.7	-63.2	24.4	-17.5	290.0	-110.5	42.7	-30.6	136.6	-54.0	190.5
15	52.5	172.4	-47.4	26.2	-17.5	301.6	-82.9	45.8	-30.6	134.4	-40.5	174.9
16	56	165.7	-63.2	24.4	-17.5	290.0	-110.5	42.7	-30.6	136.6	-54.0	190.5
17	59.5	139.6	-78.7	19.2	-17.5	244.4	-137.7	33.6	-30.6	124.3	-67.5	191.8
18	63	95.9	-94.8	10.5	-17.5	167.8	-165.8	18.3	-30.6	95.6	-81.2	176.8
19	66.5	39.1	-110.3	5.3	-24.6	68.5	-193.0	9.2	-43.0	48.7	-122.2	170.8
20	70	31.5	-145.7	5.8	-40.8	55.2	-255.0	10.2	-71.4	27.0	-186.9	213.8
21	73.5	35.8	-90.9	7.1	-22.8	62.6	-159.1	12.4	-39.9	39.8	-117.9	157.7
22	77	90.9	-76.5	14.0	-14.0	159.1	-133.9	24.6	-24.6	94.9	-59.1	154.0
23	80.5	143.0	-67.0	24.4	-12.2	250.2	-117.3	42.7	-21.4	138.0	-40.5	178.5
24	84	181.2	-57.1	31.5	-10.5	317.1	-99.9	55.1	-18.3	171.1	-34.7	205.8
25	87.5	202.3	-47.7	34.9	-8.7	354.0	-83.5	61.0	-15.3	198.4	-28.9	227.3
26	91	208.4	-38.2	34.9	-7.0	364.7	-66.8	61.0	-12.2	209.2	-23.1	232.3
27	94.5	195.1	-28.6	31.5	-5.3	341.4	-50.0	55.1	-9.2	195.5	-17.3	212.8
28	98	157.4	-19.1	24.4	-3.5	275.4	-33.4	42.7	-6.1	156.0	-11.6	167.5
29	101.5	94.8	-9.5	14.0	-1.7	165.8	-16.7	24.5	-3.1	92.7	-5.8	98.5
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Combine the truck and lane loads per each limit state and divide the moments by the distribution width, $E_{EB} = 6\ ft$ to obtain the design values for live load.

Design Live Load Envelopes		E =	6.0 ft					
	(10th points)	Service I		Strength I		Fatigue		
Joint	"X" distance	+M	-M	+M	-M	+M	-M	M_{Range}
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	18.1	-1.9	30.0	-3.1	15.5	-1.0	16.4
2	7	30.3	-3.8	50.0	-6.1	26.0	-1.9	27.9
3	10.5	37.8	-5.6	62.1	-9.2	32.6	-2.9	35.5
4	14	40.5	-7.5	66.6	-12.3	34.9	-3.9	38.7
5	17.5	39.5	-9.4	64.8	-15.4	33.1	-4.8	37.9
6	21	35.5	-11.3	58.1	-18.4	28.5	-5.8	34.3
7	24.5	27.9	-13.2	45.8	-21.6	23.0	-6.7	29.8
8	28	17.5	-15.1	28.9	-24.7	15.8	-9.8	25.7
9	31.5	7.2	-18.9	11.7	-30.3	6.6	-19.7	26.3
10	35	6.2	-31.1	10.2	-49.3	4.5	-31.2	35.6
11	38.5	7.4	-22.5	12.2	-36.3	8.1	-20.4	28.5
12	42	17.7	-18.7	29.7	-30.5	15.9	-13.5	29.5
13	45.5	26.5	-16.0	43.9	-25.9	20.7	-11.2	32.0
14	49	31.7	-13.4	52.4	-21.3	22.8	-9.0	31.8
15	52.5	33.1	-10.8	54.6	-16.7	22.4	-6.7	29.2
16	56	31.7	-13.4	52.4	-21.3	22.8	-9.0	31.8
17	59.5	26.5	-16.0	43.9	-25.9	20.7	-11.2	32.0
18	63	17.7	-18.7	29.7	-30.5	15.9	-13.5	29.5
19	66.5	7.4	-22.5	12.3	-36.3	8.1	-20.4	28.5
20	70	6.2	-31.1	10.2	-49.3	4.5	-31.2	35.6
21	73.5	7.1	-18.9	11.6	-30.3	6.6	-19.7	26.3
22	77	17.5	-15.1	28.9	-24.7	15.8	-9.8	25.7
23	80.5	27.9	-13.2	45.8	-21.6	23.0	-6.7	29.8
24	84	35.5	-11.3	58.1	-18.4	28.5	-5.8	34.3
25	87.5	39.5	-9.4	64.8	-15.4	33.1	-4.8	37.9
26	91	40.5	-7.5	66.6	-12.3	34.9	-3.9	38.7
27	94.5	37.8	-5.6	62.1	-9.2	32.6	-2.9	35.5
28	98	30.3	-3.8	50.0	-6.1	26.0	-1.9	27.9
29	101.5	18.1	-1.9	30.0	-3.1	15.5	-1.0	16.4
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0


Dead Load Moments
Moment, kip-ft
40
20
0
−20
−40
0
21
42
63
84
105
Distance Along Bridge, ft
Moment DC
Moment DW


Strength I & Service I Live Load Moments
Moment, kip-ft
100
50
0
−50
−100
0
21
42
63
84
105
Distance Along Bridge, ft
Strength 1 - Positive LL M
Strength 1 - Negative LL M
Zero moment
Service I - Positive LL M


Fatigue Envelopes
Shears, kip
40
20
0
−20
−40
0
21
42
63
84
105
Distance Along Bridge, ft
Fatigue - Pos M
Fatigue - Neg M
Zero moment

Range

C3. Limit State Moments and Shears

The service and strength limit states used to design the section are calculated as follows:

Limit State Design Loads

	(10th points)	Service I 1.0DC + 1.0DW + 1.0LL		Strength I 1.25DC + 1.50DW + 1.75LL		Fatigue 1.0DC + 1.0DW + 1.5LL MRange = 0.75LL ; $-M_{min}$ = 0.75LL			
Pt.	**"X" dist**	**+M**	**-M**	**+M**	**-M**	**+M**	**-M**	**M_{Range}**	**$-M_{min}$**
0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	3.5	29.0	9.0	45.5	10.5	34.1	9.5	12.3	-0.7
2	7	49.0	15.0	76.7	17.1	57.7	15.8	20.9	-1.4
3	10.5	61.2	17.8	95.7	19.7	72.3	19.1	26.6	-2.2
4	14	65.5	17.4	102.5	18.4	77.3	19.2	29.0	-2.9
5	17.5	62.9	14.0	98.8	13.1	73.0	16.2	28.4	-3.6
6	21	54.2	7.5	85.7	4.0	61.5	10.0	25.7	-4.3
7	24.5	38.8	-2.3	62.6	-9.3	45.4	0.8	22.3	-5.1
8	28	17.5	-15.1	30.6	-26.4	23.7	-14.8	19.3	-7.4
9	31.5	-6.8	-33.0	-5.2	-50.9	-4.1	-43.5	19.7	-14.7
10	35	-25.0	-62.3	-28.6	-94.0	-24.5	-77.9	26.7	-23.4
11	38.5	-9.8	-39.6	-8.8	-61.0	-5.0	-47.7	21.4	-15.3
12	42	11.5	-24.9	23.1	-40.6	17.7	-26.5	22.1	-10.2
13	45.5	28.0	-14.5	48.3	-26.1	32.6	-15.3	24.0	-8.4
14	49	37.9	-7.2	63.3	-15.6	40.4	-7.3	23.8	-6.7
15	52.5	40.9	-3.0	67.8	-9.0	41.4	-2.3	21.9	-5.1
16	56	37.9	-7.2	63.3	-15.6	40.4	-7.3	23.8	-6.7
17	59.5	28.0	-14.5	48.3	-26.1	32.6	-15.3	24.0	-8.4
18	63	11.5	-24.9	23.1	-40.6	17.7	-26.5	22.1	-10.2
19	66.5	-9.8	-39.6	-8.8	-61.0	-5.0	-47.7	21.4	-15.3
20	70	-25.0	-62.3	-28.6	-93.9	-24.5	-77.9	26.7	-23.4
21	73.5	-6.9	-33.0	-5.3	-50.9	-4.1	-43.5	19.7	-14.7
22	77	17.5	-15.1	30.6	-26.4	23.7	-14.8	19.3	-7.4
23	80.5	38.8	-2.3	62.6	-9.3	45.4	0.8	22.3	-5.1
24	84	54.2	7.5	85.7	4.0	61.5	10.0	25.7	-4.3
25	87.5	62.9	14.0	98.8	13.1	73.0	16.2	28.4	-3.6
26	91	65.5	17.4	102.5	18.4	77.3	19.2	29.0	-2.9
27	94.5	61.2	17.8	95.7	19.7	72.3	19.1	26.6	-2.2
28	98	49.0	15.0	76.7	17.1	57.7	15.8	20.9	-1.4
29	101.5	29.0	9.0	45.5	10.5	34.1	9.5	12.3	-0.7
30	105	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

<–Maximum positive moment and corresponding fatigue values (Pt. 4)

<–Maximum negative moment and corresponding fatigue values (Pt. 10)

Maximum negative Moments =		-62.3		-94.0	*77.3*		*29.0*	*-2.9*
Maximum positive Moments =	65.5		102.5			*-77.9*	*26.7*	*-23.4*

Defined Units

Flat Slab Edge Beam Design

References

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\203EdgeBmDesignLds.mcd(R)

Description

This section provides the design for the flat slab superstructure.

Page Contents

A. Input Variables

Maximum positive moment and corresponding fatigue values

Service	Strength	Fatigue
$M_{pos} = 65.5\,ft\cdot kip$	$M_{r.pos} = 102.5\,ft\cdot kip$	$M_{fatigue.pos} = 77.3\,ft\cdot kip$
		$M_{range.pos} = 29\,ft\cdot kip$
		$M_{min.pos} = -2.9\,ft\cdot kip$

Maximum negative moment and corresponding fatigue values

Service	Strength	Fatigue
$M_{neg} = -62.3\,ft\cdot kip$	$M_{r.neg} = -94.0\,ft\cdot kip$	$M_{fatigue.neg} = -77.9\,ft\cdot kip$
		$M_{range.neg} = 26.7\,ft\cdot kip$
		$M_{min.neg} = -23.4\,ft\cdot kip$

B. Moment Design

The design procedure consists of calculating the reinforcement required to satisfy the design moment, then checking this reinforcement against criteria for crack control, minimum reinforcement, maximum reinforcement, shrinkage and temperature reinforcement, and distribution of reinforcement. The procedure is the same for both positive and negative moment regions.

B1. Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Factored resistance $M_r = \phi \cdot M_n$

Nominal flexural resistance

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) - A'_s \cdot f'_y \cdot \left(d'_s - \frac{a}{2}\right) + 0.85 \cdot f'_c \cdot (b - b_w) \cdot \beta_1 \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Simplifying the nominal flexural resistance

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) \qquad \text{where} \qquad a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Using variables defined in this example..... $M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

where

$M_{r.pos} = 102.5\,ft \cdot kip$

$f_{c.slab} = 4.5\,ksi$

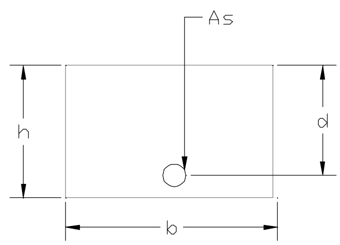
$f_y = 60\,ksi$

$\phi = 0.9$

$t_{slab} = 18\,in$

$b := 1 \cdot ft$

$t_{slab} = h$



$b = 12\,in$

Initial assumption for area of steel required

Size of bar...................................... $bar := "8"$

Proposed bar spacing...................... $spacing_{pos} := 6 \cdot in$

Bar area.. $A_{bar} = 0.790\,in^2$

Bar diameter.................................... $dia = 1.000\,in$

Area of steel provided per foot of slab... $A_{s.pos} := \dfrac{A_{bar} \cdot 1ft}{spacing_{pos}}$

$A_{s.pos} = 1.58\,in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel................ $d_{s.pos} := t_{slab} - cover_{slab} - \dfrac{dia}{2}$

$d_{s.pos} = 15.5\,in$

Solve the quadratic equation for the area of steel required

Given $M_{r.pos} = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_{s.pos} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$

Reinforcing steel required....................... $A_{s.reqd} := Find(A_{s.pos})$

$A_{s.reqd} = 1.57\,in^2$

The area of steel provided, $A_{s.pos} = 1.58\,in^2$, should be greater than the area of steel required, $A_{s.reqd} = 1.57\,in^2$. If not, decrease the spacing of the reinforcement. Once $A_{s.pos}$ is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the design moments.

