Bridge Substructure Design and Service Life Extension

Date: 7 March 2022
Session 45 – Bridges 04

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Training Program

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Types of Piers

- Pier reference: generally include bent caps and piers both.
- Single pier, multiple pier
- Solid wall, group of piles.
- Integral pier.
- Pile foundation: footings, piles or a combination.
- Don't forget riprap if needed for scour control.

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Pier Behavior

- Pier rigidly connected to foundation: cantilever action.
- Rigid bearings supported: wider foundation needed to resist overturning.
- Expansion bearings: narrower foundations.
- Any bearing friction should be considered.
- To consider: braking force, wind loads, water loads, seismic loads, debris loads, centrifugal loads, impact…
- Consider load combinations.
- Consider slenderness and end conditions.

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Design Considerations

- Overturning
- Sliding
- Soil bearing
- Differential settlement
- Structural failure
- Scour effect
- Software options
- Longitudinal vs. transverse load
- Critical loading for bent cap, pier and foundation
- Aesthetics

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Sample Piers

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Sample Piers

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Sample Piers

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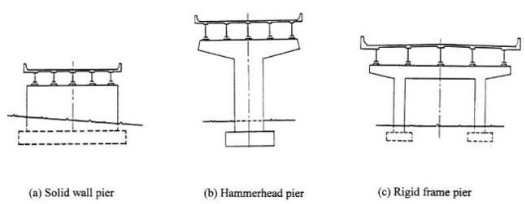
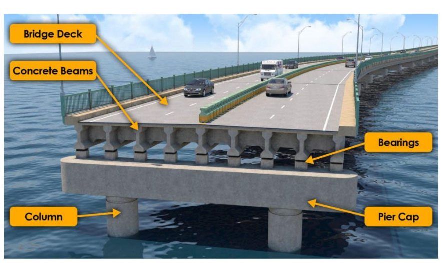
Bent Cap Design





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Bent Cap Design

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Sources of Bridge Design Examples and References

- PCA (www.cement.org)
  LRFD Design of Cast-in-Place Concrete Bridges
  AASHTO LRFD Strut & Tie Model Design Examples
- PCI (www.pci.org)
  Bridge Design Manual
- AASHTO (bridges.transportation.org)
- FHWA National Highway Institute (www.fhwa.dot.gov/bridge)
  Comprehensive LRFD Bridge Design Examples (2)

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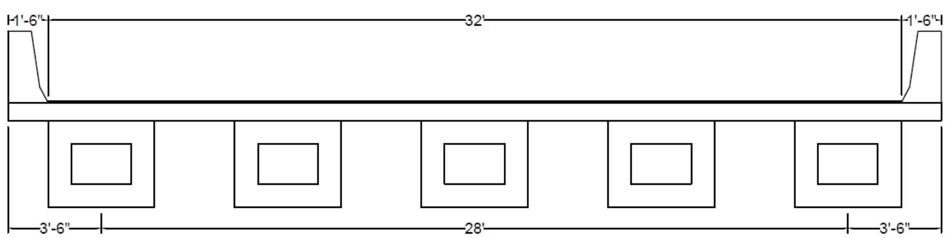
Bent Cap Design Example

For the bridge superstructure shown, design the Bent Cap. You may use CAP18 software or other similar avenues.



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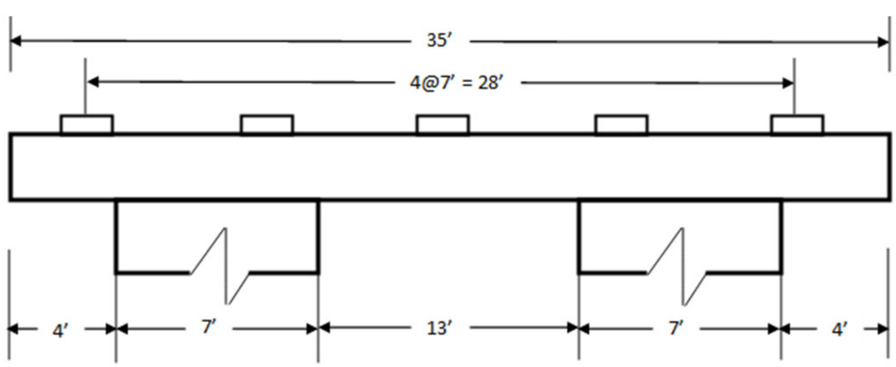
Bent Profile



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Example

Cap Dimensions

Cap Width - 3 ft. 6 in.; Cap Depth - 3 ft. 6 in., Cover - 2.5 in.

Cap Concrete

f'_c = 6 ksi; w_c = 0.150 kcf

Column Width - 3 ft. 6 in.

Girder span = 89 ft. with two spans

Girder section = 812.5 in^2

Deck thickness = 8 in.; w_c = 0.145 kcf

Barrier = 0.3 kip/ft

Wearing surface = 0.5 in. with 0.145 pcf

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Example

Dead Load Effects
(point reaction loads from girders)

Exterior and Interior Girders:

$$DC = \underbrace{\frac{812.5*0.150*89}{144}}_{\text{Girder DL}} + \underbrace{\frac{8*35*0.145*89}{12*5}}_{\text{Deck DL}} + \underbrace{\frac{2*0.3*89}{5}}_{\text{Barrier DL}} = 146 \text{ kip/girder}$$

$$DW = \frac{0.5*35*0.145*89}{12*5} = 3.76 \text{ kip/girder}$$

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Example

Live Load Effects:

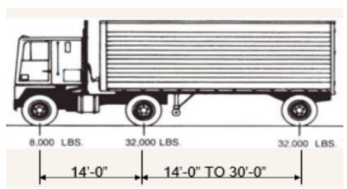
Limit States

Strength I	Service I
LL+IM: 1.75	LL+IM: 1
DC: 1.25	DC & DW: 1
DW: 1.5	

- Multiple presence factor

Multiple Presence Factors	
Number of Lanes Loaded	Multiple Presence Factor, m
1	1.20
2	1.00
3	0.85
>3	0.65

AASHTO 3.6.1, Table 3.6.1.1.2-1, page 321



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Example

Application of Live Loads

Design Lanes –10 ft. wide

Lane loading

-10 ft. wide

-Located within design lane

Truck loading

-6 ft. transverse wheel spacing

-Located within design lane

-Wheel located not less than 2 ft. from design lane edge

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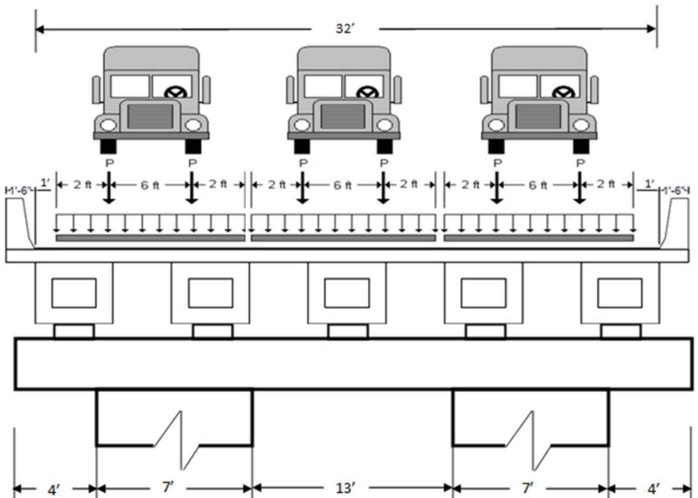
Example



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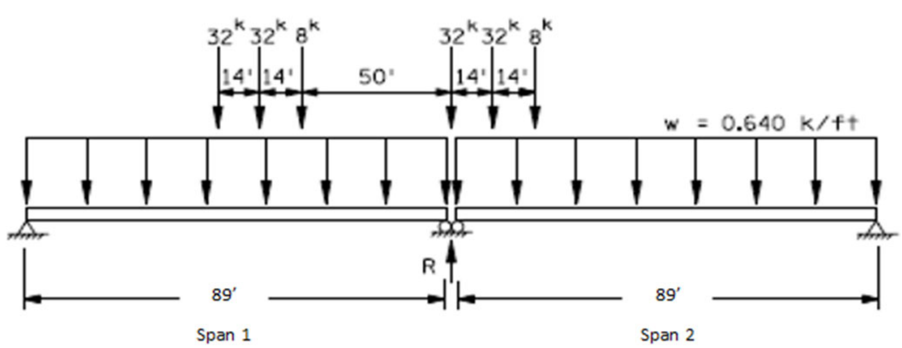
- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

AASHTO 3.6.1.3.1



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Example

Girder Reactions from superstructure analysis

$R_{truck} = \left(\frac{32*11+32*25+8*39+32*89+32*75+8*61}{89}\right) * 1.2 = 97.1$ k

$R_{lane} = \frac{2*0.64*89}{2} * 1.2 = 68.35$ kip

Wheel Loads:

$R_{truck} = 97.1$ kip

$P_{wheel} = R_{truck}/2 \times (1+IM) \times 0.90 = 58$ kip

IM = 0.33

0.9 reduction for interior piers (3.6.1.3.1)

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Example

Equivalent uniform lane loads:

$R_{lane} = 68.35$ kip

$W_{lane} = R_{lane}/10 \times 0.9 = 6.15$ kip/ft

Weight of bent cap = $\frac{42*42*0.15}{144*2}$ = 0.92 kip/pier

Braking Load Effects – BR: Greater of (3.6.4)

25% of axle weights (truck or tandem)

0.25 * (32 + 32 + 8) = 18 kip (truck) [Controls]

0.25 * (25 + 25) = 12.5 kip(tandem)

3.6.4—Braking Force: *BR*

The braking force shall be taken as the greater of:

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

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 6.0 ft above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

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Example

5% of (axle weights of truck + lane loading) =

0.05 * [(32 + 32 + 8) + (0.64 * 178)] = 9.3 kip

5% of (axle weights of tandem + lane loading)

0.05 * [(25 + 25) + (0.64 * 178)] = 8.2 kip

Braking Force per Bearing

BR = 18/ 5 = 3.6 kip/bearing

Example

Wind Load Effects – WS

Calculate Horizontal Wind Pressure (3.8.1, page 3-40):

Use base wind velocity [Assuming bridge height < 30 ft.]

Assumed V_b= 115 mph

Determine base wind pressure (ignoring various factors)

–P_b= 0.00256 $(115)^2$= 0.034 ksf

Example

Calculate Wind Pressure on Superstructure:

Superstructure Depth

H_{super}= Barrier Height + Deck thickness + Haunch thickness + Girder Height

= 3.5 + 0.667 + 0.25 [Assumed] + 3.25 =7.67 ft.

