Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

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Session 2 - Agenda

Session Two - Thursday, November 3, 2022			
Welcome & Introduction	11:00 AM	0:05	Ruth Lehmann
Design 1: Basic PS Design; Draping & Debonding	11:05 AM	1:10	Reid Castrodale
Design 2: Girder Sections; Camber	12:15 PM	0:15	Reid Castrodale
Accelerated Bridge Construction (ABC) Case Studies	12:30 PM	0:25	Brent Koch
Prestressed Concrete Pavement Panels	12:55 PM	0:20	Ruth Lehmann
Q&A	1:15 PM	0:15	Ruth Lehmann
PCI West Wrap-up	1:30 PM		

Reid W. Castrodale, PhD, PE

Castrodale Engineering Consultants, PC – Concord, NC

Structural engineering consultant - Prestressed concrete, LWC, and ABC

39 years bridge experience in design, research, promotion, & specifications

- **Formerly Portland Cement Assn. (PCA), Ralph Whitehead Assoc. (now STV), and Stalite**
- **Georgia/Carolinas PCI Bridge Consultant (~ 25 yrs)**
- **Managing Technical Editor of *ASPIRE*™ magazine – *now Emeritus***
- **Director of Engineering – ESCSI & Stalite – Lightweight aggregate industry**
- **Consultant on 5 NCHRP research project teams: 0.7" strand; deck girders; stainless steel strand; …**

Chair, PCI Committee on Bridges (1992-1998)

Co-Chair, *PCI Bridge Design Manual* Steering Committee (1993-2011)

NCHRP Report 517 "Extending Span Ranges of PC PS Concrete Girders"

Education

Georgia Institute of Technology, BCE

University of Texas at Austin, MS & PhD in Structural Engineering

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Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

Basic Design of Pretensioned Concrete Girders

NEVADA DOT SAFE AND CONNECTED

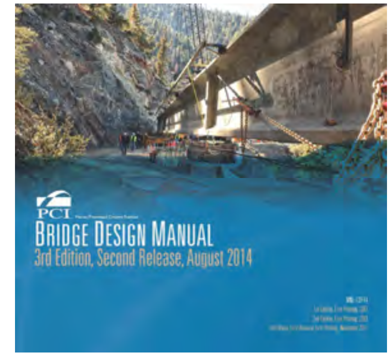
Caltrans

PCI WEST

Design of PS Girders

Based on *PCI Bridge Design Manual*

- Chapter 8 – Design Theory & Procedure
- See also design examples in Chapter 9



Initial Concrete Compressive Strength, f'_{ci}

Initial compressive strength, f'_{ci}

- Minimum: 4.5 ksi
- Preferred maximum: 6 to 7 ksi
- Higher release strengths may be required for some designs
- Typically specified in 0.1 ksi increments
  - A small difference in required strength can affect production and efficiency significantly if producer must wait to achieve the strength
- Use initial strengths that are no more than required by design
  - Don't use a fixed ratio, like $f'_{ci} = 0.8 f'_c$
    - Acceptable for initial try; reduce if possible for final design
  - Min. required f'_{ci} can be determined from concrete stresses at transfer
    - Divide concrete stress at transfer by the limiting compressive stress coefficient (0.65), then round up

Final Concrete Compressive Strength, f'_c

Final compressive strength, f'_c (28 day)

- Minimum: 6.0 ksi
- Typical maximum: 8.5 ksi
- Strengths up to 10.0 ksi are possible
- Typically specified in 0.5 ksi increments
- Specified f'_c is usually achieved well before 28 days
  - So f'_{ci} generally governs the mix design
- Must reach f'_c prior to shipping girders which can be important in some cases

Prestresser's Perspectives on Concrete Strengths

Higher release strengths can require beams to sit longer in beds

- **Increases chances of cracking**
- **Slows production and field construction schedules**
- **Ultimately forces producers to increase prices because production efficiency is reduced**
- **Some design approaches can reduce the required f'_{ci} (draping)**
- **If higher strengths are needed, so be it**
- **If not, using an unnecessarily high strength wastes time and money leading to higher construction costs for the owner**

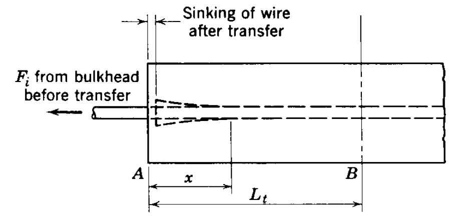
Using f'_c = 8.5 ksi is preferred

- **Also, don't use a higher f'_c than required**

Transfer of Prestress

In pretensioned members, the prestress force is transferred to the concrete entirely by bond

- **Surface adhesion**
- **Mechanical resistance from helical shape of strand**
- **"Hoyer effect" – flaring at untensioned end**

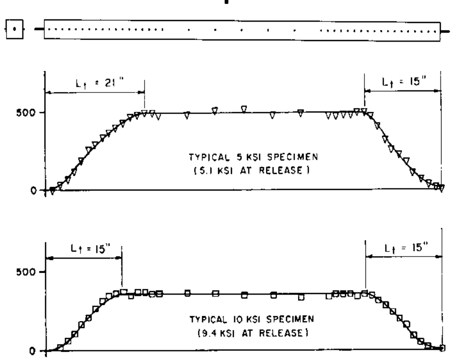


From Lin & Burns (1981)