Moment capacity provided..................... $M_{r.positive.prov} := \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_{s.pos} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b} \right) \right]$

$M_{r.positive.prov} = 104.2\,ft \cdot kip$

B2. Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Variables: $|M_{r.neg}| = 93.95\,ft \cdot kip$

$f_{c.slab} = 4.5\,ksi$

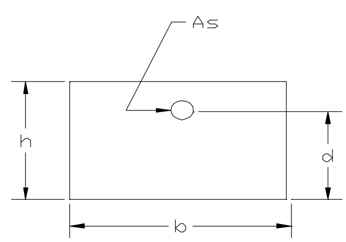
$f_y = 60\,ksi$

$\phi = 0.9$

$t_{slab} = 18\,in$

$b = 1\,ft$

$t_{slab} = h$



$b = 12\,in$

Initial assumption for area of steel required

Size of bar...................................... $bar_{neg} := "8"$

Proposed bar spacing....................... $spacing_{neg} := 6 \cdot in$

Bar area.. $A_{bar.neg} = 0.790\,in^2$

Bar diameter................................... $dia_{neg} = 1.000\,in$

Area of steel provided per foot of slab.. $A_{s.neg} := \dfrac{A_{bar.neg} \cdot 1ft}{spacing_{neg}}$

$A_{s.neg} = 1.58\,in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel................ $d_{s.neg} := t_{slab} - cover_{slab} - \dfrac{dia_{neg}}{2}$

$d_{s.neg} = 15.5\,in$

Solve the quadratic equation for the area of steel required

Given $\left|M_{r.neg}\right| = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_{s.neg} - \frac{1}{2} \cdot \left(\dfrac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

Reinforcing steel required....................... $A_{s.reqd} := \text{Find}\left(A_{s.neg}\right)$

$A_{s.reqd} = 1.43\,in^2$

The area of steel provided, $A_{s.neg} = 1.58\,in^2$, should be greater than the area of steel required, $A_{s.reqd} = 1.43\,in^2$. If not, decrease the spacing of the reinforcement. Once $A_{s.neg}$ is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the design moments.

Moment capacity provided..................... $M_{r.negative.prov} := \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_{s.neg} - \frac{1}{2} \cdot \left(\dfrac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b}\right)\right]$

$M_{r.negative.prov} = 104.2\,ft \cdot kip$

B3. Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

Stress in the mild steel reinforcement at the service limit state.................................. $f_{sa} = \dfrac{z}{\left(d_c \cdot A\right)^{\frac{1}{3}}} \le 0.6 \cdot f_y$

Crack width parameter........................... $z = \begin{pmatrix} \text{"moderate exposure"} & 170 \\ \text{"severe exposure"} & 130 \\ \text{"buried structures"} & 100 \end{pmatrix} \cdot \dfrac{kip}{in}$

The environmental classifications for Florida designs do not match the classifications to select the crack width parameter. For this example, a "Slightly" or "Moderately" aggressive environment corresponds to "moderate exposure" and an "Extremely" aggressive environment corresponds to "severe exposure".

$Environment_{super} = \text{"Slightly"}$ aggressive environment

$z := 170 \cdot \frac{kip}{in}$

Positive Moment

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)........................

$d_c := \min\left(t_{slab} - d_{s.pos}, 2 \cdot in + \frac{dia}{2}\right)$

$d_c = 2.5\,in$

Number of bars per design width of slab.........................

$n_{bar} := \frac{b}{spacing_{pos}}$

$n_{bar} = 2$

Effective tension area of concrete surrounding the flexural tension reinforcement...

$A := \frac{(b) \cdot (2 \cdot d_c)}{n_{bar}}$

$A = 30.0\,in^2$

Service limit state stress in reinforcement........................

$f_{sa} := \min\left[\frac{z}{(d_c \cdot A)^{\frac{1}{3}}}, 0.6 \cdot f_y\right]$

$f_{sa} = 36.0\,ksi$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 4.8 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.slab}} \cdot A_{s.pos} \cdot (d_{s.pos} - x)$

$x_{na} := Find(x)$

$x_{na} = 4.8\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 4.8\,in$ to equal $x_{na} = 4.8\,in$.

Tensile force in the reinforcing steel due to service limit state moment...

$T_s := \frac{M_{pos}}{d_{s.pos} - \frac{x_{na}}{3}}$

$T_s = 56.614\,kip$

Actual stress in the reinforcing steel due to service limit state moment..

$$f_{s.actual} := \frac{T_s}{A_{s.pos}}$$

$f_{s.actual} = 35.8\,ksi$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$$LRFD_{5.7.3.3.4a} := \begin{cases} \text{"OK, crack control for +M is satisfied"} & \text{if } f_{s.actual} \le f_{sa} \\ \text{"NG, crack control for +M not satisfied, provide more reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.4a} =$ "OK, crack control for +M is satisfied"

Negative Moment

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.).......................

$$d_c := \min\left(t_{slab} - d_{s.neg}, 2 \cdot in + \frac{dia_{neg}}{2}\right)$$

$d_c = 2.5\,in$

Number of bars per design width of slab.........................

$$n_{bar} := \frac{b}{spacing_{neg}}$$

$n_{bar} = 2$

Effective tension area of concrete surrounding the flexural tension reinforcement...

$$A := \frac{(b) \cdot (2 \cdot d_c)}{n_{bar}}$$

$A = 30.0\,in^2$

Service limit state stress in reinforcement.........................

$$f_{sa} := \min\left[\frac{z}{(d_c \cdot A)^{\frac{1}{3}}}, 0.6 \cdot f_y\right]$$

$f_{sa} = 36.0\,ksi$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 4.8 \cdot in$

Given $\quad \frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.slab}} \cdot A_{s.neg} \cdot (d_{s.neg} - x)$

$x_{na} := Find(x)$

$x_{na} = 4.8\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 4.8\,in$ to equal $x_{na} = 4.8\,in$.

Tensile force in the reinforcing steel due to service limit state moment..

$$T_s := \frac{|M_{neg}|}{d_{s.neg} - \frac{x_{na}}{3}}$$

$T_s = 53.872\ \text{kip}$

Actual stress in the reinforcing steel due to service limit state moment..

$$f_{s.actual} := \frac{T_s}{A_{s.neg}}$$

$f_{s.actual} = 34.1\ \text{ksi}$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$$LRFD_{5.7.3.3.4b} := \begin{cases} \text{"OK, crack control for -M is satisfied"} & \text{if } f_{s.actual} \le f_{sa} \\ \text{"NG, crack control for -M not satisfied, provide more reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.4b} =$ "OK, crack control for -M is satisfied"

B4. Limits for Reinforcement [LRFD 5.7.3.3]

Maximum Reinforcement

The maximum reinforcement requirements ensure the section has sufficient ductility and is not overreinforced. The greater reinforcement from the positive and negative moment sections is checked.

Area of steel provided............................

$A_{s.pos} = 1.58\ \text{in}^2$

$A_{s.neg} = 1.58\ \text{in}^2$

Stress block factor...............................

$$\beta_1 := \max\left[0.85 - 0.05 \cdot \left(\frac{f_{c.slab} - 4000 \cdot \text{psi}}{1000 \cdot \text{psi}}\right), 0.65\right]$$

$\beta_1 = 0.825$

Distance from extreme compression fiber to the neutral axis of section...................

$$c_{pos} := \frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot \beta_1 \cdot b} \quad \text{and} \quad c_{neg} := \frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot \beta_1 \cdot b}$$

$c_{pos} = 2.5\ \text{in}$

$c_{neg} = 2.5\ \text{in}$

Effective depth from extreme compression fiber to centroid of the tensile reinforcement

$$d_e = \frac{A_s \cdot f_{ps} \cdot d_p + A_s \cdot f_y \cdot d_s}{A_{ps} \cdot f_{ps} + A_s \cdot f_y}$$

Simplifying for this example.............

$$d_{e.pos} := d_{s.pos} \quad \text{and} \quad d_{e.neg} := d_{s.neg}$$

$d_{e.pos} = 15.5\ \text{in}$

$d_{e.neg} = 15.5\ \text{in}$

The $\frac{c_{pos}}{d_{e.pos}} = 0.162$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$$LRFD_{5.7.3.3.1} := \begin{cases} \text{"OK, maximum reinforcement in +M region"} & \text{if } \frac{c_{pos}}{d_{e.pos}} \le 0.42 \\ \text{"NG, section is over-reinforced in +M region, see LRFD eq. C5.7.3.3.1-1"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement in +M region"

The $\frac{c_{neg}}{d_{e.neg}} = 0.162$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$$LRFD_{5.7.3.3.1} := \begin{cases} \text{"OK, maximum reinforcement in -M region"} & \text{if } \frac{c_{neg}}{d_{e.neg}} \le 0.42 \\ \text{"NG, section is over-reinforced in -M region, see LRFD eq. C5.7.3.3.1-1"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement in -M region"

Minimum Reinforcement

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture.............................. $f_r := 0.24 \cdot \sqrt{f_{c.slab} \cdot ksi}$

$f_r = 509.1\ psi$

Section modulus.................................. $S := \frac{b \cdot t_{slab}^2}{6}$

$S = 648.0\ in^3$

Cracking moment................................. $M_{cr} := f_r \cdot S$

$M_{cr} = 27.5\ kip \cdot ft$

Required flexural resistance (+M)........... $M_{r.reqd} := \min(1.2 \cdot M_{cr}, 133 \cdot \% \cdot M_{r.pos})$

$M_{r.reqd} = 33.0\ ft \cdot kip$

Check that the capacity provided, $M_{r.positive.prov} = 104.2\ ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 33\ ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{cases} \text{"OK, minimum reinforcement for positive moment is satisfied"} & \text{if } M_{r.positive.prov} \ge M_{r.reqd} \\ \text{"NG, reinforcement for positive moment is less than minimum"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for positive moment is satisfied"

Required flexural resistance (-M)............. $M_{r.reqd} := \min\left(1.2 \cdot M_{cr}, 133 \cdot \% \cdot \left|M_{r.neg}\right|\right)$

$M_{r.reqd} = 33.0\,ft \cdot kip$

Check that the capacity provided, $M_{r.negative.prov} = 104.2\,ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 33\,ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{vmatrix} \text{"OK, minimum reinforcement for negative moment is satisfied"} & \text{if } M_{r.positive.prov} \geq M_{r.reqd} \\ \text{"NG, reinforcement for negative moment is less than minimum"} & \text{otherwise} \end{vmatrix}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for negative moment is satisfied"

B5. Shrinkage and Temperature Reinforcement [LRFD 5.10.8.2]

Shrinkage and temperature reinforcement provided

Size of bar ("4" "5" "6")................... $bar_{st} :=$ "5"

Bar spacing.................................... $bar_{spa.st} := 9 \cdot in$

Bar area.. $A_{bar} = 0.31\,in^2$

Bar diameter.................................... $dia = 0.625\,in$

Gross area of section............................ $A_g := b \cdot t_{slab}$

$A_g = 216.0\,in^2$

Minimum area of shrinkage and temperature reinforcement....................................... $A_{ST} := \dfrac{0.11 \cdot ksi \cdot A_g}{f_y}$

$A_{ST} = 0.40\,in^2$

Maximum spacing for shrinkage and temperature reinforcement...................... $spacing_{ST} := \min\left(\dfrac{b}{\dfrac{A_{ST}}{A_{bar}}}, 3 \cdot t_{slab}, 18 \cdot in\right)$

$spacing_{ST} = 9.4\,in$

The bar spacing should be less than the maximum spacing for shrinkage and temperature reinforcement

$$LRFD_{5.7.10.8} := \begin{vmatrix} \text{"OK, minimum shrinkage and temperature requirements"} & \text{if } bar_{spa.st} \leq spacing_{ST} \\ \text{"NG, minimum shrinkage and temperature requirements"} & \text{otherwise} \end{vmatrix}$$

$LRFD_{5.7.10.8} =$ "OK, minimum shrinkage and temperature requirements"

B6. Distribution of Reinforcement [LRFD 5.14.4]

Transverse distribution reinforcement shall be placed in the bottom of the slab. The amount to place is based on a percentage of the longitudinal main steel.

Distribution reinforcement provided

Size of bar ("4" "5" "6")................... $bar_{dist} := "5"$

Bar spacing.................................... $bar_{spa.dist} := 12 \cdot in$

Bar area.. $A_{bar} = 0.31\,in^2$

Bar diameter.................................... $dia = 0.625\,in$

The area for secondary reinforcement should not exceed 50% of the area for primary reinforcement......................................

$$\%A_{steel} := \min\left(\frac{100}{\sqrt{\frac{L_{span}}{ft}}}\%, 50\%\right)$$

$\%A_{steel} = 0.17$

Required area for secondary reinforcement. $A_{s.DistR} := A_{s.pos} \cdot \%A_{steel}$

$A_{s.DistR} = 0.27\,in^2$

Maximum spacing for secondary reinforcement.......................................

$$MaxSpacing_{DistR} := \frac{b}{\left(\frac{A_{s.DistR}}{A_{bar}}\right)}$$

$MaxSpacing_{DistR} = 13.9\,in$

The bar spacing should not exceed the maximum spacing for secondary reinforcement

$$LRFD_{5.14.4} := \begin{cases} \text{"OK, distribution reinforcement requirements"} & \text{if } bar_{spa.dist} \le MaxSpacing_{DistR} \\ \text{"NG, distribution reinforcement requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{5.14.4} =$ "OK, distribution reinforcement requirements"

B7. Fatigue Limit State

The section properties for fatigue shall be based on cracked sections where the sum of stresses due to unfactored permanent loads and 1.5 times the fatigue load is tensile and exceeds $0.095\sqrt{f_c}$.