WS = 0.034 ksf x 7.67 ft. = 0.26 kip/ft. < 0.3 kip/ft

Use WS = 0.3 k/ft. or 0.039 ksf

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Example

Calculate Wind Load Force Effects – WS

Use coefficients from Table 3.8.1.2.3a -1 (page 3-49)

Example for 60^0 wind angle:

–WS_{trans}= A_{trans} x 0.34 x 0.039 = 682 x 0.34 x 0.039 = 9.08 kip

–WS_{long} = A_{long} x 0.38 x 0.039 = 1365 x 0.38 x 0.039 = 20.23 kip

Table 3.8.1.2.3a-1—Skew Coefficients for Various Skew Angles of Attack

	Trusses, Columns, and Arches		Girders	
Skew Angle (degree)	Transverse Skew Coefficient	Longitudinal Skew Coefficient	Transverse Skew Coefficient	Longitudinal Skew Coefficient
0	1.000	0.000	1.000	0.000
15	0.933	0.160	0.880	0.120
30	0.867	0.373	0.820	0.240
45	0.627	0.547	0.660	0.320
60	0.320	0.667	0.340	0.380

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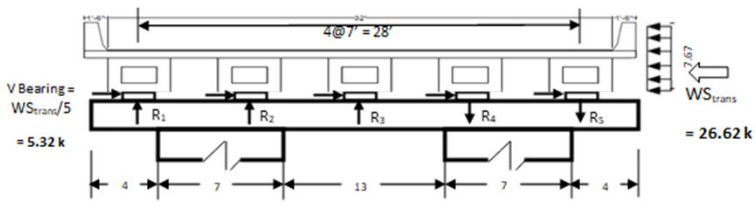
Example

Pier Design WS Force Effects		
Wind Attack Angle	Bridge Transverse Axis	Bridge Longitudinal Axis
Degree	kip	kip
0	26.62	0
15	23.43	6.39
30	21.83	12.78
45	17.57	17.04
60	9.08	20.23



Example

Calculate Bearing Reactions for Wind Force Effects -WS

0^0 wind attack angle

$M_{trans} = WS_{trans} \times (H_{super}/2) = 26.62 \text{ k} \times 7.67/2 = 102$ kip-ft.

$I_{box\ girders} = I_g + Ad^2 = \Sigma d^2 = 2(14')^2 + 2(7')^2 = 490 \text{ ft}^2$

$R_1 = R_5 = (102 \text{ k-ft} \times 14')/490 \text{ ft}^2 = 2.92$ kip

$R_2 = R_4 = (102 \text{ k-ft} \times 7')/490 \text{ ft}^2 = 1.46$ kip

Example

Vertical Wind Load Effects

Calculate Bearing Reactions for Vertical Wind Effects -WS

Resultant wind loading at Pier

From 3.8.2,

A vertical upward wind force of

0.02 ksf times the width of the deck

= 0.02 ksf * 35 ft. = 0.7 kip/ft.

Eccentricity: ¼ x width = 8.75 ft.

3.8.2—Vertical Wind Load

The effect of forces tending to overturn structures, unless otherwise determined in Article 3.8.3, shall be calculated as a vertical upward wind load equal to:

- 0.020 ksf for Strength III load combination and
- 0.010 ksf for Service IV load combination

times the width of the deck, including parapets and sidewalks, shall be applied as a line load. This force shall be applied only when the direction of horizontal wind is taken to be perpendicular to the longitudinal axis of the bridge. This line load shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads specified in Article 3.8.1.

Example

-Vertical force = 0.7 kip/ft x tributary length = 0.7 x 89 = 62.3 kip

-Moment = Vert. force x eccentricity = 62.3 x 8.75’ = 545 k-ft.

-Reaction = P/A ± (M x C)/I

R1 = -62.3 k / 5 + (545 k-ft x 14’)/ 490 ft.^{2} = 3.1 kip

R2 = -62.3 k / 5 + (545 k-ft x 7’)/490 ft.^{2} = -4.67 kip

R3 = -62.3 k / 5 = -12.46 k

R4 = -62.3 k / 5 - (545 k-ft x 7’)/490 ft.^{2} = -20.2 kip

R5 = -62.3 k / 5 - (545 k-ft x 14’)/490 ft.^{2} = -28.0 kip

Example

Calculate Horizontal Wind Force on cap and pier (3.8.1.2.3)

Base Wind Pressure P_b= 0.05 ksf

Calculate Substructure Tributary Area

Pier Cap Tributary Area

–Cap Front Area = 3.5’ x 35’ = 122 ft^2

–Cap End Area = 3.5’ x 3.5’ = 12.2 ft^2

Pier Column Tributary Area [Assuming pier height of 15 ft.]

–Column Front Area = 15’ x 7’ = 105 $ft.^2$

–Column End Area = 15’ x 3.5’ = 52.5 $ft.^2$

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Example

Attack Angle, deg.	$AP_{cap,}$ ft^2	$Ap_{col,}$ ft^2	Total Wind Load, kip	$Ws_{tran,}$ kip	$Ws_{long,}$ kip
0	12.25	60	3.61	3.6	0
15	43.54	85.1	6.43	6.2	1.67
30	71.86	104.5	8.82	7.6	4.4
45	95.3	116.7	10.6	7.5	7.5

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Example

Wind on Live Load (3.8.1.3):

Transverse Tributary Length = ½ (Span 1 + Span 2) = 89'

Longitudinal Tributary Length = Span 1 + Span 2 = 178'

Calculate Wind Load Force Effects –WL

Table 3.8.1.3-1—Wind Load Components on Live Load

Skew Angle (degrees)	Transverse Component (klf)	Longitudinal Component (klf)
0	0.100	0.000
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

30-degree wind attack angle

$WL_{trans} = L_{trans}$ x 0.082 k/ft. = 89 ft x 0.082 = 7.3 k

$WL_{long} = L_{long}$ x 0.024 k/ft. = 178 ft x 0.024 = 4.3 k

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Example

Calculate Bearing Reactions for Wind on LL –WL

30 deg. wind attack angle

$M_{trans} = WL_{trans}$ x Height = 7.3 x (7.67'+ 2.5') = 74 kip-ft

$R_1 = R_5$ = (74.24 x 14')/490 = 2.12 kip

$R_2 = R_4$ = (74.24 x 7')/490 = 1.06 kip

Temperature Force Effects -TU

Assume a 20-kip total force: TU = 20k

TU per bearing = TU/5 bearings = 4.0 kip/bearing (3.12.2)