Transfer of Prestress

Experimental data on transfer of prestress

- **4 x 4 in. concrete prism with 0.5 in. diam. pretensioned strand**



Source: Castrodale et al. 1988

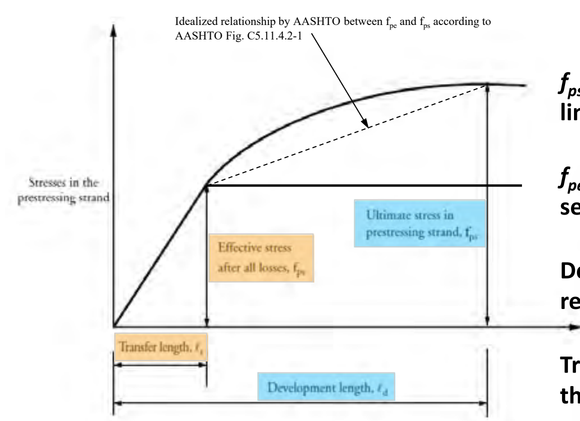
Concrete strains are measured on sides of square prism of concrete after transfer

Strains increase from zero at each end until they reach a constant level indicating the effective prestress has been reached

Distance required for that build up is the transfer length, ℓ_t

Transfer and Development of Prestress

BDM Figure 8.3-1 Strand Transfer and Development Lengths



f_{ps} – stress in strands at strength limit state (beyond ℓ_d)

f_{pe} – effective stress in strands at service limit state (beyond ℓ_t)

Development length, ℓ_d, affects resistance at strength limit state

Transfer length, ℓ_t , is included in the development length, ℓ_d

Concept of Prestress Loss

After the prestress force has been applied to a member:
- **shortening of the member from any cause will result in …**
- **shortening of the prestressing tendons (strands), which results in …**
- **a reduction in the stress in the prestressing strands.**

This reduction in the stress (and force) in prestressing strands is prestress loss

Consequences of prestress loss
- **Reduced prestress force**
- **Reduced precompression**
- **Increased tensile stresses in concrete**
- **Reduced camber**

Components of Prestress Loss in Pretensioned Girders

- **Elastic Shortening**	**Instantaneous**
- **Seating Loss**	**Instantaneous**
- **Steel Relaxation**	**Time-dependent**
- **Concrete Creep**	**Time-dependent**
- **Concrete Shrinkage**	**Time-dependent**

***Elastic regain* is the increase in strand stress (or reduction in prestress loss) that occurs when loads are applied**
- **Included in the refined loss estimation method**

Details will not be discussed today (slides hidden)
- **For more details, see Chapter 8 of *PCI BDM* or the free eLearning course T135: *Refined and Approximate Estimates of Prestress Losses***

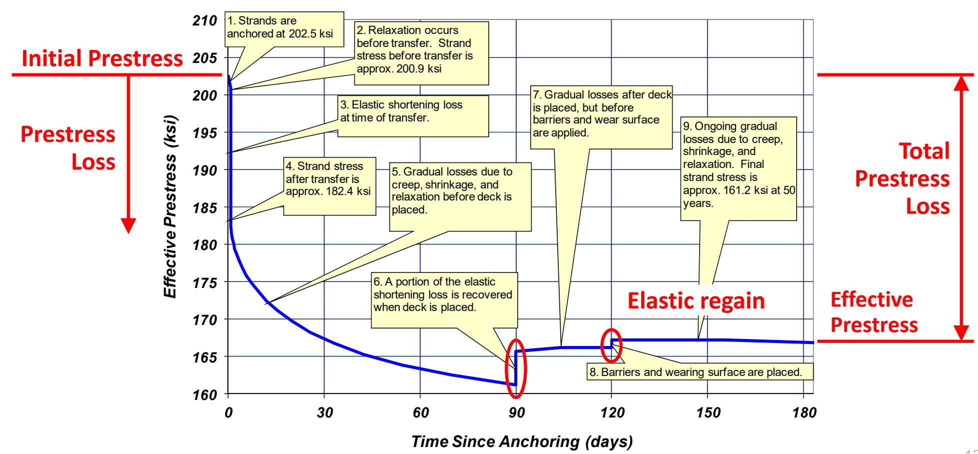
Estimating Prestress Loss

Based on NCHRP Report 496 (2003)
- **"Prestress Losses in Pretensioned High-Strength Concrete Bridge Girders"**
- **NCHRP Project 18-07**

Method provides more realistic loss estimates for higher values of f'_c

Previous method overestimated losses, resulting in
- **More strands**
- **Higher required values of f'_{ci}**
- **Greater cambers**

Prestress vs. Time



Timeframes

- Transfer of prestress
- Erection (just prior to deck placement)
- Final (long-term)

Transfer (Release)

- Elastic shortening is the immediate prestress loss that occurs at transfer
- Automatically accounted for if transformed properties are used
- Must be computed explicitly if gross section properties are used

Elastic Shortening

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

where,

E_p = 28,500 ksi

E_{ct} = 121,000 $w_c^{2.0} f'^{0.33}_{ci}$ New equation in 2015 Interim

f_{cgp} = Net stress at c.g. of the strands at transfer (ksi)

Long-Term Losses Refined Method

$$\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$$

where,

id = Transfer to deck placement – 1st stage

df = Deck placement to final time – 2nd stage

Two-stage analysis is major change introduced with new loss calculation procedure

Time-Dependent Losses

Transfer to Deck Placement

- **Prestress loss due to shrinkage of girder concrete:**

$$\Delta f_{pSR} = \varepsilon_{bid} E_p K_{id}$$

- **Prestress loss due to creep of girder concrete:**

$$\Delta f_{pCR} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b (t_d, t_i) K_{id}$$

- **Prestress loss due to relaxation of strands:**

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L} \left(\frac{f_{pt}}{f_{py}} - 0.55 \right)$$

Time-Dependent Losses

Deck Placement to Final Time

- **Prestress loss due to shrinkage of girder concrete:**

$$\Delta f_{pSD} = \varepsilon_{bdf} E_p K_{df}$$

- **Prestress loss due to creep of girder concrete:**

$$\Delta f_{pCD} = \frac{E_p}{E_{ci}} f_{cgp} \psi_b \left[(t_f, t_i) - \psi_b (t_d, t_i) \right] K_{df} + \frac{E_p}{E_c} \Delta f_{cd} \psi_b \left(t_f, t_d \right) K_{df}$$