Allowable tensile stress for fatigue............ $f_{tensile} := 0.095\sqrt{f_{c.slab} \cdot ksi}$

$f_{tensile} = 0.202\,ksi$

Positive Moment Region

Stress due to positive moment................ $f_{fatigue.pos} := \dfrac{M_{fatigue.pos}}{S}$

$f_{fatigue.pos} = 1.431\,ksi$

$$Fatigue_{section} := \begin{cases} \text{"Use Cracked section"} & \text{if } f_{fatigue.pos} > f_{tensile} \\ \text{"Use Uncracked section"} & \text{otherwise} \end{cases}$$

$Fatigue_{section} =$ "Use Cracked section"

Minimum stress in reinforcement due to minimum live load................................ $f_{min} := \dfrac{M_{min.pos}}{A_{s.pos} \cdot \left(d_{s.pos} - \dfrac{c_{pos}}{3}\right)}$

$f_{min} = -1.498\,ksi$

Ratio of r/h is taken as $r\_h := 0.3$, therefore the allowable stress range is given by........ $f_{t.allow} := (21 \cdot ksi - 0.33 \cdot f_{min}) + 8 \cdot ksi \cdot (r\_h)$

$f_{t.allow} = 23.894\,ksi$

Actual stress range................................ $f_t := \dfrac{M_{range.pos}}{A_{s.pos} \cdot \left(d_{s.pos} - \dfrac{c_{pos}}{3}\right)}$

$f_t = 15.038\,ksi$

$$LRFD_{5.5.3.2} := \begin{cases} \text{"OK, fatigue stress range requirement for +M region"} & \text{if } f_t \le f_{t.allow} \\ \text{"NG, fatigue stress range requirements for +M region"} & \text{otherwise} \end{cases}$$

$LRFD_{5.5.3.2} =$ "OK, fatigue stress range requirement for +M region"

Negative Moment Region

Stress due to positive moment.................. $f_{fatigue.neg} := \frac{|M_{fatigue.neg}|}{S}$

$f_{fatigue.neg} = 1.443\,ksi$

$$Fatigue_{section} := \begin{cases} \text{"Use Cracked section"} & \text{if } f_{fatigue.neg} > f_{tensile} \\ \text{"Use Uncracked section"} & \text{otherwise} \end{cases}$$

$Fatigue_{section} =$ "Use Cracked section"

Minimum stress in reinforcement due to minimum live load................................ $f_{min} := \frac{M_{min.neg}}{A_{s.neg} \cdot \left(d_{s.neg} - \frac{c_{neg}}{3}\right)}$

$f_{min} = -12.099\,ksi$

Ratio of r/h is taken as 0.3, therefore the allowable stress range is given by............ $f_{t.allow} := (21 \cdot ksi - 0.33 \cdot f_{min}) + 8 \cdot ksi \cdot (r\_h)$

$f_{t.allow} = 27.393\,ksi$

Actual stress range................................ $f_t := \frac{M_{range.neg}}{A_{s.neg} \cdot \left(d_{s.neg} - \frac{c_{neg}}{3}\right)}$

$f_t = 13.84\,ksi$

$$LRFD_{5.5.3.2} := \begin{cases} \text{"OK, fatigue stress range requirement for -M region"} & \text{if } f_t \le f_{t.allow} \\ \text{"NG, fatigue stress range requirements for -M region"} & \text{otherwise} \end{cases}$$

$LRFD_{5.5.3.2} =$ "OK, fatigue stress range requirement for -M region"

B8. Summary of Reinforcement Provided

Main reinforcing

Top bar size (-M) $bar_{neg} = "8"$

Top spacing $spacing_{neg} = 6.0\ in$

Bottom bar size (+M) $bar = "8"$

Bottom spacing $spacing_{pos} = 6.0\ in$

Shrinkage and temperature reinforcing

Bar size $bar_{st} = "5"$

Bottom spacing $bar_{spa.st} = 9.0\ in$

$LRFD_{5.7.10.8}$ = "OK, minimum shrinkage and temperature requirements"

Longitudinal Distribution reinforcing

Bar size $bar_{dist} = "5"$

Bottom spacing $bar_{spa.dist} = 12.0\ in$

$LRFD_{5.14.4}$ = "OK, distribution reinforcement requirements"

Defined Units

Expansion Joint Design

References

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\204EdgeBeam.mcd(R)

Description

This section provides the design of the bridge expansion joints.

Page Contents

LRFD Criteria

Uniform Temperature [3.12.2]

Superseded by SDG 2.7.2 and SDG 6.4.

Shrinkage and Creep [5.4.2.3]

Movement and Loads - General [14.4.1]

Bridge Joints [14.5]

FDOT Criteria

Uniform Temperature - Joints and Bearings [SDG 2.7.2]

Delete LRFD [3.12.2] and substitute in lieu thereof SDG Chapter 6.

Expansion Joints [SDG 6.4]

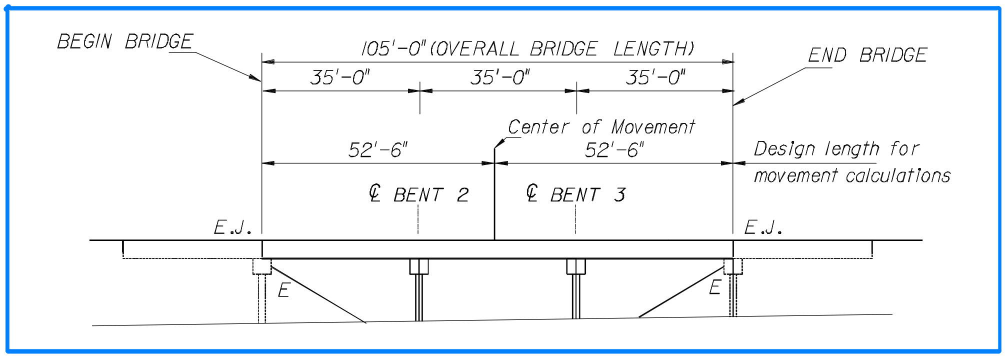
A. Input Variables

A1. Bridge Geometry

Overall bridge length....... $L_{bridge} = 105\ ft$

Bridge design span length.. $L_{span} = 35\ ft$

Skew angle.................... $Skew = -30\ deg$



Design length for movement $L_{design} := 52.5 \cdot ft$

A2. Temperature Movement [SDG 6.3]

Structural Material of Superstructure	**Temperature** (Degrees Fahrenheit) Mean	High	Low	Range
Concrete Only	70	95	45	50
Concrete Deck on Steel Girder	70	110	30	80
Steel Only	70	120	30	90

The temperature values for "Concrete Only" in the preceding table apply to this example.

Temperature mean.... $t_{mean} = 70\,°F$

Temperature high..... $t_{high} = 95\,°F$

Temperature low...... $t_{low} = 45\,°F$

Temperature rise...... $\Delta t_{rise} := t_{high} - t_{mean}$ $\Delta t_{rise} = 25\,°F$

Temperature fall....... $\Delta t_{fall} := t_{mean} - t_{low}$ $\Delta t_{fall} = 25\,°F$

Coefficient of thermal expansion **[LRFD 5.4.2.2]** for normal weight concrete............ $\alpha_t = 6 \times 10^{-6} \frac{1}{°F}$

A3. Expansion Joints [SDG 6.4]

Joint Type	Maximum Joint Width *
Poured Rubber	¾"
Silicone Seal	2"
Strip Seal	3"
Modular Joint	Unlimited
Finger Joint	Unlimited

*Joints in sidewalks must meet all requirements of Americans with Disabilities Act.

For new construction, use only the joint types listed in the preceding table. A typical joint for most flat slab bridges is the silicone seal.

Maximum joint width....................... $W_{max} := 2 \cdot in$

Minimum joint width at 70º F............ $W_{min} := \frac{5}{8} \cdot in$

Proposed joint width at 70º F........... $W := 1 \cdot in$

A4. Movement [6.4.2]

Temperature

The movement along the beam due to temperature should be resolved along the axis of the expansion joint or skew.

Displacements normal to skew at top of bents

Temperature rise......... $\Delta z_{TempR} := \alpha_t \cdot \Delta t_{rise} \cdot \cos(|Skew|) \cdot L_{design}$ $\Delta z_{TempR} = 0.08\,in$

Temperature Fall........ $\Delta z_{TempF} := \alpha_t \cdot \Delta t_{fall} \cdot \cos(|Skew|) \cdot L_{design}$ $\Delta z_{TempF} = 0.08\,in$

Displacements parallel to skew at top of bents

Temperature rise........ $\Delta x_{TempR} := \alpha_t \cdot \Delta t_{rise} \cdot \sin(|Skew|) \cdot L_{design}$ $\Delta x_{TempR} = 0.05\,in$

Temperature Fall........ $\Delta x_{TempF} := \alpha_t \cdot \Delta t_{fall} \cdot (\sin(|Skew|) \cdot L_{design})$ $\Delta x_{TempF} = 0.05\,in$

For silicone seals, displacements parallel to the skew are not significant in most joint designs. For this example, these displacements are ignored.

Creep and Shrinkage

The following assumptions are used in this design example:

- Creep of the concrete for expansion joint design is ignored.
- Shrinkage of the concrete for the flat slab is cast-in-place flat slab will be taken as per LRFD 5.4.3.2.1 as the total shrinkage after one year of drying.

Creep strain.. $\varepsilon_{CR} := 0.$

Shrinkage strain.................................... $\varepsilon_{SH} := 0.0005$

Strain due to creep and shrinkage

$\varepsilon_{CS} := \varepsilon_{CR} + \varepsilon_{SH}$ $\qquad$ $\varepsilon_{CS} = 0.00050$

The movement along the beam due to creep and shrinkage should be resolved along the axis of the expansion joint or skew.

Displacements normal to skew at top of bents.. $\Delta z_{CS} := \varepsilon_{CS} \cdot \cos(|Skew|) \cdot L_{design}$

$\Delta z_{CS} = 0.27\,in$

Displacements parallel to skew at top of bents.. $\Delta x_{CS} := \varepsilon_{CS} \cdot \sin(|Skew|) \cdot L_{design}$

$\Delta x_{CS} = 0.16\,in$

For silicone seals, displacements parallel to the skew are not significant in most joint designs. For this example, these displacements are ignored.

B. Expansion Joint Design

For conventional concrete structures, the movement is based on the greater of two cases:

- Movement from the combination of temperature fall, creep, and shrinkage
- Movement from factored effects of temperature

B1. Movement from Creep, Shrinkage and Temperature (SDG 6.4.2)

The combination of creep, shrinkage, and temperature fall tends to "open" the expansion joint.

Movement from the combination of temperature fall, creep, and shrinkage.......

$$\Delta z_{Temperature.Fall} = \Delta z_{temperature.fall} + \Delta z_{creep.shrinkage}$$

Using variables defined in this example......

$$\Delta_{CST} := \Delta z_{CS} + \Delta z_{TempF}$$

$$\Delta_{CST} = 0.35\,\text{in}$$

Joint width from opening caused by creep, shrinkage, and temperature......................

$$W_{CSTopen} := W + \Delta_{CST}$$

$$W_{CSTopen} = 1.35\,\text{in}$$

The joint width from opening should not exceed the maximum joint width.

$$CST_{Jt\_Open} := \begin{array}{|l} \text{"OK, joint width does not exceed maximum joint width"} \quad \text{if } W_{CSTopen} \le W_{max} \\ \text{"NG, joint width exceeds maximum joint width"} \quad \text{otherwise} \end{array}$$

$$CST_{Jt\_Open} = \text{"OK, joint width does not exceed maximum joint width"}$$

B2. Movement from Temperature (SDG 6.4.2)

Movement from factored effects of temperature rise

$$\Delta z_{rise.or.fall} = 1.15 \cdot \Delta z_{temperature.rise.or.fall}$$

Using variables defined in this example,

Joint width from opening caused by factored temperature fall.....................................

$$W_{Topen} := W + 1.15 \cdot \Delta z_{TempF}$$

$$W_{Topen} = 1.09\,\text{in}$$

Joint width from closing caused by factored temperature rise.....................................

$$W_{Tclose} := W - 1.15 \cdot \Delta z_{TempR}$$

$$W_{Tclose} = 0.91\,\text{in}$$

The joint width from opening should not exceed the maximum joint width.

$$\text{Temperature}_{Jt\_Open} := \begin{cases} \text{"OK, joint width does not exceed maximum joint width"} & \text{if } W_{Topen} \le W_{max} \\ \text{"NG, joint width exceeds maximum joint width"} & \text{otherwise} \end{cases}$$

$$\text{Temperature}_{Jt\_Open} = \text{"OK, joint width does not exceed maximum joint width"}$$

The joint width from closing should not be less than the minimum joint width.

$$\text{Temperature}_{Jt\_Close} := \begin{cases} \text{"OK, joint width is not less than minimum joint width"} & \text{if } W_{Tclose} \ge W_{min} \\ \text{"NG, joint width exceeds minimum joint width"} & \text{otherwise} \end{cases}$$

$$\text{Temperature}_{Jt\_Close} = \text{"OK, joint width is not less than minimum joint width"}$$

B3. Temperature Adjustment for Field Placement of Joint

For field temperatures other than 70° F, a temperature adjustment is provided. The adjustment is used during construction to obtain the desired joint width..

$$T_{Adj} := \frac{\Delta z_{TempR}}{\Delta t_{rise}}$$

$$T_{Adj} = 0.0033 \frac{in}{°F}$$

B4. Design Movement/Strain

For the lateral forces into the substructure piles, the following strain due to temperature, creep and shrinkage will be utilized....................................