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CAP 18 Output and Load Combinations

	Max + M	Max - M	Max + Shear	Max - Shear
Dead Load	546 kip-ft	-1251 kip-ft	313 kip	-467 kip
Service	2281 kip-ft	-1912 kip-ft	712 kip	-478 kip
Ultimate	3380 kip-ft	-2727 kip-ft.	1016 kip	-682 kip

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.40	1.40	0.40	0.40	0.30	0.30
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

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Example

Governing Load Combination:

Strength I

DC * 1.25 + DW * 1.5 + (LL+IM) * 1.75 + BR * 1.75+ TU * 1.2 + EV * 1.35

Flexural Reinforcement:

Assuming Maximum Moments:

M_{dl} = 1251 kip-ft.

M_u = 3380 kip-ft.

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Example

Try 14-#11 bars Top and Bottom of bent cap:

A_s = 21.84 in^2 $d_{stirrup}$ = 0.625 in

d_s = cap depth – cover – $d_{stirrup}$ - d_{bar} /2 = 38.4 in

b = cap width = 42 in; c = 4.11 in; a = c * β_1 = 3.5 in

M_n = 3977 kip-ft;

M_r = ϕ *M_n= 0.9*3977 = 3579 kip-ft > M_u [OK]

Example

Skin Reinforcement (5.7.3.4.2)

No. of # 5 rebars = 5; Bar Area = 0.31 in^2

A_{sk} = 1.55 in^2

h_{skin} = 34.97 in = 2.91 ft

$A_{sk(reqd)}$ = 0.286 in^2 < A_{sk} [OK]

Actual Spacing of Skin Reinforcement

s_{sk} = 5.83 in

Required Spacing of Skin Reinforcement

S_{sk_req} = 6.36 in > s_{sk} [OK]

Use 5 #5 on each side of bent

Example: Bridge Pier Design

The columns are fixed at the top and bottom in both directions.

Design Data

Concrete strength $f_c' = 3.5\ ksi$
Yield strength of reinforcement $f_y = 60\ ksi$

Factored loads for the Extreme Event I load combination for each column are shown below.

$$M_u = 4050\ kip - ft$$

$$V_u = 600\ kip$$

$$P_u = 2700\ kip\ (compression)$$

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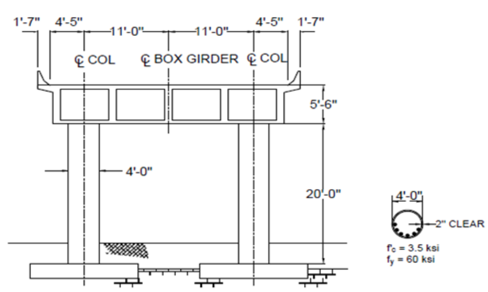
Example: Bridge Pier Design

(a) For the Extreme Event I load combination, determine the vertical reinforcing required for the column. Neglect any slenderness effect.
(b) For the Extreme Event I load combination, design the required spiral reinforcing.



TRANSVERSE SECTION OF PIER

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Example: Bridge Pier Design

Design for moment and axial force

- Column requirement

(5.11.4.1, 5-193)

$h/D_c =$
$ratio\ of\ clear\ height\ to\ maximum\ plan\ dimension\ of\ column$
$= 20/4 = 5 > 2.5$

The column qualifies to be designed as a column and not as a pier.

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Example: Bridge Pier Design

Vertical reinforcement, flexural strength

Design for $M_u = 4050\ kip - ft$ and $P_{u\ max} = 2700\ kip$

$$\phi = 0.9$$

$$M_n = nominal\ moment = \frac{M_u}{\phi} = \frac{4050}{0.9} = 4500\ kip - ft$$

5.11.4.1.2—Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4. The column shall be investigated for both directional combinations of force effects specified in Article 3.10.8, at the extreme event limit state. The resistance factors of Article 5.5.4.2 shall be replaced for columns with either spiral or tie reinforcement by the value of 0.9.

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$$P_n = nominal\ axial\ force = \frac{P_u}{\phi} = \frac{2700}{0.9} = 3000\ kip$$

From the interaction diagram for $M_n = 4500\ kip - ft$ and $P_u = 3000\ kip,$

$\rho = 0.025$

Limits of vertical reinforcement

5.11.4.1.1—Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.04 times the gross cross-section area, A_g.

$0.01 < \rho = 0.025 < 0.04\ OK$

$A_s = 0.025 \times \pi(48)^2/4 = 45.24\ in^2$

Using 29 #36 bars with $A_s = 29 \times 1.56 = 45.24\ in^2 \approx 29187\ mm^2\ will\ staisfy\ the\ requirement$