- **Relaxation of prestressing strands:**

$$\Delta f_{pR2} = \Delta f_{pR1}$$

Time-Dependent Losses

Deck Placement to Final Time

- **Shrinkage of deck concrete:**

$$\Delta f_{pSS} = \frac{E_p}{E_c} \Delta f_{cdf} K_{df} \left[1 + 0.7 \psi_b \left(t_f, t_d \right) \right]$$

Partial Nomenclature

Where,

ε_{bid} = shrinkage of girder between transfer and deck placement (in./in.)

k_e, k_f,… = correction factors

K_{id} = transformed section age-adjusted effective modulus of elasticity factor (transfer to deck placement)

ψ_{bid} = girder creep coefficient minus ratio of strain that exists at time of deck placement to the elastic strain caused when load applied at the time of transfer

χ = aging coefficient to account for concrete stress variability with time (constant 0.7)

Flexural Design

Service Limit State – Check Stresses

- Check initial stresses after transfer
- Check final stresses after all losses (Service I & III)

Strength Limit State – Check Resistance

- Strength I & II, possibly others

Fatigue Limit State

- Fatigue in strands does not need to be checked for girder designs that satisfy limiting stresses in LRFD Art. 5.5.3.1
- Stress range cannot get high enough with uncracked concrete to cause fatigue problems

Stress Limits for Prestressing Strand

Limiting stresses in strands (for low relaxation strands)

- Prior to transfer	0.75 f_{pu}	202.5 ksi
- At service limit state	0.80 f_{py}	194 ksi – rarely governs
- Yield stress, f_{py}	0.90 f_{pu}	243 ksi
- Jacking stress	0.80 f_{pu}	216 ksi – in LRFD Construction Specs Art. 10.10.1

Stress Limits for Concrete

Limiting stresses for concrete

- "Before losses" – immediately after transfer and elastic shortening
  - Really means before time dependent losses have occurred
- "After losses" – final condition, after all time dependent losses have occurred

Stresses in Concrete

Stress in concrete is computed by basic materials science relationships:

$$f_c = \frac{P}{A} \pm \frac{Pe}{S} \mp \frac{M}{S}$$

where:

P = Prestress force
e = eccentricity of prestress force
A = Area of cross section
S = Section modulus
M = Moments due to self-weight and applied loads

Stress Limits: Prestressed Concrete

At transfer (before losses):

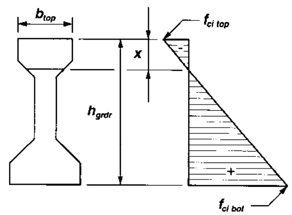
Compression: $0.65f'_{ci}$ Increased from $0.60\,f'_{ci}$ in 2016 Interim

Tension:

- **No bonded reinforcement** $0.0948\lambda\sqrt{f'_{ci}} \le 0.2$ ksi
- **With bonded reinforcement sufficient to resist tension force based upon an uncracked section** $0.24\lambda\sqrt{f'_{ci}}$ ksi
  - Determine area of reinforcement using a working stress f_s of $0.5\,f_y$
  - Clarification adopted in 2022

$$T = \frac{f_{ci\,top}}{2} b_{top} x$$

$$A_s = \frac{T}{f_s}$$



Stress Limits: Prestressed Concrete

At service limit state (after losses):

Compression:

- **Due to permanent loads** $0.45f'_c$
- **Permanent + transient loads** $0.60f'_c$

Tension:

- **Bonded P/S tendons (ksi)** $0.19\lambda\sqrt{f'_c} \le 0.6$ ksi
- **Severe corrosion (ksi)** $0.0948\lambda\phi_w\sqrt{f'_c} \le 0.3$ ksi
  where $\phi_w = 1.0$

Some DOTs use stress limits that differ from these

Caltrans amendments limit stresses to values computed for f'_c = 10 ksi

Stress Check at Transfer

Once strand pattern has been established at midspan, check stresses in girder at transfer for the midspan pattern

- **Ends of girders are most affected**
  - Low self-weight moment cannot counteract the prestress moment
- **A critical location may also be at hold-down for draped strands**

This almost always results in

- **Excessive compression in bottom**
- **Excessive tension in top**

Debonding, draping, and other approaches may be used to control these stresses – discussed later

Check Strength Limit State

Compute factored moment, M_u

- **For Strength Limit State I**

  M_u = 1.25DC + 1.50DW + 1.75LL

Compute nominal flexural resistance, M_n

Assure factored flexural resistance, $M_r = \phi M_n \ge M_u$

Strength limit state does not generally govern for composite girder designs

- **It is more likely to govern for voided slabs or box beam designs**

Check maximum reinforcement

- **Now we modify ϕ to address this**

Check minimum reinforcement

Two Approaches for Nominal Flexural Resistance, M_n

Approximate Method

- Specified in LRFD Specs
- Simple formula-based approach
- Closed-form solution

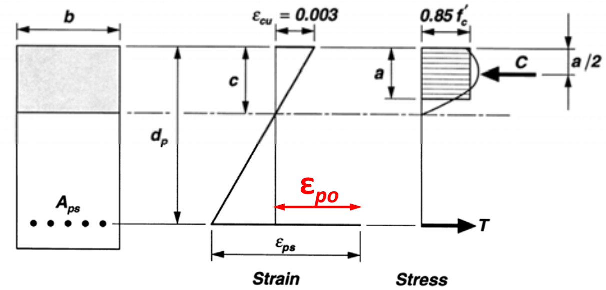
Strain Compatibility (aka – "moment curvature")

- General approach for any shape or material(s)
- Applicable to wide range of cases
- Iterative, calculation-intensive method
- Gives more accurate results

Conditions at Nominal Flexural Resistance, M_n

Same approach as used for reinforced concrete, except for the additional component of initial strain in strand, ε_{po}