$$\varepsilon_{CST} := \left(\varepsilon_{CR} + \varepsilon_{SH} + \alpha_t \cdot \Delta t_{fall}\right)$$

$$\varepsilon_{CST} = 0.00065$$

C. Design Summary

Joint width at 70°.................................. $W = 1\ in$

Joint width from opening caused by creep, shrinkage, and temperature..................... $W_{CSTopen} = 1.35\ in$

CST_{Jt_Open} = "OK, joint width does not exceed maximum joint width" .... $W_{max} = 2\ in$

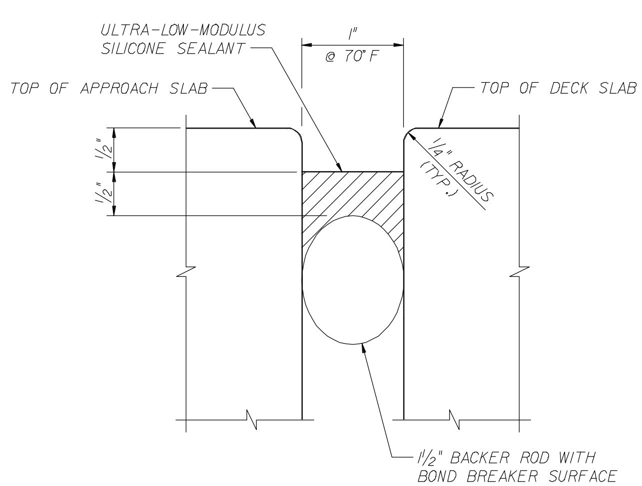
Joint width from opening caused by factored temperature............................. $W_{Topen} = 1.09\ in$

$Temperature_{Jt\_Open}$ = "OK, joint width does not exceed maximum joint width" .... $W_{max} = 2\ in$

Joint width from closing caused by factored temperature............................. $W_{Tclose} = 0.91\ in$

$Temperature_{Jt\_Close}$ = "OK, joint width is not less than minimum joint width" .... $W_{min} = 0.625\ in$

Adjustment for field temperatures other than 70°... $T_{Adj} = 0.0033\ \frac{in}{°F}$



EXPANSION JOINT DETAIL

Defined Units

Bent 2 Cap Design Loads

Reference

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\205ExpJt.mcd(R)

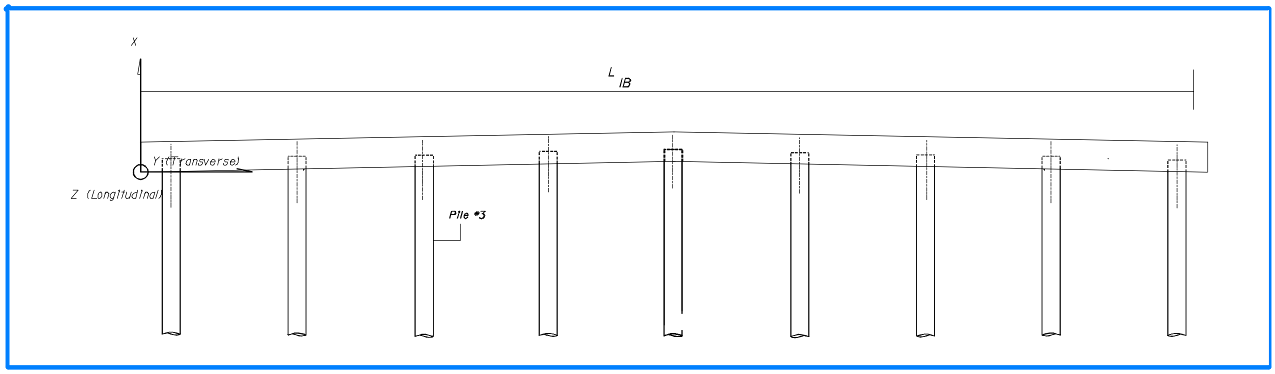
Description

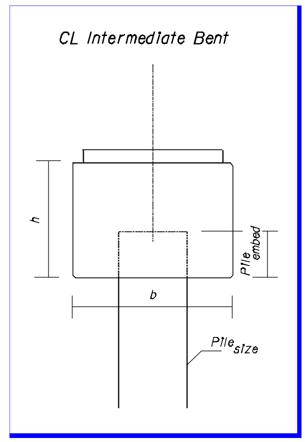
This section provides the design dead loads applied to the substructure from the superstructure. The self-weight of the substructure is generated by the analysis program for the substructure model.

Page Contents

A. General Criteria

A1. Intermediate Bent Geometry





Depth of intermediate bent cap.... $h = 2.5 \text{ ft}$

Width of intermediate bent cap..... $b = 3.5 \text{ ft}$

Length of intermediate bent cap... $L = 102.86 \text{ ft}$

Pile Embedment Depth................ $Pile_{embed} = 1 \text{ ft}$

Pile Size.................................... $Pile_{size} = 18 \text{ in}$

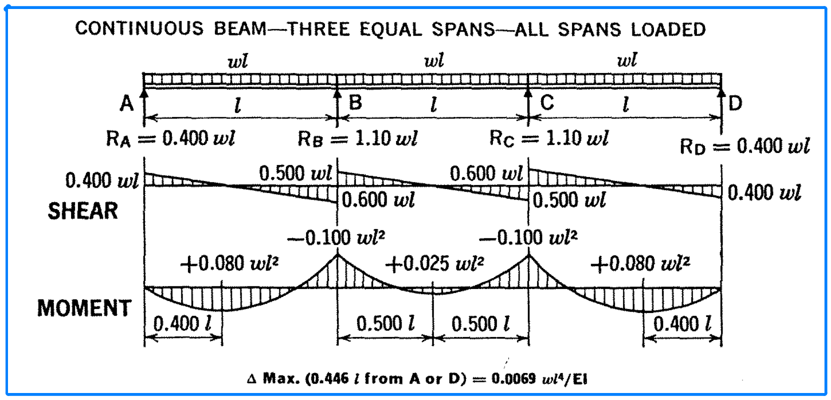
*(**Note**: For this design example, only the intermediate bent will be evaluated).*

B. Loads (DC, DW, LL)

B1. Longitudinal Analysis

Dead Loads

The dead loads of the superstructure (moment and shears) were previously computed on a per foot basis utilizing the AISC's Moments, Shears and Reactions for Continuous Highway Bridges, published 1966. The dead loads and shear could have been calculated utilizing the AISC's Steel Construction Manual - Beam Diagrams and Deflections charts. Based on the following chart, the reactions at the intermediate bent (Point B) can be calculated.



Reaction at B: $R_B = V_{left} + V_{right}$

where based on previous calculations for dead loads:

$w_{DC} := 0.240 \cdot ksf$

$w_{DW} := 0.015 \cdot ksf$

For a 1' design strip, $V_{DC.left} := 0.6 \cdot w_{DC} \cdot L_{span}$ $\quad V_{DC.left} = 5.04\,klf$

$V_{DC.right} := 0.5 \cdot w_{DC} \cdot L_{span}$ $\quad V_{DC.right} = 4.2\,klf$

similarly $V_{DW.left} := 0.6 \cdot w_{DW} \cdot L_{span}$ $\quad V_{DW.left} = 0.32\,klf$

$V_{DW.right} := 0.5 \cdot w_{DW} \cdot L_{span}$ $\quad V_{DW.right} = 0.26\,klf$

the reactions at B: $R_{DC} := V_{DC.left} + V_{DC.right}$ $\quad R_{DC} = 9.2\,klf$

$R_{DW} := V_{DW.left} + V_{DW.right}$ $\quad R_{DW} = 0.6\,klf$

*(**Note**: These are the same values summarized in Sect. 2.01 Design Loads - Dead Load Analysis utilizing*

Live loads

The live load reaction at the intermediate bent can be computed utilizing computer programs or similar methods. For purposes of this design example, the HL-93 live load reaction at B is given as:

HL-93 live load reaction at B $R_{LL} := 112.9 \cdot kip$ *(**Note**: Includes lane load and impact on truck; 112.9kip = truck (64.7kipx1.33)+lane(26.88kip)).*

Live load reaction for an interior strip, $E = 12.5\,ft$ .... $R_{LL.Interior} := \dfrac{R_{LL}}{E}$

$R_{LL.Interior} = 9\,klf$

Since the live load applied to the edge beam is different than an interior strip, the live load reaction for the edge beam is computed separately,

HL-93 live load reaction at B.. $R_{LL.EB} := 55.0 \cdot kip$ *(**Note**: Includes lane load and impact on truck; 112.9kip = truck (64.7kip)x1.33x0.5 Factor$_{truck}$ + lane(26.88kipx0.446 Factor$_{lane}$)).*

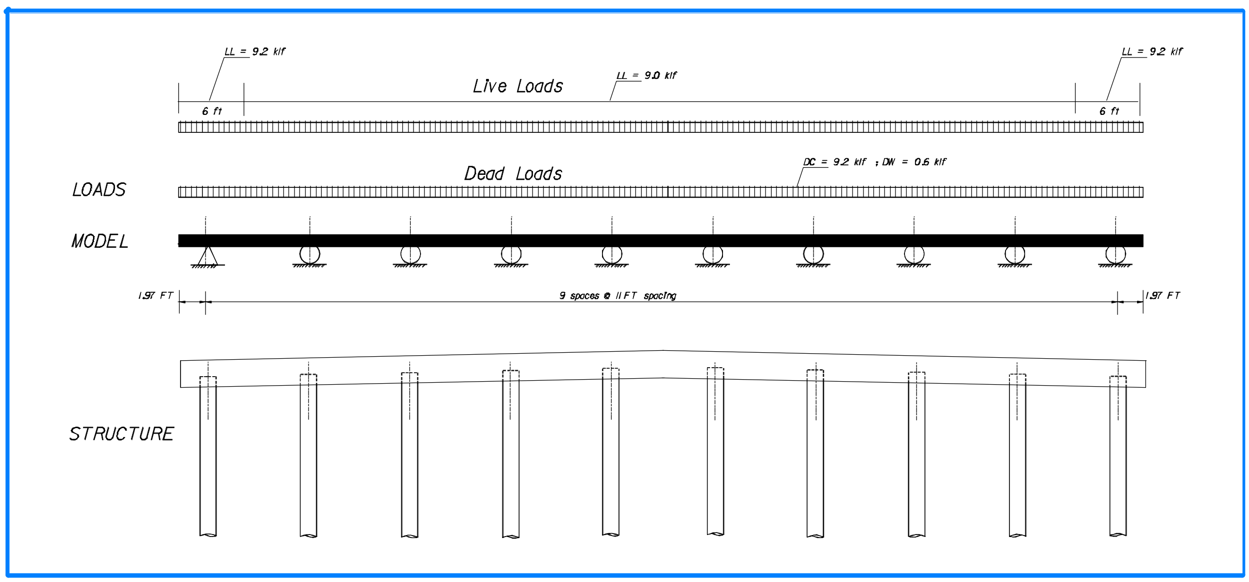
Live load reaction for an edge beam strip, $E_{EB} = 6\,ft$ .... $R_{LL.EB} := \dfrac{R_{LL.EB}}{E_{EB}}$

$R_{LL.EB} = 9.2\,klf$

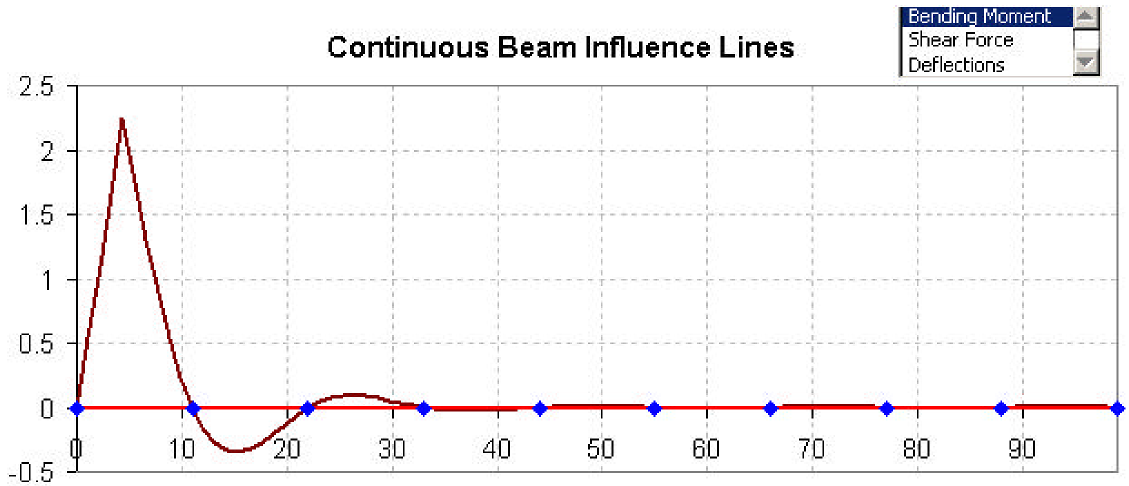
B1. Transverse Analysis

The loads calculated in the longitudinal analysis can be applied transversely for (1) design of the pier cap and (2) design of the maximum pile force.