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Example: Bridge Pier Design

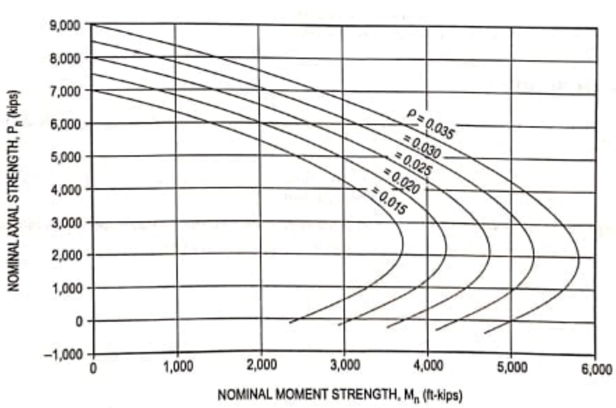

FIGURE 901B

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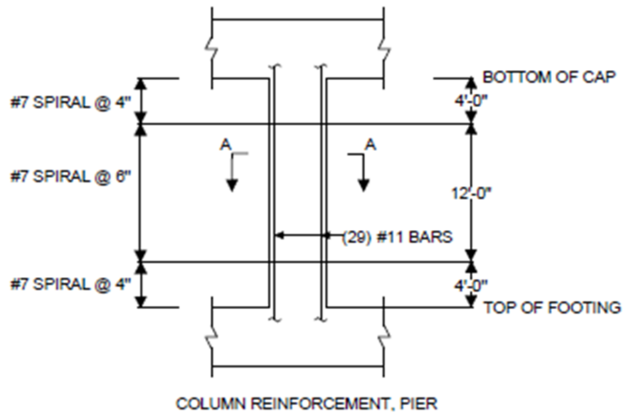
Example: Bridge Pier Design



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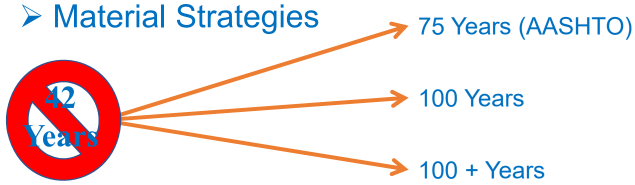
Can We Achieve 100+ year Service Life?

Extend Service Life Through:

- Structural Strategies
- Material Strategies



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Curing

- Continuous Wet Curing Preferred
  - Immediately After Finishing; 7 Days or More
- Types of Wet Curing
  - Ponding
  - Fogging (Initial Curing)
  - Sprinkler/Soaker Hoses
- Often Used With Burlap/Mats or Plastic
- Enhances Performance
  - Concrete Gains Strength Before Drying
  - Reduction in Shrinkage Cracking

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Curing

Fogging

Sprinkler Saturating Burlap

Photos: *Design and Control of Concrete Mixtures*, by Kosmatka, Kerkhoff, and Panarese, 14th Edition, CD100, Portland Cement Association, 2009.

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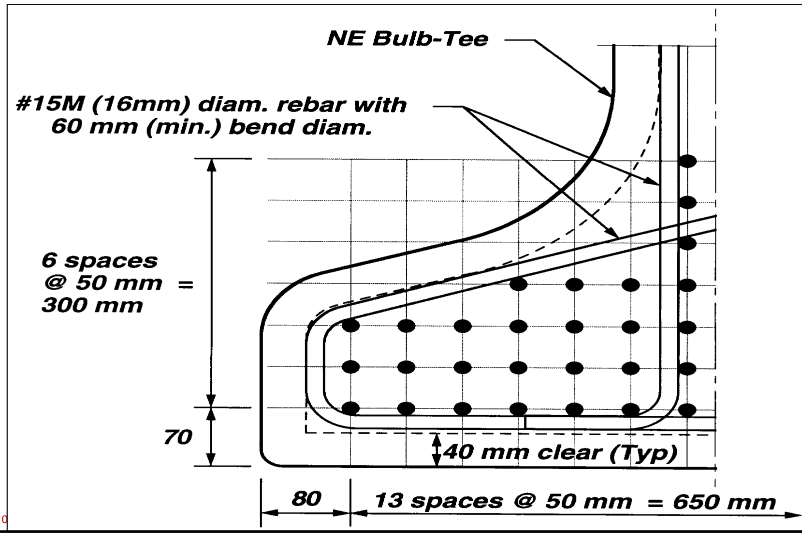
Cover



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Minimum Concrete Cover

AASHTO LRFD Article 5.10.1

SITUATION	COVER (IN)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments - General - Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0

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Minimum Concrete Cover Issues

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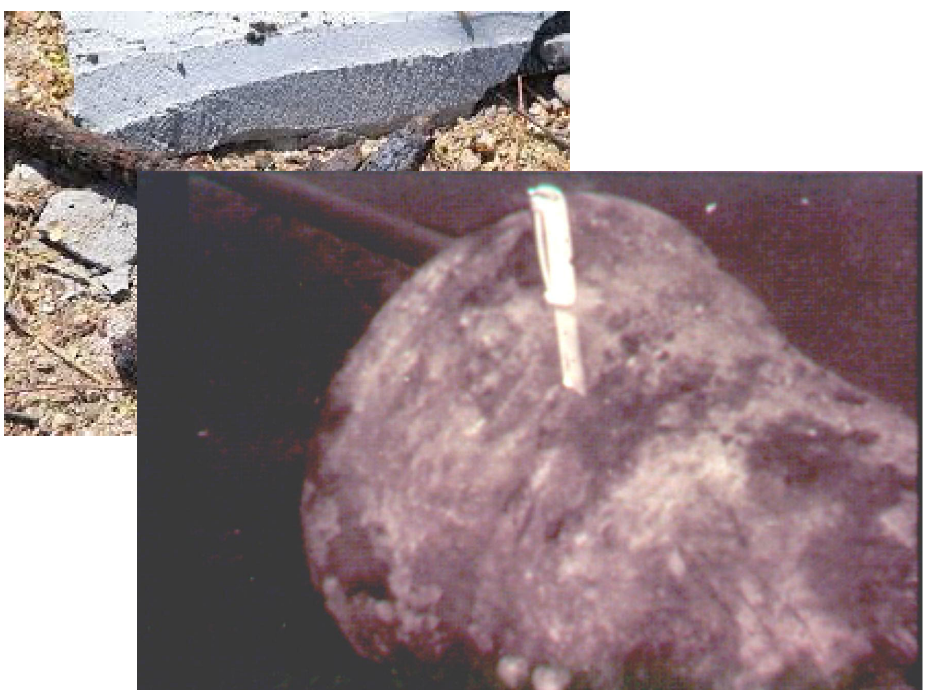