- The initial strain in the strand, ε_{po}, caused by prestressing, must be included in the analysis



Approximate Method

An approximate formula for estimating average stress in pretensioning strands at ultimate flexure is given in LRFD Specifications

Simplifies the process of calculating M_n

- Eliminates consideration of nonlinear material properties of both concrete and prestressing steel
- Same as for reinforced concrete, except for stress in strand

This method should be used with caution if used beyond the limits for which it was developed (unusual)

Steel Stress at Nominal Resistance

Average stress in prestressing strands at strength limit state

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{For } f_{pe} \geq 0.5\, f_{pu}$$

where:

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right)$$ = 0.28 for low relaxation strands with $f_{py} = 0.9\, f_{pu}$

Nominal Flexural Resistance

Typical equation is for rectangular sections with only prestressed reinforcement and where neutral axis remains in the deck:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

Same as for reinforced concrete members

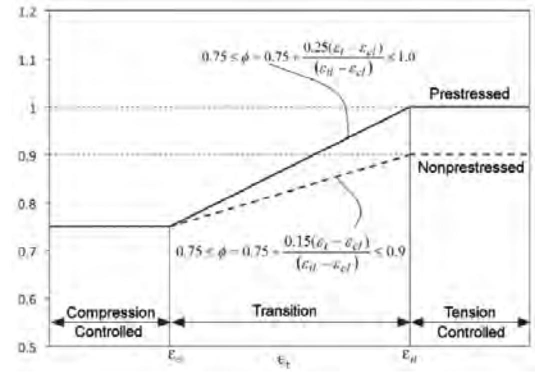
For situations with flanged section behavior where neutral axis is below the bottom of the deck:

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d_s - \frac{a}{2}\right) - A_s^{'} f_y^{'}\left(d_s^{'} - \frac{a}{2}\right) + 0.85 f_c^{'}(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

For rectangular sections, $b_w = b$, so last term drops out

Maximum Reinforcement ⇒ Resistance Factor

The maximum reinforcement limit was removed in 2005 when the resistance factor (ϕ) transition concept was introduced



LRFD Fig. C5.5.4.2-1

Reducing the ϕ factor is used to avoid the less desirable “brittle” or “non-ductile” failure which is equivalent to the intent of the maximum reinforcement limit

Caltrans amendment for CIP PT and spliced girders

Minimum Reinforcement

The factored flexural resistance M_r shall be at least equal to the lesser of:

$M_r \geq 1.33M_u$

$M_r \geq M_{cr}$ where M_{cr}, the cracking moment, is based on $f_r = -0.24\lambda\sqrt{f'_c}$

- **The second limit used to be $M_r \geq 1.2M_{cr}$**
- **The new expression for M_{cr} includes several factors that makes the limit similar to the previous one**

Reinforcement Limits with New Reinforcement Materials

Both of the current maximum and minimum reinforcement concepts encounter some difficulties when being applied to members with new reinforcement materials, like carbon fiber and stainless steel strand, or for UHPC members

- **Typical concepts of yielding and ductility no longer apply**
- **Adequate ductility can be achieved with proper design even with materials that lack traditional ductility**

Expect to see activity in this area in near future, including NCHRP studies and a recent series of *ASPIRE* articles

Strain Compatibility Approach

The strain compatibility approach is based on three well accepted fundamental assumptions

- **plane sections remain plane after bending**
- **compatibility of strains, i.e., full bond between steel and concrete at all sections**
- **equilibrium of forces within a section**

Since strains are computed across the depth of the section, general stress-strain relationships are typically used for both concrete and strands

Strain Compatibility Approach

Step 1: **Assume a top fiber strain, ε_c, and a neutral axis depth, *c***

Step 2: **Compute the stress and force in compression block**

Step 3: **Compute strain in each steel layer**

Step 4: **Compute the stress and force in each steel layer**

Step 5: **Use equilibrium of forces to check assumed neutral axis depth**

Step 6: **Vary neutral axis depth *c* and repeat above steps until forces are in equilibrium, i.e., *C* = *T***

Step 7: **Calculate the nominal flexural capacity by summing moments of all forces about any horizontal axis**

Strain Compatibility Approach

Flexural Strength Relationship

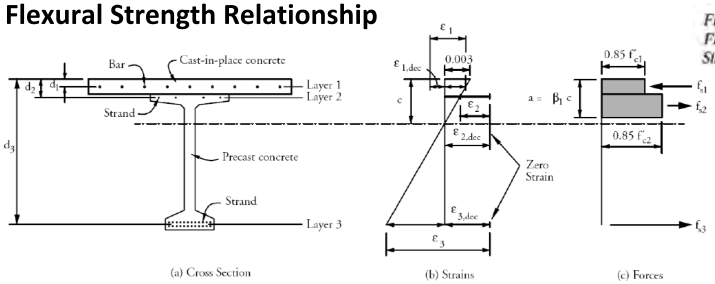


Figure 8.2.2.6-1 Flexural Strength Relationships for Strain Compatibility Analysis

- **Sum forces for equilibrium**
- **Adjust location of neutral axis, *c*, if required**
- **When balanced, compute moment**
- **See discussion in *PCI BDM* for more details**

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Basic Design of Pretensioned Concrete Girders

NEVADA DOT SAFE AND CONNECTED · Caltrans · PCI WEST

Prestressed Concrete Bridge Design Seminar

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Stress Control Approaches for Pretensioned Concrete Girders

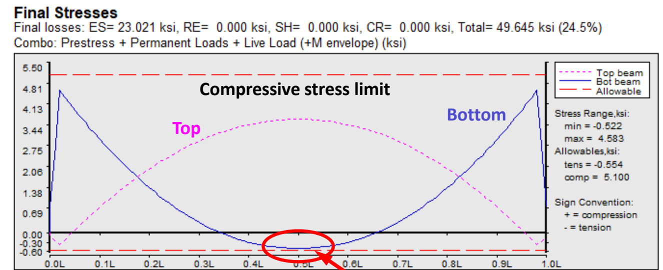
NEVADA DOT SAFE AND CONNECTED — Caltrans — PCI WEST