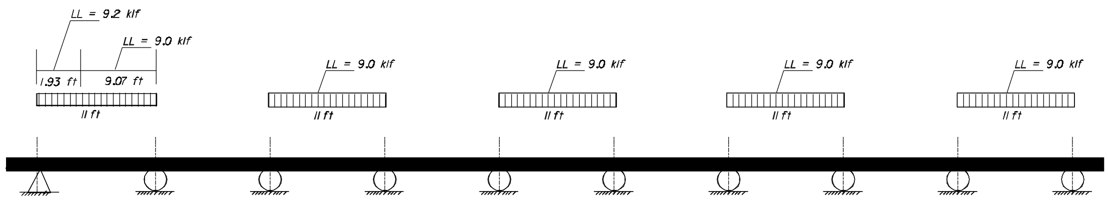
Pier cap design



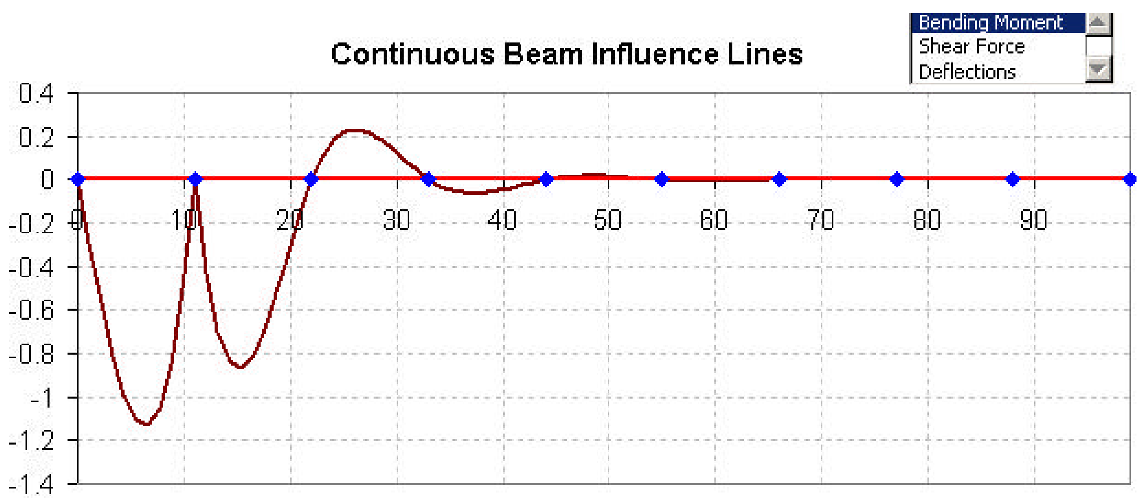
The live loads can be positioned to maximize the loads for the design of the intermediate end bent cap. For instance, for the maximum positive moment in the intermediate end bent cap, the influence line is shaped as follows:



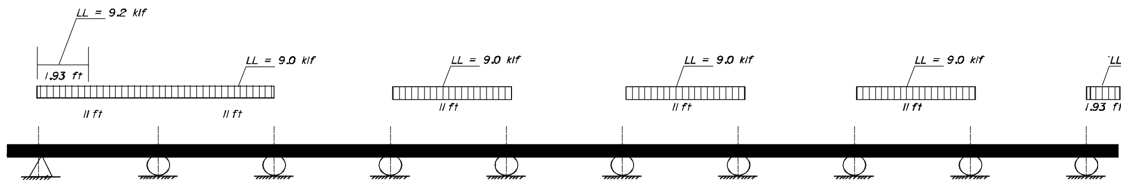
the corresponding live load loading is therefore,



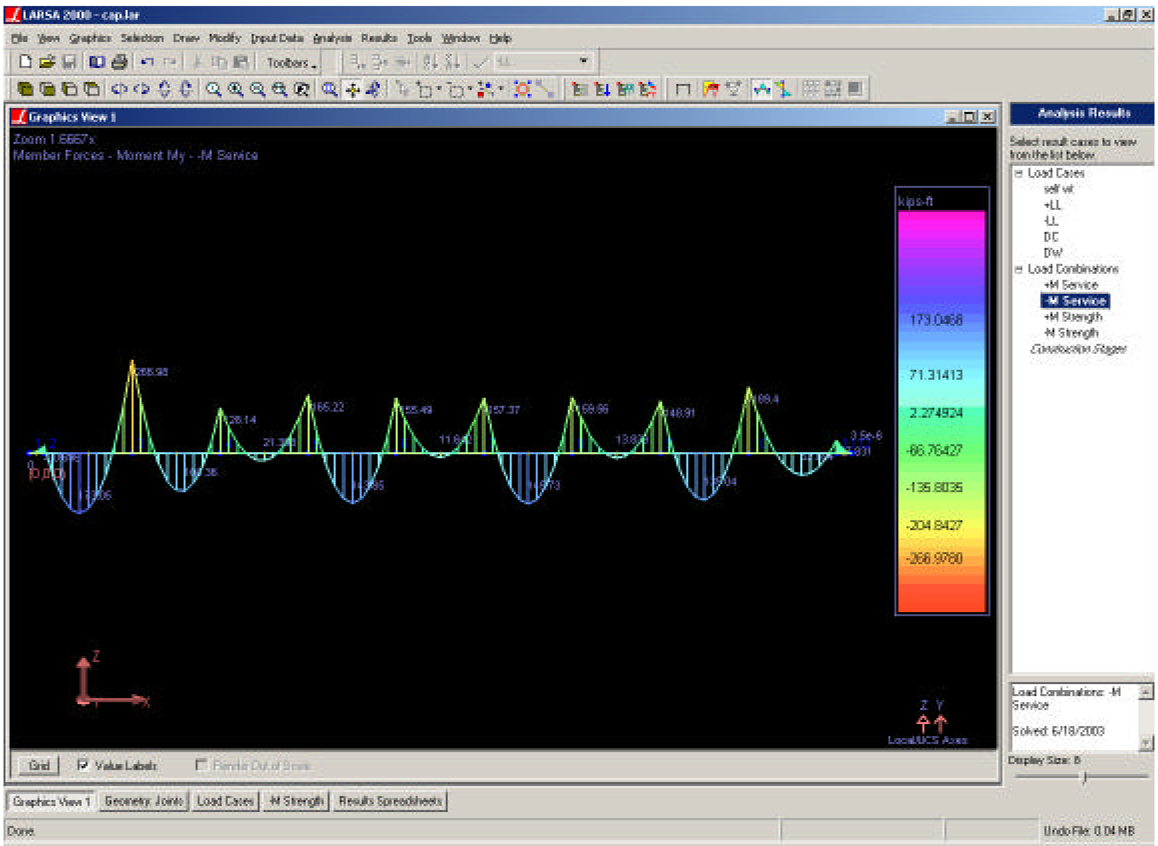
For the maximum negative moment in the intermediate end bent cap, the influence line is shaped as follows:



the corresponding live load loading is therefore,



The dead load DC, DW for both superstructure and cap were evaluated, combined with the appropriate live load utilizing LARSA 2000. Any frame analysis program could be utilized to obtain the results. In addition, the load combinations were performed within LARSA for both the Service I and Strength I limit states. The following is a summary of the results:



LARSA 2000 Analysis Results

	Max. +M (ft-kip)	V (kip)	Max. -M (ft-kip)	V (kip)
Superstructure DC Moment	78.1	59.3	113.0	59.3
Superstructure DW Moment	5.1	3.9	7.4	3.9
Substructure Cap DC Moment	11.1	8.5	16.1	8.5
LL Moment	107.9	61.4	130.5	61.4
Service I Limit State	202.4	126.6	267.3	133.2
Strength I Limit State	352.5	220.5	465.9	232.1
Max. Service I Reaction	191.9	---	256.5	---
Max. Strength I Reaction	334.1	---	447.0	---

Defined Units

Bent 2 Cap Design

References

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\301DsnLds.mcd(R)

Description

This section provides the criteria for the intermediate bent cap design.

Page Contents

A. Input Variables

Design Loads - Moments and Shears

Moment (-M) - Service.......................... $M_{Service1.neg} = 267.3\ ft \cdot kip$

Moment (-M) - Strength......................... $M_{Strength1.neg} = 465.9\ ft \cdot kip$

Corresponding Shear (-M) - Strength........ $V_{Strength1.neg} = 232.1\ kip$

Moment (+M) - Service.......................... $M_{Service1.pos} = 202.3\ ft \cdot kip$

Moment (+M) - Strength......................... $M_{Strength1.pos} = 352.5\ ft \cdot kip$

Corresponding Shear (+M) - Strength....... $V_{Strength1.pos} = 220.5\ kip$

B. Positive Moment Design

A few recommendations on bar size and spacing are available to minimize problems during construction.

- Use the same size and spacing of reinforcing for both the negative and positive moment regions. This prevents field errors whereas the top steel is mistakenly placed at the bottom or vice versa.
- If this arrangement is not possible, give preference to maintaining the same spacing between the top and bottom reinforcement. Same grid pattern allows the concrete vibrator to be more effective in reaching the full depth of the cap.

The design procedure consists of calculating the reinforcement required to satisfy the design moment, then checking this reinforcement against criteria for crack control, minimum reinforcement, maximum reinforcement, shrinkage and temperature reinforcement, and distribution of reinforcement. The procedure is the same for both positive and negative moment regions.

$$M_r := M_{Strength1.pos}$$

$$M_r = 352.5\,ft\cdot kip$$

Factored resistance

$$M_r = \phi \cdot M_n$$

Nominal flexural resistance

$$M_n = A_{ps}\cdot f_{ps}\cdot\left(d_p - \frac{a}{2}\right) + A_s\cdot f_y\cdot\left(d_s - \frac{a}{2}\right) - A'_s\cdot f'_y\cdot\left(d'_s - \frac{a}{2}\right) + 0.85\cdot f'_c\cdot\left(b - b_w\right)\cdot\beta_1\cdot h_f\cdot\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

For a rectangular, non-prestressed section,

$$M_n = A_s\cdot f_y\cdot\left(d_s - \frac{a}{2}\right)$$

$$a = \frac{A_s\cdot f_y}{0.85\cdot f'_c\cdot b}$$

B1. Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Using variables defined in this example......

$$M_r = \phi\cdot A_{s.pos}\cdot f_y\cdot\left[d_s - \frac{1}{2}\cdot\left(\frac{A_{s.pos}\cdot f_y}{0.85\cdot f_{c.slab}\cdot b}\right)\right]$$

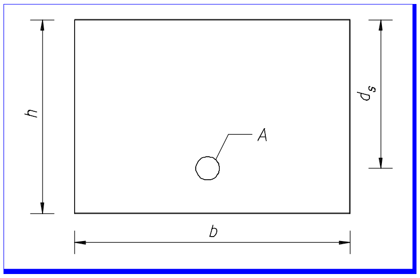
where $f_{c.sub} = 5.5\,ksi$

$f_y = 60\,ksi$

$\phi = 0.9$

$h = 30\,in$

$b = 42\,in$



Initial assumption for area of steel required

Number of bars............................... $n_{bar} := 5$

Size of bar...................................... $bar := "8"$

Note: if bar spacing is "-1", the spacing is less than 3", and a bigger bar size should be selected.

Bar area.. $A_{bar} = 0.790\,in^2$

Bar diameter................................... $dia = 1.000\,in$

Equivalent bar spacing....................... $bar_{spa} = 8.7\,in$

Area of steel provided............................ $A_s := n_{bar} \cdot A_{bar}$

$A_s = 3.95\,in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel (assuming a #5 stirrup).. $d_s := h - cover_{sub} - \frac{dia}{2} - \frac{5}{8} in$

$d_s = 25.9\,in$

Solve the quadratic equation for the area of steel required....................................... Given $M_r = \phi \cdot A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b} \right) \right]$

Area of steel required............................. $A_{s.reqd} := Find(A_s)$

$A_{s.reqd} = 3.25\,in^2$

The area of steel provided, $A_s = 3.95\,in^2$, should be greater than the area of steel required, $A_{s.reqd} = 3.25\,in^2$. If not, decrease the spacing of the reinforcement. Once A_s is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the applied moments.

Moment capacity provided.................... $M_{r.pos} := \phi \cdot A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b} \right) \right]$

$M_{r.pos} = 449.2\,ft \cdot kip$

B2. Limits for Reinforcement [LRFD 5.7.3.3]

Maximum Reinforcement

The maximum reinforcement requirements ensure the section has sufficient ductility and is not overreinforced.

Area of steel provided............................