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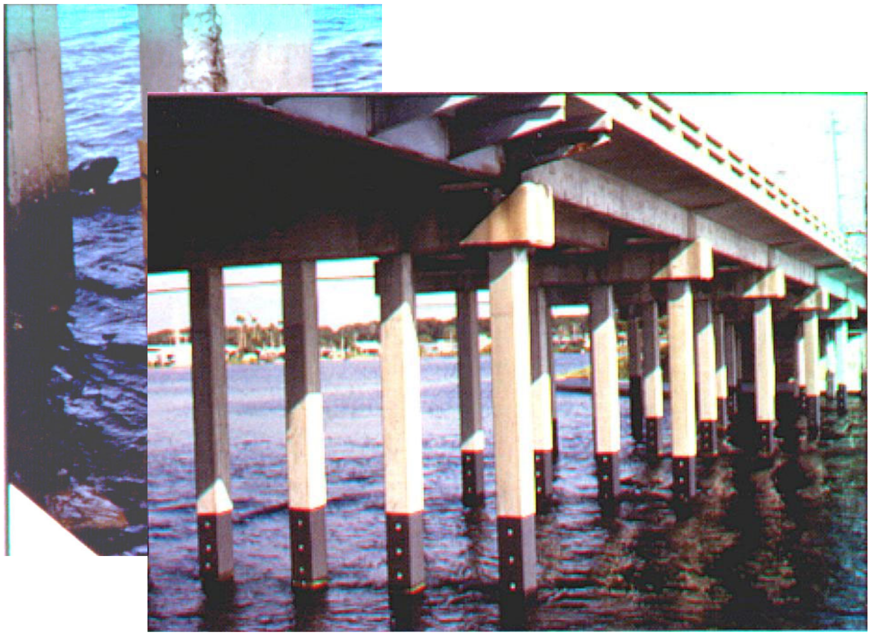

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Compressive Strength

Min. f'_c = 2.4 ksi for Structural Applications

Min. f'_c = 4.0 ksi for Decks

Min. f'_c = 4.0 ksi for Prestressed Concrete Members

5.4.2—Normal Weight and Lightweight Concrete

5.4.2.1—Compressive Strength

For each component, the compressive strength of concrete for use in design, f'_c, or the class of concrete shall be shown in the contract documents.

Design concrete compressive strengths above 10.0 ksi for normal weight concrete shall be used only when allowed by specific Articles or when physical tests are made to establish the relationships between the concrete strength and other properties. Concrete with compressive strengths used in design below 2.4 ksi should not be used in structural applications.

The design concrete compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

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Compressive Strength

AASHTO LRFD, 5.4.2.1:

Concrete strength > 10 ksi used only when physical tests are made to establish relationships between concrete strength and other properties, or as allowed by other articles in AASHTO LRFD.

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Compressive Strength

AASHTO LRFD:

Provisions valid for $f'_c \leq 15.0$ ksi for calculating:

- Shrinkage and Creep (5.4.2.3.1, p. 5-17)
- Modulus of Elasticity of Concrete (5.4.2.4, p. 5-19)
- Modulus of Rupture of Concrete (5.4.2.6, p. 5-20)
- Prestress Losses (5.9.3.1, p. 5-127)

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Control of Cracking by Distribution of Reinforcement

- Provide for maximum rebar spacings

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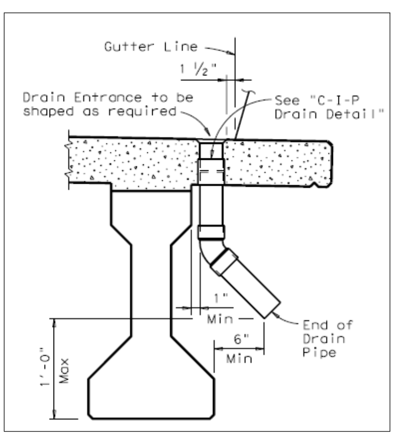
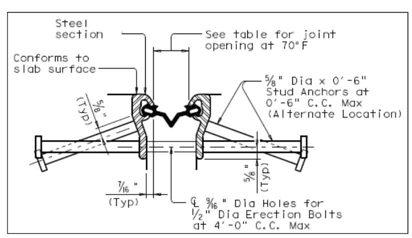
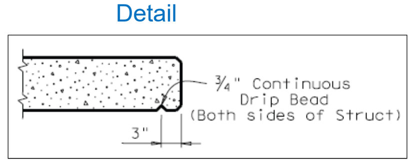
Detailing



Drain Detail



Sealed Joint Detail



Drip Bead Detail

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High Performance Concrete

Concrete That Attains Mechanical, Durability, or Constructability Properties Exceeding Those of Normal Concrete

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High Performance Concrete (HPC)

Increased Durability and Normal Strength

or

Increased Durability and High Strength

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Designing HPC Mix

- Strategic Selection of Quantities
- Low Water–to–Cementitious Materials Ratio
- Key Additions to Basic Concrete Mix
  - Supplementary Cementitious Materials
  - Chemical Admixtures

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Performance Classification for HPC for Bridges

Introduced By FHWA In February, 1996
and Updated in August, 2006

Performance Characteristic Grades 1 – 3

+ Grade 1 = Good Performance
+ Grade 3 = Best Performance

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Freeze/Thaw Durability	70% ≤ F/T < 80%	80% ≤ F/T < 90%	90% ≤ F/T
Scaling Resistance	3.0 ≥ SR > 2.0	2.0 ≥ SR > 1.0	1.0 ≥ SR > 0.0
Abrasion Resistance	2.0 ≥ AR ≥ 1.0 mm	1.0 ≥ AR ≥ 0.5 mm	0.5 mm > AR
Chloride Penetration (coulombs)	2500 ≥ CP > 1500	1500 ≥ CP > 500	500 ≥ CP