Flexural Design

To determine number of strands at midspan, check:

- Stresses at Service Limit State (Service I & III)
- Factored resistance at Strength Limit State (Strength I & II)

For girders, service limit state design nearly always governs



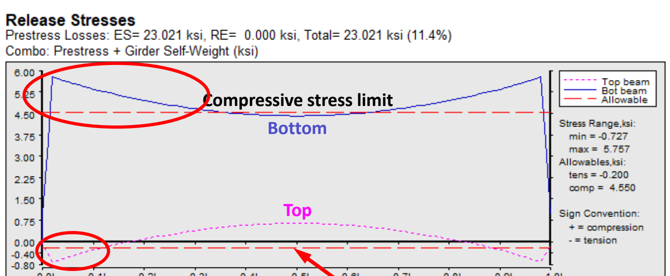
From PSBeam – Eriksson Technologies

Tensile stress limit at Midspan – governs design

Stress Check at Transfer

Check stresses at transfer using midspan strand pattern

- Ends of girders
  - Little or no dead load moment to counteract the effect of the prestress force
  - Compression and tension limits are often exceeded at top and/or bottom



Tensile stress limit

Methods to Address Excessive End Stresses

Available methods:

1. Draping (harping or deflecting)
2. Debonding (blanketing or shielding)
3. Adding top strands
4. Adding strands near midheight (in web between flanges)
5. Adding mild reinforcement for higher allowable tensile stress
6. Increasing compressive strength at release, f'_{ci}
   - Not effective if tensile stress governs
     - Tensile stress is limited to $0.0948\sqrt{f'_{ci}}$ or 0.2 KSI
     - Note: when f'_{ci} > 4.45 ksi, 0.2 ksi limit governs
   - Could be useful if compressive stress governs

Methods 5 & 6 will not be discussed further today

1. Draping, Harping, or Deflecting

Concrete stresses are reduced by reducing eccentricity of prestress at ends

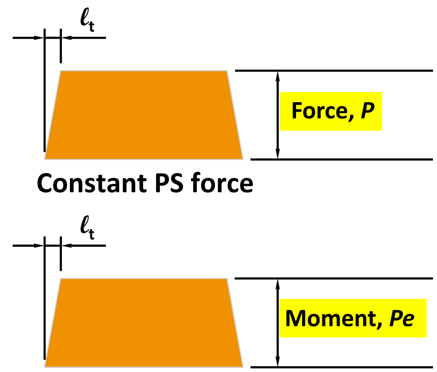
- *Reduces PS moment by reducing the eccentricity, but maintains a constant prestress force*
- Raise strands in the web until stress limits are satisfied at end
  - Only strands in web can be draped
- All strands are active for full length of girder, so full prestress force is available (except within transfer length, ℓ_t)
- Prestress bed must resist the hold-down force at drape points

Shear resistance is improved

- Additional shear resistance from vertical component of inclined PS force, V_p
- Full PS force is available for full length of beam to improve V_c

Draping

Straight Strands

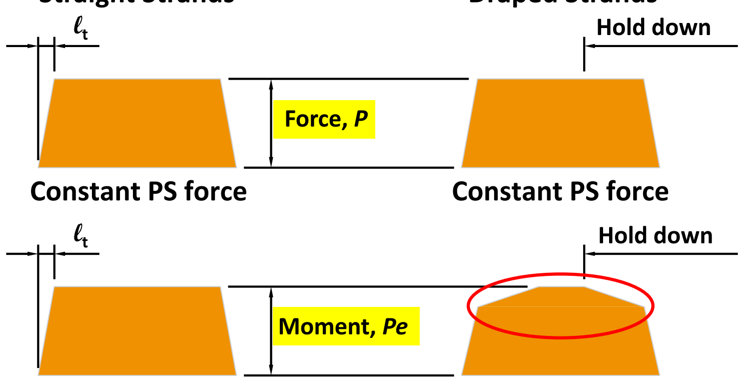


Constant PS Moment

Excessive stresses

Draping

Straight Strands | Draped Strands



Constant PS Moment — *Excessive stresses*

Reduced PS Moment — *Acceptable stresses*

Draping Considerations

Draped strand designs typically reduce concrete stresses at ends of girder at transfer which can lead to lower concrete release strengths, f'_{ci}

Usually use 2 hold-downs each side of beam centerline

- Caltrans practice is to place hold down at 0.33L to 0.40L from end of girder
- Location closer to midspan reduces hold down force - preferred by fabricators
  - 5 ft each side of CL works well – standard for some DOTs
- Hardware can only be positioned at certain locations, and holes have to be cut in soffit form for hold-down anchors

A single hold-down at midspan is rarely used

- Hold-down force is nearly twice the force for 2 hold downs

Draping Considerations

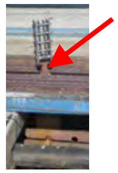
Splitting or repositioning hold-down does not significantly affect the design and should be allowed

- Prestressers show split hold-down locations on shop drawings when required
- Designers don't have to worry about this

Split hold-down –
Design just shows one

Hold down capacity limited by capacity of hardware system

- 48 kips – Limited by hold down hardware and rod attachment to bed
- 7.5 kips per strand – Limited by roller

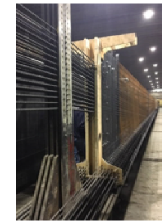


Load on hold down is affected by number and slope of the strands

Draping Considerations

Safety is a major concern when draping strands

- Tensioned strands are typically raised to drape position
- Any problems can cause serious injury or death



Stressing strands in draped position is preferred by fabricators

- Some DOTs allow this practice, which greatly improves safety
- Standard procedures are given in PCI QC manual