$A_s = 3.95\,in^2$

Stress block factor................................ $\beta_1 := \max\left[0.85 - 0.05\cdot\left(\frac{f_{c.sub} - 4000\cdot psi}{1000\cdot psi}\right), 0.65\right]$

$\beta_1 = 0.775$

Distance from extreme compression fiber to the neutral axis of section................... $c := \frac{A_s\cdot f_y}{0.85\cdot f_{c.sub}\cdot \beta_1\cdot b}$

$c = 1.6\,in$

Effective depth from extreme compression fiber to centroid of the tensile reinforcement...................................... $d_e = \frac{A_{ps}\cdot f_{ps}\cdot d_p + A_s\cdot f_y\cdot d_s}{A_{ps}\cdot f_{ps} + A_s\cdot f_y}$

for non-prestressed sections............. $d_e := d_s$

$d_e = 25.9\,in$

The $\frac{c}{d_e} = 0.06$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$LRFD_{5.7.3.3.1} :=$ | "OK, maximum reinforcement requirements for positive moment are satisfied" if $\frac{c}{d_e} \le 0.42$
| "NG, section is over-reinforced, see LRFD equation C5.7.3.3.1-1" otherwise

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement requirements for positive moment are satisfied"

Minimum Reinforcement

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture.............................. $f_r := 0.24\cdot\sqrt{f_{c.sub}\cdot ksi}$

$f_r = 562.8\,psi$

Section modulus of cap.......................... $S := \frac{b\cdot h^2}{6}$

$S = 3.6\,ft^3$

Cracking moment................................. $M_{cr} := f_r\cdot S$

$M_{cr} = 295.5\,kip\cdot ft$

Required flexural resistance............ $M_{r.reqd} := \min(1.2 \cdot M_{cr}, 133 \cdot \% \cdot M_r)$

$M_{r.reqd} = 354.6\ ft \cdot kip$

Check that the capacity provided, $M_{r.pos} = 449.2\ ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 354.6\ ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{cases} \text{"OK, minimum reinforcement for positive moment is satisfied"} & \text{if } M_{r.pos} \geq M_{r.reqd} \\ \text{"NG, reinforcement for positive moment is less than minimum"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for positive moment is satisfied"

B3. Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

Stress in the mild steel reinforcement at the service limit state..................................

$$f_{sa} = \frac{z}{(d_c \cdot A)^{\frac{1}{3}}} \leq 0.6 \cdot f_y$$

Crack width parameter...........................

$$z = \begin{pmatrix} \text{"moderate exposure"} & 170 \\ \text{"severe exposure"} & 130 \\ \text{"buried structures"} & 100 \end{pmatrix} \cdot \frac{kip}{in}$$

The environmental classifications for Florida designs do not match the classifications to select the crack width parameter. For this example, a "Slightly" or "Moderately" aggressive environment corresponds to "moderate exposure" and an "Extremely" aggressive environment corresponds to "severe exposure".

$Environment_{super} =$ "Slightly" aggressive environment

$$z := 170 \cdot \frac{kip}{in}$$

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)..................................

$$d_c := \min\left(h - d_s, 2 \cdot in + \frac{dia}{2}\right)$$

$d_c = 2.5\ in$

Number of bars per design width of slab... $n_{bar} = 5$

Effective tension area of concrete surrounding the flexural tension reinforcement......................................

$$A := \frac{(b) \cdot (2 \cdot d_c)}{n_{bar}}$$

$A = 42.0\ in^2$

Service limit state stress in reinforcement..

$$f_{sa} := \min\left[\frac{z}{(d_c \cdot A)^{\frac{1}{3}}}, 0.6 \cdot f_y\right]$$

$f_{sa} = 36.0\,\text{ksi}$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 9.1 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_{c.sub}} \cdot A_s \cdot (d_s - x)$

$x_{na} := \text{Find}(x)$

$x_{na} = 9.1\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 9.1\,in$ to equal $x_{na} = 9.1\,in$.

Tensile force in the reinforcing steel due to service limit state moment. ....

$$T_s := \frac{M_{Service1.pos}}{d_s - \frac{x_{na}}{3}}$$

$T_s = 106.343\,\text{kip}$

Actual stress in the reinforcing steel due to service limit state moment......................

$$f_{s.actual} := \frac{T_s}{A_s}$$

$f_{s.actual} = 26.9\,\text{ksi}$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$LRFD_{5.7.3.3.4} :=$ | "OK, crack control for positive moment is satisfied" if $f_{s.actual} \le f_{sa}$
| "NG, crack control for positive moment not satisfied, provide more reinforcement" otherwise

$LRFD_{5.7.3.3.4} =$ "OK, crack control for positive moment is satisfied"

B4. Shrinkage and Temperature Reinforcement [LRFD 5.10.8.2]

Initial assumption for area of steel required

Size of bar......................................

$$bar_{st} := \begin{cases} "5" & \text{if } (b < 48in)\cdot(h < 48in) \\ "6" & \text{otherwise} \end{cases}$$

$bar_{st} = "5"$

Spacing of bar................................ $bar_{spa.st} := 12\cdot in$

Bar area.. $A_{bar} = 0.31\, in^2$

Bar diameter................................... $dia = 0.625\, in$

Gross area of section............................. $A_g := b\cdot h$

$A_g = 1260.0\, in^2$

Minimum area of shrinkage and temperature reinforcement..................... $A_{shrink.temp} := 0.0015\cdot A_g$

$A_{shrink.temp} = 1.9\, in^2$

Maximum spacing of shrinkage and temperature reinforcement

$$spacing_{shrink.temp} := \begin{cases} \min\left(\dfrac{b}{\dfrac{A_{shrink.temp}}{A_{bar}\cdot 2}}, 12\cdot in\right) & \text{if } (b < 48in)\cdot(h < 48in) \\ \dfrac{100\cdot A_{bar}}{\min(2\cdot d_c + dia, 3in)} & \text{otherwise} \end{cases}$$

$spacing_{shrink.temp} = 12.0\, in$

The bar spacing should be less than the maximum spacing for shrinkage and temperature reinforcement

$$LRFD_{5.7.10.8} := \begin{cases} \text{"OK, minimum shrinkage and temperature requirements"} & \text{if } bar_{spa.st} \le spacing_{shrink.temp} \\ \text{"NG, minimum shrinkage and temperature requirements"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.10.8} =$ "OK, minimum shrinkage and temperature requirements"

B5. Mass Concrete Provisions

Surface area of pier cap.......................... $Surface_{cap} := 2 \cdot b \cdot h + (2b + 2h) \cdot L$

$Surface_{cap} = 1251.8\, ft^2$

Volume of pier cap................................ $Volume_{cap} := b \cdot h \cdot L$

$Volume_{cap} = 900\, ft^3$

Mass concrete provisions apply if the volume to surface area ratio, $\frac{Volume_{cap}}{Surface_{cap}} = 0.719\, ft$, exceeds 1 ft and any dimension exceeds 3 feet

$$SDG_{3.9} := \begin{cases} \text{"Use mass concrete provisions"} & \text{if } \frac{Volume_{cap}}{Surface_{cap}} > 1.0 \cdot ft \wedge (b > 3ft \vee h > 3ft) \\ \text{"Use regular concrete provisions"} & \text{otherwise} \end{cases}$$

$SDG_{3.9} =$ "Use regular concrete provisions"

C. Negative Moment Design

A few recommendations on bar size and spacing are available to minimize problems during construction.

The same size and spacing of reinforcing should be utilized for both the negative and positive moment regions.

If this arrangement is not possible, the top and bottom reinforcement should be spaced as a multiple of each other. This pattern places the top and bottom bars in the same grid pattern, and any additional steel is placed between these bars.

The design procedure consists of calculating the reinforcement required to satisfy the design moment, then checking this reinforcement against criteria for crack control, minimum reinforcement, maximum reinforcement, shrinkage and temperature reinforcement, and distribution of reinforcement. The procedure is the same for both positive and negative moment regions.

$$M_r := \left|M_{Strength1.neg}\right|$$

$$M_r = 465.9\ ft \cdot kip$$

Factored resistance

$$M_r = \phi \cdot M_n$$

Nominal flexural resistance

$$M_n = A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right) - A'_s \cdot f'_y \cdot \left(d'_s - \frac{a}{2}\right) + 0.85 \cdot f'_c \cdot (b - b_w) \cdot \beta_1 \cdot h_f \cdot \left(\frac{a}{2} - \frac{h_f}{2}\right)$$

For a rectangular, non-prestressed section,

$$M_n = A_s \cdot f_y \cdot \left(d_s - \frac{a}{2}\right)$$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

C1. Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Using variables defined in this example,

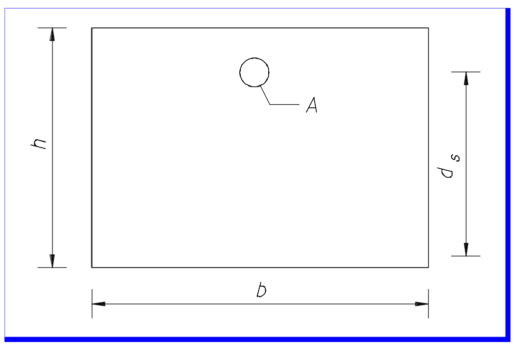
where $f_{c.sub} = 5.5\ ksi$

$f_y = 60\ ksi$

$\phi = 0.9$

$h = 30\ in$

$b = 42\ in$



Initial assumption for area of steel required

Number of bars............................... $n_{bar} := 6$

Size of bar...................................... $bar := "8"$

Note: if bar spacing is "-1", the spacing is less than 3", and a bigger bar size should be selected.

Bar area... $A_{bar} = 0.790\,in^2$

Bar diameter................................... $dia = 1.000\,in$

Equivalent bar spacing..................... $bar_{spa} = 8.7\,in$

Area of steel provided............................ $A_s := n_{bar} \cdot A_{bar}$

$A_s = 4.74\,in^2$

Distance from extreme compressive fiber to centroid of reinforcing steel (assuming a #5 stirrup)... $d_s := h - cover_{sub} - \frac{dia}{2} - \frac{5}{8}in$

$d_s = 25.9\,in$

Solve the quadratic equation for the area of steel required.. Given $M_r = \phi \cdot A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b} \right) \right]$

Area of steel required............................ $A_{s.reqd} := Find(A_s)$

$A_{s.reqd} = 4.10\,in^2$

The area of steel provided, $A_s = 4.74\,in^2$, should be greater than the area of steel required, $A_{s.reqd} = 4.10\,in^2$. If not, decrease the spacing of the reinforcement. Once A_s is greater than $A_{s.reqd}$, the proposed reinforcing is adequate for the applied moments.

Moment capacity provided.................... $M_{r.neg} := \phi \cdot A_s \cdot f_y \cdot \left[d_s - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b} \right) \right]$

$M_{r.neg} = 536.5\,ft \cdot kip$

C2. Limits for Reinforcement [LRFD 5.7.3.3]

Maximum Reinforcement

The maximum reinforcement requirements ensure the section has sufficient ductility and is not overreinforced.

Area of steel provided............................

$A_s = 4.74\,in^2$

Stress block factor................................ $\beta_1 := \max\left[0.85 - 0.05\cdot\left(\frac{f_{c.sub} - 4000\cdot psi}{1000\cdot psi}\right), 0.65\right]$

$\beta_1 = 0.775$

Distance from extreme compression fiber to the neutral axis of section................... $c := \frac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot \beta_1 \cdot b}$

$c = 1.9\, in$

Effective depth from extreme compression fiber to centroid of the tensile reinforcement...................................... $d_e = \frac{A_{ps}\cdot f_{ps}\cdot d_p + A_s\cdot f_y\cdot d_s}{A_{ps}\cdot f_{ps} + A_s\cdot f_y}$

for non-prestressed sections............. $d_e := d_s$

$d_e = 25.9\, in$

The $\frac{c}{d_e} = 0.072$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

$LRFD_{5.7.3.3.1} :=$ | "OK, maximum reinforcement requirements for negative moment are satisfied" if $\frac{c}{d_e} \le 0.42$
"NG, section is over-reinforced, see LRFD equation C5.7.3.3.1-1" otherwise

$LRFD_{5.7.3.3.1} =$ "OK, maximum reinforcement requirements for negative moment are satisfied"

Minimum Reinforcement

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture............................... $f_r := 0.24\cdot\sqrt{f_{c.sub}\cdot ksi}$

$f_r = 562.8\, psi$

Distance from the extreme tensile fiber to the neutral axis of the composite section... $y := \frac{h}{2}$

$y = 15.0\, in$

Moment of inertia for the section............ $I := \frac{1}{12}\cdot b\cdot h^3$

$I = 4.6\, ft^4$

Section modulus of cap.......................... $S := \frac{b\cdot h^2}{6}$

$S = 3.6\, ft^3$

Cracking moment................................. $M_{cr} := f_r\cdot S$

$M_{cr} = 295.5\, kip\cdot ft$

Required flexural resistance............ $M_{r.reqd} := \min(1.2 \cdot M_{cr}, 133 \cdot \% \cdot M_r)$

$M_{r.reqd} = 354.6\,ft \cdot kip$

Check that the capacity provided, $M_{r.neg} = 536.5\,ft \cdot kip$, exceeds minimum requirements, $M_{r.reqd} = 354.6\,ft \cdot kip$.

$$LRFD_{5.7.3.3.2} := \begin{cases} \text{"OK, minimum reinforcement for negative moment is satisfied"} & \text{if } M_{r.neg} \geq M_{r.reqd} \\ \text{"NG, reinforcement for negative moment is less than minimum"} & \text{otherwise} \end{cases}$$

$LRFD_{5.7.3.3.2} =$ "OK, minimum reinforcement for negative moment is satisfied"

C3. Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

Stress in the mild steel reinforcement at the service limit state...................................

$$f_{sa} = \frac{z}{(d_c \cdot A)^{\frac{1}{3}}} \leq 0.6 \cdot f_y$$

Crack width parameter..........................

$$z = \begin{pmatrix} \text{"moderate exposure"} & 170 \\ \text{"severe exposure"} & 130 \\ \text{"buried structures"} & 100 \end{pmatrix} \cdot \frac{kip}{in}$$

The environmental classifications for Florida designs do not match the classifications to select the crack width parameter. For this example, a "Slightly" or "Moderately" aggressive environment corresponds to "moderate exposure" and an "Extremely" aggressive environment corresponds to "severe exposure".