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Sulfate Resistance	SR ≤ 0.10 @ 6 mo.	SR ≤ 0.10 @ 1 yr.	SR ≤ 0.10 @ 1.5 yr.
Alkali-Silica Reactivity	0.20 ≥ ASR > 0.15	0.15 ≥ ASR > 0.10	0.10 ≥ ASR
Workability—Slump	SL > 7½ in		
Workability—Slump Flow (SCC)	SF < 20 in	20 ≤ SF ≤ 24 in	24 in < SF

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Strength (ksi)	$8 \leq f'_c < 10$	$10 \leq f'_c < 14$	$14 \leq f'_c$
Elasticity ($x\ 10^6$ psi)	$5 \leq E_c < 6$	$6 \leq E_c < 7$	$7 \leq E_c$
Shrinkage (microstrain)	$800 > S \geq 600$	$600 > S \geq 400$	$400 > S$
Creep (microstrain/psi)	$0.52 \geq C > 0.38$	$0.38 \geq C > 0.21$	$0.21 \geq C$

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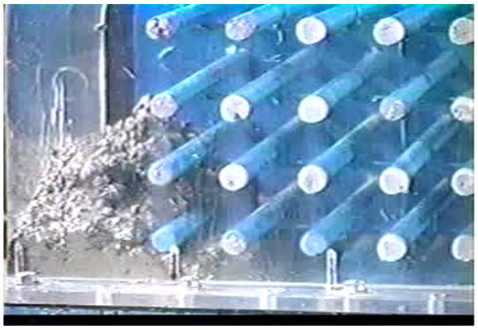
Self Consolidating Concrete



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I-35W St. Anthony Falls Bridge

Photo Courtesy of Minnesota DOT

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Ultra-High Performance Concrete

- Very High Strength Concrete ≥ 20 ksi
- Research by FHWA—Turner Fairbanks
- Increased Durability

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Corrosion-Resistant Reinforcement

- Epoxy-Coated
- Hot-Dip Galvanized
- Dual-Coated
- Stainless Steel
- Low-Carbon, Chromium
- Stainless-Clad
- Fiber-Reinforced Polymer (FRP) Composite

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Corrosion-Resistant Reinforcement

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Epoxy-Coated Reinforcing Steel

Photo: *Design and Control of Concrete Mixtures*, by Kosmatka, Kerkhoff, and Panarese, 14th Edition, CD100, Portland Cement Association, 2009.

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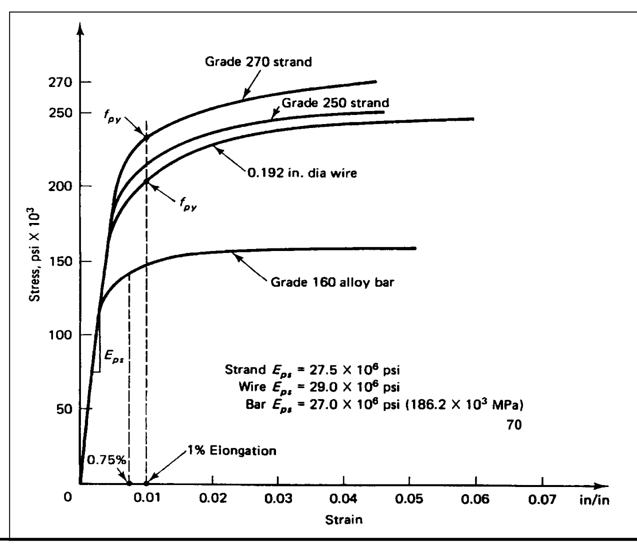
Typical Stress-Strain Curves for Prestressing Steel

Grade 270 Typical



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Plastic Duct

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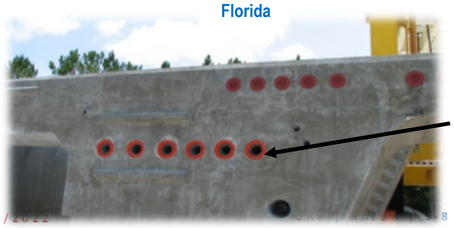
GTI Segmental Duct Coupler

I-95 / I-295 Interchange, Jacksonville, Florida

Match Cast Coupler



Bulkhead Coupler

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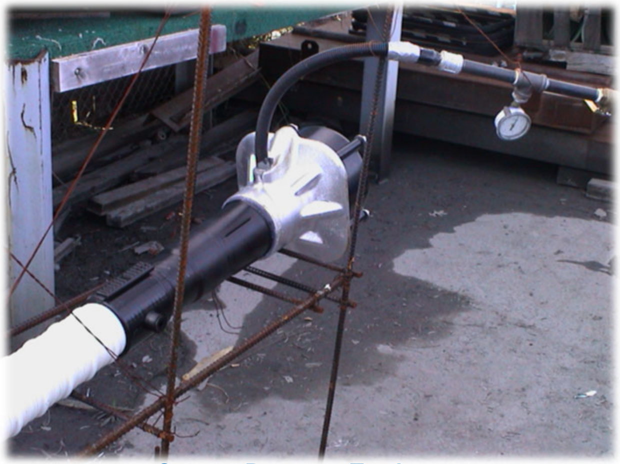
VSL Multi-Strand Anchorage System