Draping Considerations

When draped strands are raised into final position

- Raising strands provides additional elongation, and therefore increases stress in the strands
- Draped strands are tensioned to a lower "final" stress to account for the additional tension that will occur from raising strands

When draped strands are stressed in final position

- Draped strands will be slightly longer than the straight strands, so elongations will be different from straight strands … but only very slightly different

In both cases, draping adds complexity to stressing operations because forces and/or elongations for draped strands will be different

Draping Considerations

More labor is required to install draped strands

- 500 ft of 0.6 in. diam. strand weighs about 370 lbs
- A sled can be used to drag several straight strands at once
- A sled cannot be used for draped strands

2. Debonding or Shielding

Concrete stresses are reduced by reducing effective PS force at section

- Debonding strands prevents transfer of prestress force from the strand to the concrete until farther from the end of the girder
- *Reduces both PS force and moment*
- Debonding must extend to end of girder to allow stress to be relieved

Preferred by almost all prestressers

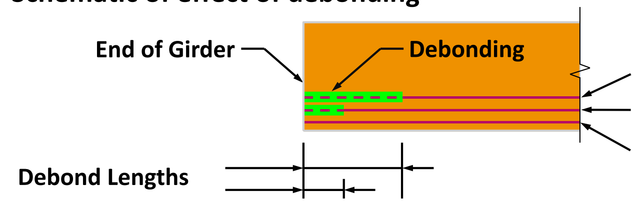
- Safer for workers
- Eliminates cost of hold-downs
- But installation is labor intensive

Debonded designs require more strands than draped – typically about 6

Debonding is the only option for sections with sloped webs like tub girders

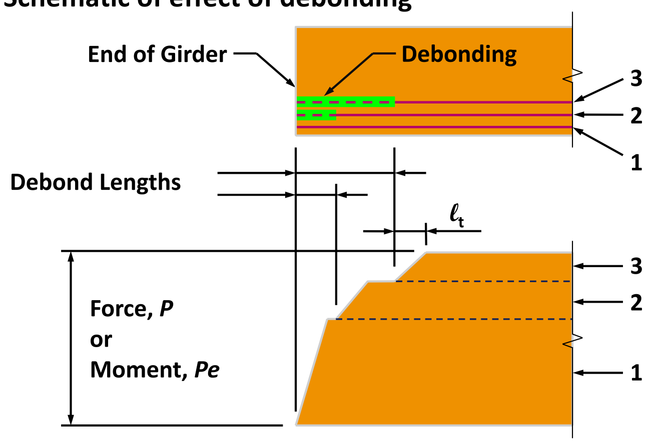
Debonding

Schematic of effect of debonding



Debonding

Schematic of effect of debonding



Debonding

For debonding to be effective

- Bond must be prevented to end of girder and throughout length of debonding to allow strand to be unbonded and unstressed as intended
- Ends and length of sheath must be sealed to prevent entry of concrete

Debonding material

- Two-piece snap together sleeves
- Solid tube
- Split sleeve (not allowed in some states)

Installation

- Must access strands to place and seal debonding
- Not all strands are easily accessible

Debonding

Effect of debonding on design (compared to draping)

- **Shear resistance is reduced**
  - **Reduced PS force because of disabled strands at ends; high shear locations**
  - **No vertical component of inclined PS force, V_p**
  - **Can be addressed by adding stirrups, if necessary**
- **Longitudinal reinforcement capacity is reduced**
  - **Can be addressed by adding stirrups**
  - **Can also be addressed by adding longitudinal reinforcement, but this approach is generally avoided because adding bars increases congestion**
- **Camber is affected**

Designer must consider effects and adjust the design as needed

Debonding Limits - *Revisions*

Significant revisions to LRFD Art. 5.9.4.3.3 were adopted in June 2019

- **Mostly based on findings of NCHRP Report 849**

Summary of requirements in the 9th edition (2020)

- **Development length, ℓ_d, shall be doubled if tension exists in "precompressed tensile zone"**

Caltrans exceptions to 8th ed.:

- ~~**No. of debonded strands should not exceed 25%**~~ — **33% maximum**
- **No. of debonded strands per row shall be ≤ ~~40%~~ 45%** — **50% maximum**
- **All limit states must be satisfied**
- **Not more than ~~40% of debonded strands or 4~~ 6 strands shall have debonding terminated at a location (or in some cases 4)**
- **Debond symmetrically about centerline**
- **Exterior strands in each full width horizontal row shall be bonded**

Debonding Limits - *Revisions*

A. The number of strands debonded per row shall not exceed 45 percent of the strands provided in that row, unless otherwise approved by the Owner.

- ***No limit on the total number debonded***
- ***This and other limits will restrict the number of strands debonded***
- ***Example provided later***

- ***Caltrans exceptions***
- ***50% maximum per row with 33% maximum total of debonded strands***

Debonding Limits - *Revisions*

B. Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section.

Debonding Limits - *Revisions*

C. Longitudinal spacing of debonding termination locations shall be at least $60d_b$ apart.

- *For 0.6 in. diameter strands, minimum longitudinal distance between debonding termination locations = 36 in.*
- *This provision staggers debonding termination locations to avoid stress concentrations*

Debonding Limits - *Revisions*

D. Debonded strands shall be symmetrically distributed about the vertical centerline of the cross-section of the member. Debonding shall be terminated symmetrically at the same longitudinal location.

Debonding Limits - *Revisions*

E. Alternate bonded and debonded strand locations both horizontally and vertically.

- *Intended meaning: Debonded strands shall not be located adjacent to other debonded strands both horizontally and vertically*
- *In other words, do not place debonded strands next to each other either horizontally or vertically*

Debonding Limits - *Revisions*

F. Where a portion or portions of a pretensioning strand are debonded and where service tension exists in the precompressed tensile zone, the development lengths, measured from the end of the debonded zone, shall be determined using Eq. 5.9.4.3.2-1 with a value of $\kappa = 2.0$.