$Environment_{super} =$ "Slightly"

$z := 170 \cdot \frac{kip}{in}$

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)...................................

$$d_c := \min\left(h - d_s, 2 \cdot in + \frac{dia}{2}\right)$$

$d_c = 2.5\,in$

Number of bars per design width of slab...

$n_{bar} = 6$

Effective tension area of concrete surrounding the flexural tension reinforcement.......................................

$$A := \frac{(b)\cdot(2\cdot d_c)}{n_{bar}}$$

$A = 35.0\,in^2$

Service limit state stress in reinforcement..

$$f_{sa} := \min\left[\frac{z}{(d_c\cdot A)^{\frac{1}{3}}}, 0.6\cdot f_y\right]$$

$f_{sa} = 36.0\,ksi$

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

$x := 9.8\cdot in$

Given $\frac{1}{2}\cdot b\cdot x^2 = \frac{E_s}{E_{c.sub}}\cdot A_s\cdot(d_s - x)$

$x_{na} := \text{Find}(x)$

$x_{na} = 9.8\,in$

Compare the calculated neutral axis x_{na} with the initial assumption x. If the values are not equal, adjust $x = 9.8\,in$ to equal $x_{na} = 9.8\,in$.

Tensile force in the reinforcing steel due to service limit state moment. ....

$$T_s := \frac{|M_{Service1.neg}|}{d_s - \frac{x_{na}}{3}}$$

$T_s = 141.9\,kip$

Actual stress in the reinforcing steel due to service limit state moment......................

$$f_{s.actual} := \frac{T_s}{A_s}$$

$f_{s.actual} = 29.9\,ksi$

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

$LRFD_{5.7.3.3.4} :=$ | "OK, crack control for positive moment is satisfied" if $f_{s.actual} \le f_{sa}$
| "NG, crack control for positive moment not satisfied, provide more reinforcement" otherwise

$LRFD_{5.7.3.3.4} =$ "OK, crack control for positive moment is satisfied"

D. Shear Design [LRFD 5.8]

D1. Determine Nominal Shear Resistance

$V_u := |V_{Strength1.neg}|$

Effective width of the section................. $b_v := b$

$b_v = 42.0\,in$

Effective shear depth............................. $a := \dfrac{A_s \cdot f_y}{0.85 \cdot f_{c.sub} \cdot b}$

$a = 1.448\,in$

$d_v = 25.2\,in$ $\quad d_v := \max\left(d_s - \dfrac{a}{2}, 0.9 \cdot d_s, 0.72 \cdot h\right)$

Determination of β and θ **(LRFD 5.8.3.4)**

The pier cap is a non-prestressed concrete section not subjected to axial tension. It should also have the least amount of transverse reinforcement specified in **LRFD 5.8.2.5** or an overall depth of less than 16 in.

$\beta := 2$

$\theta := 45 \cdot deg$

Nominal shear resistance of concrete section.. $V_c := 0.0316 \cdot \beta \cdot \sqrt{f_{c.sub} \cdot ksi} \cdot b_v \cdot d_v$

$V_c = 156.6\,kip$

D2. Transverse Reinforcement

Transverse reinforcement shall be provided in the pier cap according to **LRFD 5.8.2.4.**

$V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p)$

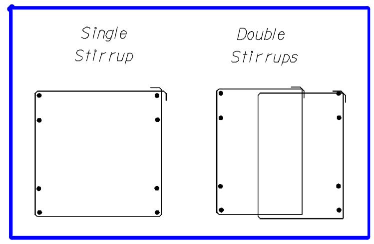
The pier cap has no prestressing.

$V_p := 0 \cdot kip$

Is transverse reinforcement required?

$$LRFD_{5.8.2.4} := \begin{cases} \text{" Transverse reinforcement shall be provided"} & \text{if } V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p) \\ \text{" Transverse reinforcement not required, provide minimum reinforcement"} & \text{otherwise} \end{cases}$$

$LRFD_{5.8.2.4} =$ " Transverse reinforcement shall be provided"



Stirrups

Size of stirrup bar ("4" "5" "6" "7")... $bar := "5"$

Number of stirrup bars ("single" "double") .... $n_{bar} := "double"$

Area of shear reinforcement................... $A_v = 1.240\,in^2$

Diameter of shear reinforcement............. $dia = 0.625\,in$

Nominal shear strength provided by shear reinforcement

$$V_n = V_c + V_p + V_s$$

where... $V_n := \min\left(\frac{V_u}{\phi_v}, 0.25 \cdot f_{c.sub} \cdot b_v \cdot d_v + V_p\right)$

$V_n = 257.9\,kip$

and.. $V_s := V_n - V_c - V_p$

$V_s = 101.3\,kip$

Spacing of stirrups

Minimum transverse reinforcement.......... $s_{min} := \dfrac{A_v \cdot f_y}{0.0316 \cdot b_v \cdot \sqrt{f_{c.sub} \cdot ksi}}$

$s_{min} = 23.9\,in$

Transverse reinforcement required........... $s_{req} := \text{if}\left(V_s \le 0, s_{min}, \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{V_s}\right)$

$s_{req} = 18.5\,in$

Minimum transverse reinforcement required.. $s := \min(s_{min}, s_{req})$

$s = 18.5\,in$

Maximum transverse reinforcement

$$s_{max} := \text{if}\left[\frac{V_u - \phi_v \cdot V_p}{\phi_v \cdot (b_v \cdot d_v)} < 0.125 \cdot f_{c.sub}, \min(0.8 \cdot d_v, 24 \cdot in), \min(0.4 \cdot d_v, 12 \cdot in)\right]$$

$s_{max} = 20.121\,in$

Spacing of transverse reinforcement cannot exceed the following spacing........ $spacing := \text{if}(s_{max} > s, s, s_{max})$

$spacing = 18.5\,in$

E. Summary of Reinforcement Provided in the Moment Region

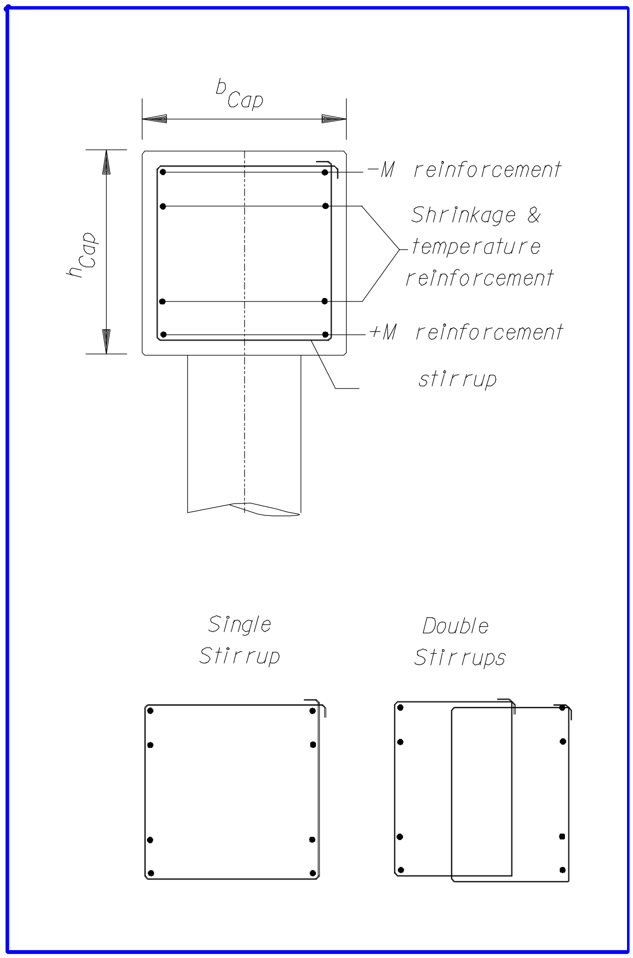
Negative moment (top) reinforcement

Bar size............ $bar_{negM} = "8"$

Number of bars.. $n_{bar.negM} = 6$

Bar spacing....... $bar_{spa.negM} = 8.7\ in$

(***Note***: *As a detailing alternative, 5-#8 bars top and bottom can be specified. In addition, 4-#5 bars can be added in between the #8 bars at the top over the negative moment areas only).*



Positive moment (bottom) reinforcement

Bar size............ $bar_{posM} = "8"$

Number of bars.. $n_{bar.posM} = 5$

Bar spacing....... $bar_{spa.posM} = 8.7\ in$

Transverse reinforcement

Bar size............ $bar = "5"$

Bar spacing....... $spacing = 18.5\ in$

Type of stirrups. $n_{bar} = "double"$

Temperature and Shrinkage

Bar size............ $bar_{shrink.temp} = "5"$

Bar spacing....... $bar_{spa.st} = 12\ in$

▶ Defined Units

Bent 2 Piles Vertical Load Design

References

Reference:F:\HDRDesignExamples\Ex2_FlatSlab\302BentCap.mcd(R)

Description

This section provides the design of the piles for vertical loads (exclude lateral load design). For this design example, only the maximum loaded pile is evaluated.

Page Contents

FDOT Criteria

Minimum Sizes [SDG 3.5.2]

Use 18" square piling, except for extremely aggressive salt water environments.

Spacing, Clearances and Embedment and Size [SDG 3.5.3]

Minimum pile spacing center-to-center must be at least three times the least width of the deep foundation element measured at the ground line.

Resistance Factors [SDG 3.5.5]

The resistance factor utilizing SPT97 for piles under compression shall be... $\phi_{SPT97} := 0.65$

Minimum Pile Tip [SDG 3.5.7]

The minimum pile tip elevation must be the deepest of the minimum elevations that satisfy lateral stability requirements for the three limit states. Since this bridge is not over water, scour and ship impact are not design issues. The design criteria for minimum tip elevation are based on vertical load requirements and lateral load analysis.

Pile Driving Resistance [SDG 3.5.11]

The Required Driving Resistance for an 18" square concrete pile must not exceed.. $UBC_{FDOT\_18} := 300 \cdot Ton$

The Required Driving Resistance for an 24" square concrete pile must not exceed.. $UBC_{FDOT\_24} := 450 \cdot Ton$

A. Input Variables

Maximum Strength I pile reaction $R_{Strength1} = 447\ kip$ or $R_{Strength1} = 223\ Ton$

Required driving resistance (RDR)........... $RDR = UBC = \dfrac{\text{Factored Design Load} + \text{Net Scour} + \text{Downdrag}}{\phi}$

Using variables defined in this example...... $UBC := \dfrac{R_{Strength1}}{\phi_{SPT97}}$

$UBC = 343.8\ Ton$

This value should not exceed the limit specified by FDOT................................. $UBC_{FDOT\_18} = 300\ Ton$

Since the RDR value is exceeded, the consultant needs to evaluate the following costs:

1. Reducing the pile spacing from 11' and adding an extra pile or two
2. Utilizing 24" diameter piles.

For purposes of the design example, pile driving vibrations are not an issue, neither is accessibility to the job site for pile driving equipment; therefore, 24" square piles will be utilized, $UBC_{FDOT\_24} = 450\ Ton$.

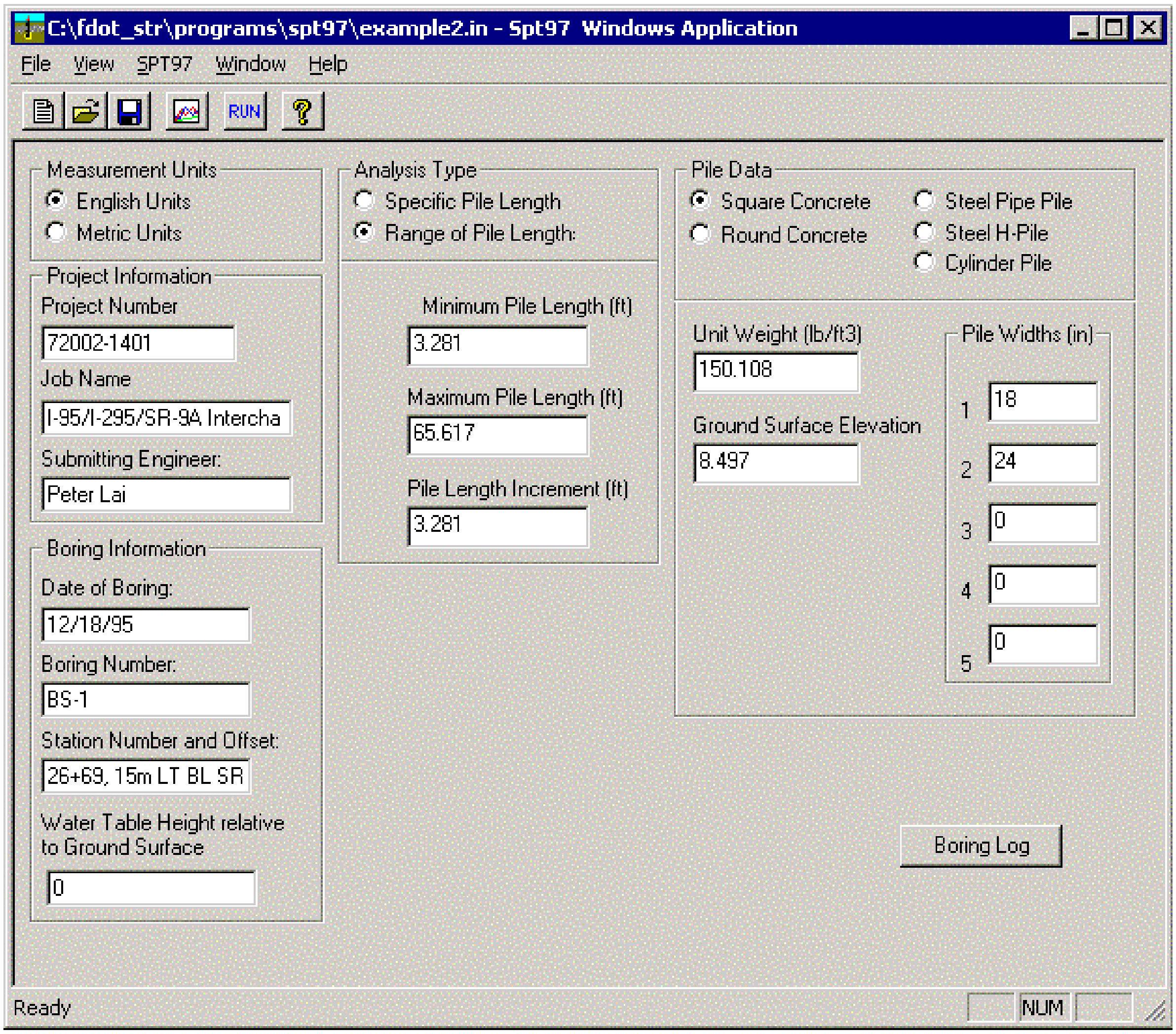
B. Pile Tip Elevations for Vertical Load