System Pressure Testing

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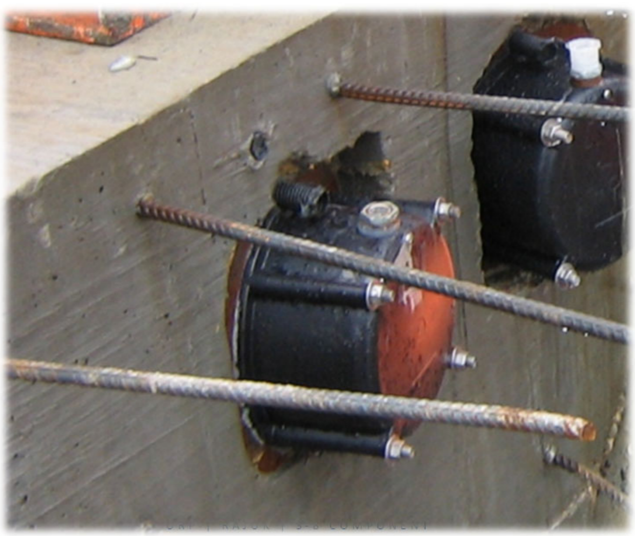
Permanent Grout Cap



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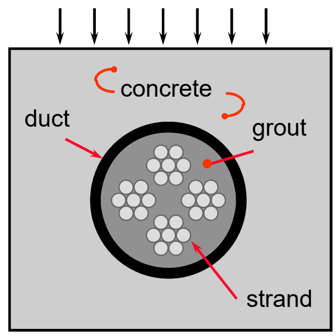
Grout for Post-Tensioning

- Extremely Durable When Properly Constructed
  - Crack Control
  - Multiple Levels of Protection
- *The Grout is the Third Level of Protection for the Strand*



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Keys to a Good Grout

- Completely Fills Duct
- Low Permeability
- Appropriate Bleed Resistance
- Careful Use of Admixtures
  - May Enhance One Property While Degrading Another

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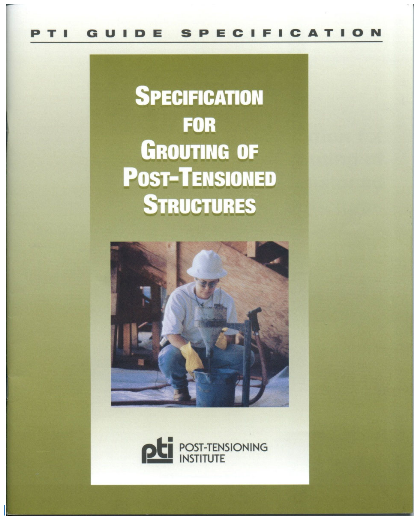
Specifications for Grouting Materials

Post-Tensioning Institute

- PTI Guide Specifications
  - Section 2: Materials



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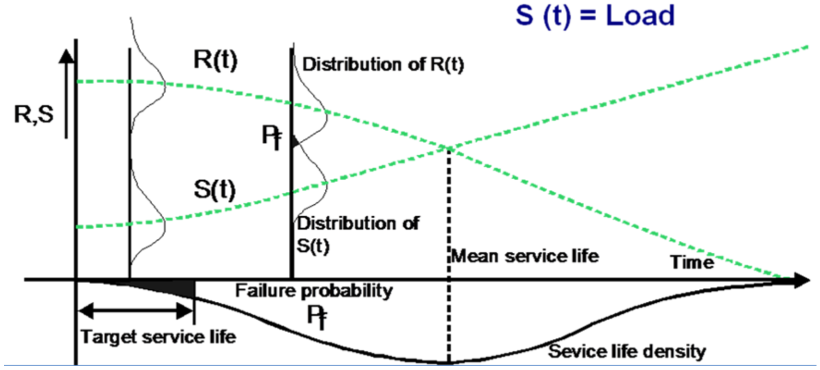
Failure Probability and Service Life

R (t) = Resistance

S (t) = Load



Failure Probability and Service Life, *fib* Bulletin 34, "Model Code for Service Life Design"

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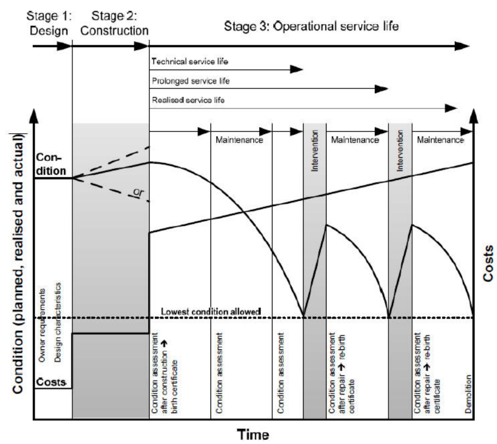
Complete Service Life



Complete Service Life, "Condition Control of Existing Structures by Performance Testing" by Gehlen, C., Dauberschmidt, C., and Nurnberger, U.

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Design for Service Life

- Strategic Highway Research Program (SHRP 2)
  - SHRP 2 R19(A), "Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components"
    - Status: Active
  - SHRP 2 R19(B), "Bridges for Service Life Beyond 100 Years: Service Limit State Design"
    - Status: Active

For More Information, Visit www.trb.org

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Conclusions

Design for 100+ Year Life Today

Structural Strategies for Durability

+

Material Strategies for Durability

=

Durable Concrete Bridges

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Points to Remember

- AASHTO LRFD Bridge Design Specifications Provides for 75 Year Design Life
- Design Life Can be Extended by Selecting Appropriate Materials, Design and Construction approaches for the Service Environment
- Full Probabilistic Design is Limited at this time due to Inaccuracies in Deterioration Modeling
- Achieving a 100+ Year Bridge Life is Possible Using Service Life Design Principles Along with Timely Intervention Strategies

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QUESTIONS?

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