- *No change*
- *When you get tension in the precompressed tensile zone, you have to design for twice the development length, i.e.,* $\kappa = 2.0$
- *Center regions of girders typically must satisfy this requirement*
- *However, strength rarely governs so this generally has little or no impact on designs*

Debonding Limits - *Revisions*

G. For simple span precast, pretensioned girders, debonding length from the beam end should be limited to 20 percent of the span length or one half the span length minus the development length, whichever is less.

Maximum length of debonding for simple spans:

- ***20% of span length***
- ***(span / 2) - ℓ_d***

- ***This is intended to limit debonding of strands where they are being used for the capacity of the section at strength limit state***
- ***While not stated, if you have more capacity than needed, you could use longer debond lengths***
- ***This probably has a minor impact on designs***

Debonding Limits - *Revisions*

H. For simple span precast girders made continuous using positive moment connections, the interaction between debonding and restraint moments from time-dependent effects (such as creep, shrinkage and temperature variations) shall be considered. For additional guidance refer to Article 5.12.3.3.

- ***Basically, consider all effects on the design***
- ***Only applies to continuous for live load designs***

- ***ALDOT prohibits the use of simple span precast girders made continuous***

Debonding Limits - *Revisions*

I. For single-web flanged sections (I-beams, bulb-tees, and inverted-tees):

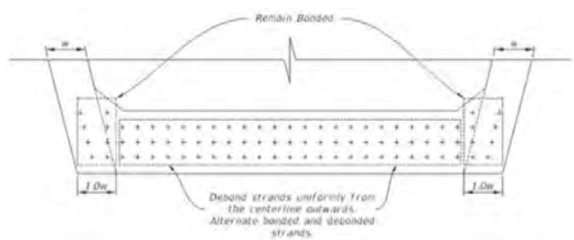
- **Bond all strands within the horizontal limits of the web when the total number of debonded strands exceeds 25 percent.**
- **Bond all strands within the horizontal limits of the web when the bottom flange to web width ratio, b_f/b_w, exceeds 4.**
- **Bond the outer-most strands in all rows located within the full-width section of the flange.**
- **Position debonded strands furthest from the vertical centerline.**

- ***See later examples for bullets 1-3***
- ***Start debonding strands as far from web(s) as possible***

Debonding Limits - *Revisions*

J. For multi-web sections having bottom flanges (voided slab, box beams and U-beams)

- **Uniformly distribute debonded strands between webs.**
- **Strands shall be bonded within 1.0 times the web width projection.**
- **Bond the outer-most strands within the section.**



- ***Bond strands in projected area of the web extended up including outmost strands***
- ***Add debonding from CL, farthest from webs***

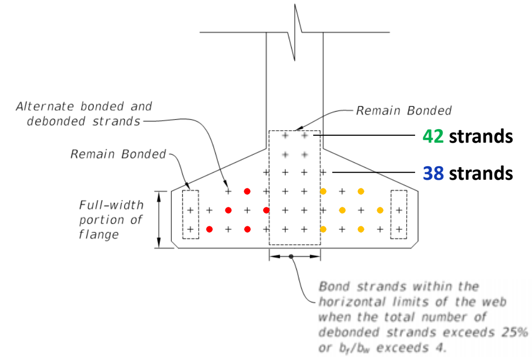
Debonding Limits - *Revisions*

K. For all other sections

- Debond uniformly across the width of the section.
- Bond the outer-most strands located within the section, stem, or web.

Debonding Limits - *Revisions*

Example of PCI BT from commentary to LRFD Art. 5.9.4.3.3 (9th ed.)



LRFD Fig. C5.9.4.3.3-1 – Details for Single-Web Flanged Sections

• • *Different debonding patterns on each side of girder: 10 or 12*

Strands fully within limits of the web are bonded if either

- b_f/b_w = 26"/6" = 4.33 > 4 ***Yes***
- no. of debonded > 25% ***Yes***
  - 12 of 38 debonded = 32% *26%*
  - 12 of 42 debonded = 29% *24%*

Exterior strands bonded only in bottom 2 rows (full width)

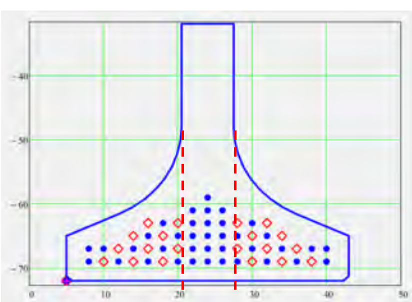
Debonded strands not adjacent

Limit of 45% in row prevents debonding in 4th row

Debonding Limits - *Revisions*

FIB-72 Example received from Will Potter (FDOT) for LRFD 9th ed.

- 171 ft simple span with 7 ft girder spacing



Outer-most strands in widest rows bonded

- Rows 1, 2: Debond 6 of 17 = 35% < 45% OK
- Row 3: Debond 4 of 11 = 36% < 45% OK
- Row 4: Debond 4 of 9 = 44% < 45% OK

Total Debonded: 20 of 58 = 34% > 25%

- If remove top 4 strands: 20 of 54 = 37%

Check b_f/b_w = 38"/7" = 5.42 > 4
All strands in web must be bonded

Revised provisions allow more debonded strands

Debonding Considerations

Consider access when selecting debonded strands

- Workers must access strands to apply and seal ends of debonding sleeves
- When side forms are fixed, such as NEXT beams, strands can only be accessed from above
- Fabricators can adjust debonded strand locations in shop drawings

Ends of debonded strands should be sealed after cutting flush

Using debonded strands eliminates possible conflicts between draped strands and diaphragm holes for bolts, rebar, or inserts

Draping or Debonding?