B1. Pile Capacities as per SPT97

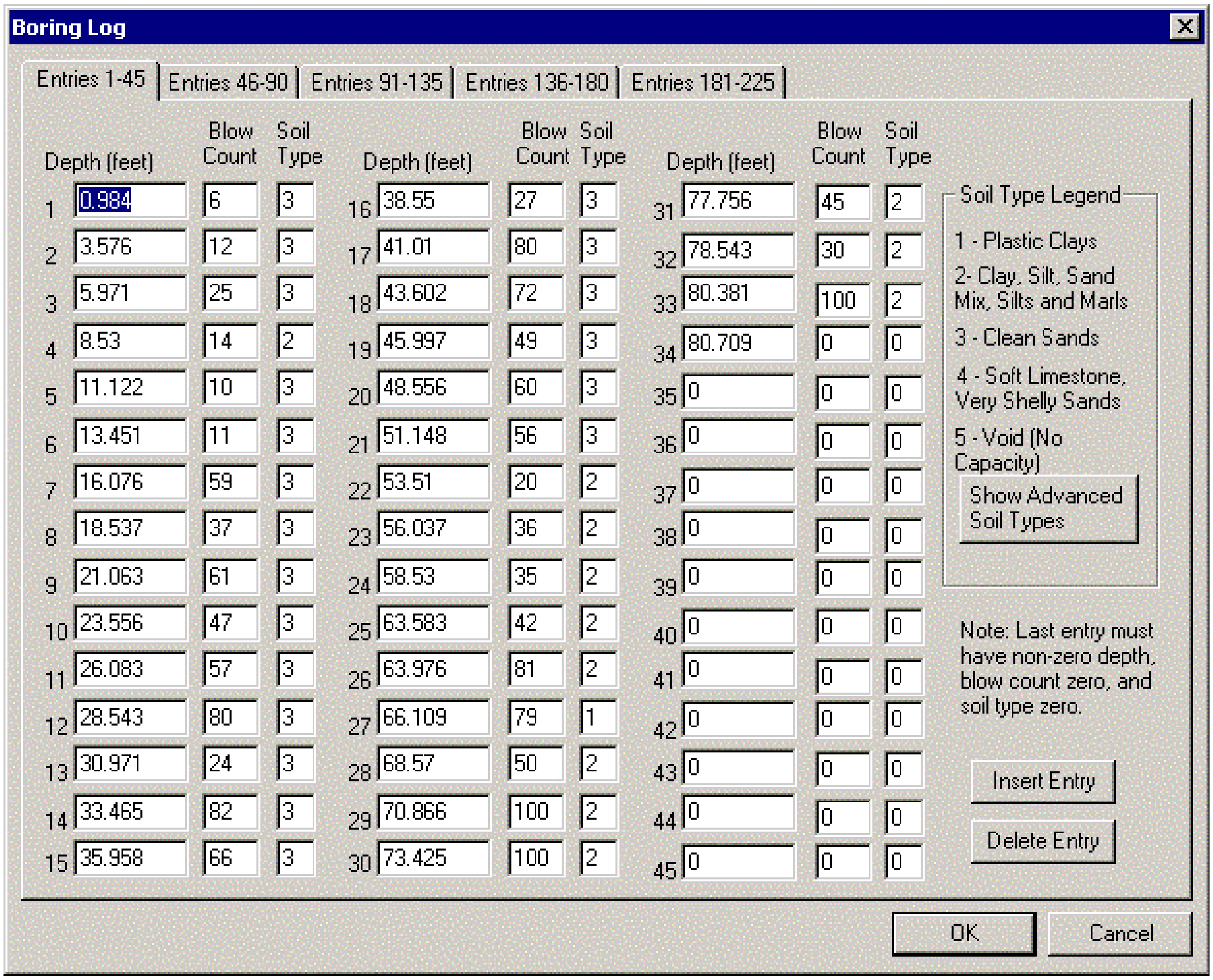
The Static Pile Capacity Analysis Program, SPT97 NT v1.5 dated 6/2/00, was utilized to determine the pile capacity. Using boring data, the program can analyze concrete piles, H-piles, pipe piles, and cylinder piles. It is available at the following FDOT website:

http://www11.myflorida.com/structures/programs/spt97setup.exe

For this design example, the boring data is based on Example2 in the program, which is part of the install package.



The following picture shows the boring log entries in Example2.in.



Boring Log

Entries 1-45 | Entries 46-90 | Entries 91-135 | Entries 136-180 | Entries 181-225

	Depth (feet)	Blow Count	Soil Type		Depth (feet)	Blow Count	Soil Type		Depth (feet)	Blow Count	Soil Type
1	0.984	6	3	16	38.55	27	3	31	77.756	45	2
2	3.576	12	3	17	41.01	80	3	32	78.543	30	2
3	5.971	25	3	18	43.602	72	3	33	80.381	100	2
4	8.53	14	2	19	45.997	49	3	34	80.709	0	0
5	11.122	10	3	20	48.556	60	3	35	0	0	0
6	13.451	11	3	21	51.148	56	3	36	0	0	0
7	16.076	59	3	22	53.51	20	2	37	0	0	0
8	18.537	37	3	23	56.037	36	2	38	0	0	0
9	21.063	61	3	24	58.53	35	2	39	0	0	0
10	23.556	47	3	25	63.583	42	2	40	0	0	0
11	26.083	57	3	26	63.976	81	2	41	0	0	0
12	28.543	80	3	27	66.109	79	1	42	0	0	0
13	30.971	24	3	28	68.57	50	2	43	0	0	0
14	33.465	82	3	29	70.866	100	2	44	0	0	0
15	35.958	66	3	30	73.425	100	2	45	0	0	0

Soil Type Legend

1 - Plastic Clays

2- Clay, Silt, Sand Mix, Silts and Marls

3 - Clean Sands

4 - Soft Limestone, Very Shelly Sands

5 - Void (No Capacity)

Show Advanced Soil Types

Note: Last entry must have non-zero depth, blow count zero, and soil type zero.

Insert Entry

Delete Entry

OK

Cancel

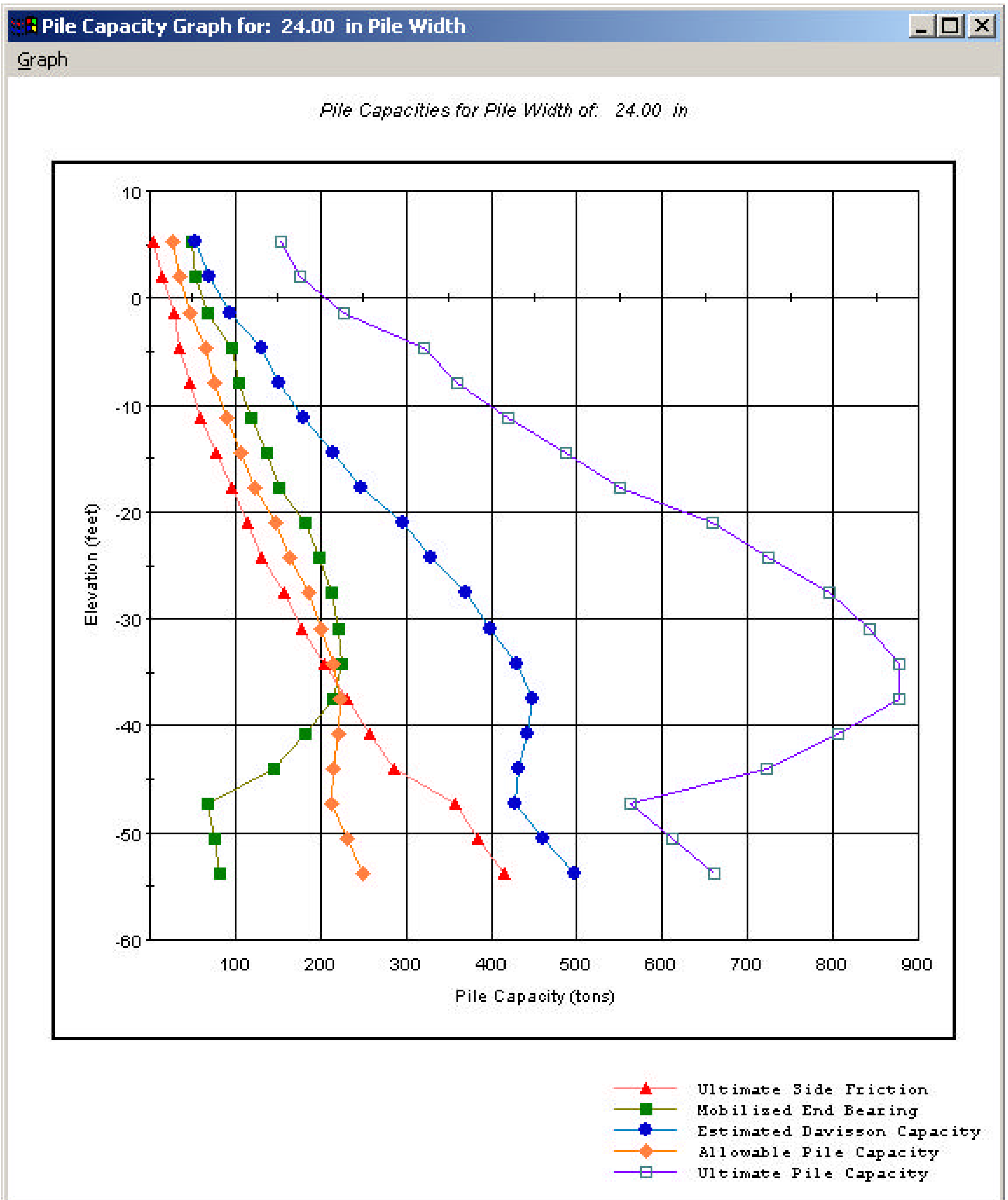
Recall that the ultimate bearing capacity, UBC, is given by....................................

$$\text{UBC} = \frac{\text{Factored Design Load} + \text{Net Scour} + \text{Downdrag}}{\phi}$$

In this design example, net scour and downdrag are zero, so the UBC is............

$$\text{UBC} = 343.8\ \text{Ton}$$

The program was executed, and the output can be summarized as follows:



D. PILE CAPACITY VS. PENETRATION

TEST PILE LENGTH (FT)	PILE TIP ELEV (FT)	ULTIMATE SIDE FRICTION (TONS)	MOBILIZED END BEARING (TONS)	ESTIMATED DAVISSON CAPACITY (TONS)	ALLOWABLE PILE CAPACITY (TONS)	ULTIMATE PILE CAPACITY (TONS)
32.8	-24.3	131.43	197.43	328.86	164.43	723.73
36.1	-27.6	157.91	212.88	370.79	185.39	796.55

A lateral load analysis may require the pile tip elevations to be driven deeper for stability purposes. This file only evaluates the vertical load requirements based on the boring capacity curves.

Calculate the pile length required.....................

$$pile_{length} := (UBC - 328.96 \cdot Ton) \cdot \left(\frac{36.1 \cdot ft - 32.8 \cdot ft}{370.79 \cdot Ton - 328.86 \cdot Ton} \right) ... + 32.8 \cdot ft$$

$pile_{length} = 34\,ft$

Calculate the pile tip elevation required.............................

$$pile_{tip} := (UBC - 328.96 \cdot Ton) \cdot \left(\frac{-27.6 \cdot ft - -24.3 \cdot ft}{370.79 \cdot Ton - 328.86 \cdot Ton} \right) + -24.3 \cdot ft$$

$pile_{tip} = -25.5\,ft$

...based on the Estimated Davisson pile capacity curve given above, the pile lengths for vertical load will require a specified Tip Elevation = -25.5 ft. Therefore, the pile in the ground length is 34 ft.

All piles at the Intermediate Bent will be specified the same.

Defined Units