It is recommended to try straight strand designs first (no debonding)

If necessary (which is usually the case), add debonding

- A debonded straight strand design generally requires about 6 more strands than a draped strand design

Provide the shortest debond length required by design to control stresses

Draping and debonding can be combined if needed

3. Adding Top Strands

Adding top strands to a pattern will improve stress conditions at the ends of a girder

Adding top strands is thought to require adding an equal no. of bottom strands to compensate

- It is not that simple because each strand also adds precompression to the section
- See article in *ASPIRE* by Dr. Bruce Russell

Use of temporary top strands eliminates detrimental effects away from girder ends where their contribution is not required

- See brief discussion that follows

Adding Top Strands

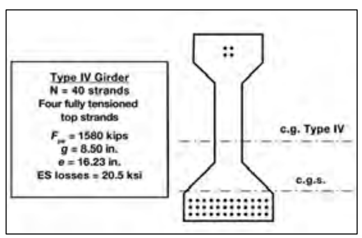
Article by Russell in Summer 2018 issue of *ASPIRE*

- Type IV girder with 36 strands; add 4 top strands

Table 1. Concrete stresses at midspan at service load comparing designs with straight strands. Case I: thirty-six bottom strands as in example girder; Case II: Case I plus four fully bonded top strands. The values show that the addition of the top strands adds 137 psi tension to the bottom fiber of the composite section.

Concrete Stresses at Midspan at Service Loads After All Prestress Losses (ksi) Both designs contain 36 fully tensioned bottom strands				
DL + SDL				
	Case I No Top Strands		Case II Four Top Strands	
	f_{top}	f_{bot}	f_{top}	f_{bot}
Prestress-Axial (F_{pe}/A) Prestress-Bending ($F_{pe}e/S$)	−1.568 2.880	−1.568 −2.433	−1.745 2.508	−1.745 −2.119
Stresses due to Prestressing Only ($[F_{pe}/A] + [F_{pe}e/S]$) (Difference: Case II − Case I)	**1.312**	**−4.001**	**0.763** (−0.549)	**−3.864** (+0.137)
Girder Self Weight (M_g/S) Slab Weight (M_{slab}/S) SDL on Non-Composite (M_{SDL}/S) SDL on Composite (M_{SDL}/S)	−1.481 −1.478 −0.142 −0.098	1.251 1.248 0.120 0.264	−1.481 −1.478 −0.142 −0.098	1.251 1.248 0.120 0.264
Total Stresses with DL + SDL	**−1.887**	**−1.118**	**−2.435**	**−0.980**
Add Effect of LL: DL + SDL + LL				
LL on Composite	−0.567	1.524	−0.567	1.524
Total Stresses @ Midspan @ DL + SDL + LL (Difference: Case II − Case I)	**−2.454**	**0.407**	**3.002** (+0.548)	**0.544** (+0.137)

Note: DL = dead load; LL = live load; and SDL = superimposed dead load. Compressive stresses are negative, and stresses shown are in ksi.

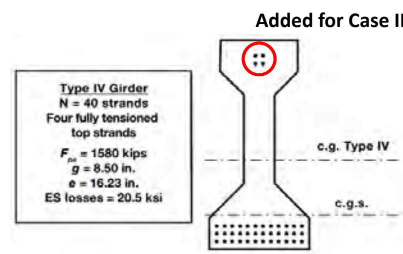


- Summary on next slide

Adding Top Strands

Assumptions:

- Type IV girder
- Case I: 36 strands
- Case II: Add 4 top strands



Change in stresses from adding 4 top strands was minor for this girder

- Top flange stress was improved significantly (more compressive)
- Bottom flange increase in tensile stress (decrease in compression) was 0.137 ksi at transfer and 0.110 ksi at full service limit state

If strands placed at top kern point – no increase in tension at full service

Temporary Top Strands

Revision to AASHTO LRFD approved in 2018 includes provisions for temporary top strands

- A portion of requirements appear below

Add the following Article:

5.9.4.5 Temporary Strands

Temporary top strands may be used to control tensile stresses in precast prestressed girders during handling and transportation. These strands may be pretensioned or post-tensioned prior to lifting the girder from the casting bed or post-tensioned prior to transportation of the girder. Detensioning of temporary strands shall be shown in the construction sequence and typically occurs after the girders are securely braced and before construction of intermediate concrete diaphragms, if applicable.

Pretensioned temporary strands are debonded over the center portion of the girder. If pretensioned, the development length, measured from the end of the debonded zone, shall be determined as described in 5.9.4.3.3 No other provisions of 5.9.4.3.3 apply to temporary strands.

- Based on experience from WSDOT
- Not yet widely used in many areas of US

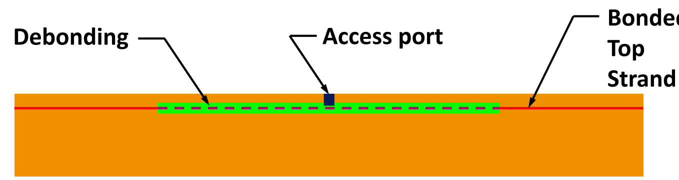
Temporary Top Strands

Problem: Reduce concrete stresses at ends by adding top flange strands

- But adding top strands decreases effect of prestress at midspan

Solution: Make some or all top strands temporary by debonding them in center portion of girder

- Strands must be detensioned to disable
- Provide access port for detensioning strands



Temporary Top Strands

Usually wait to detension temporary top strands until after erection

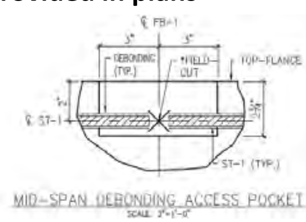
- Improves lateral stability and concrete stresses during handling, shipping, and erection
- Reduces camber growth prior to erection
- Contractor must anticipate camber that occurs at detensioning
- Plan notes must inform contractor about detensioning procedures

Top strands may be post-tensioned, but they still must be detensioned

Temporary Top Strands

Access pocket detail in top flange for detensioning temporary strands must be provided in plans

Example:



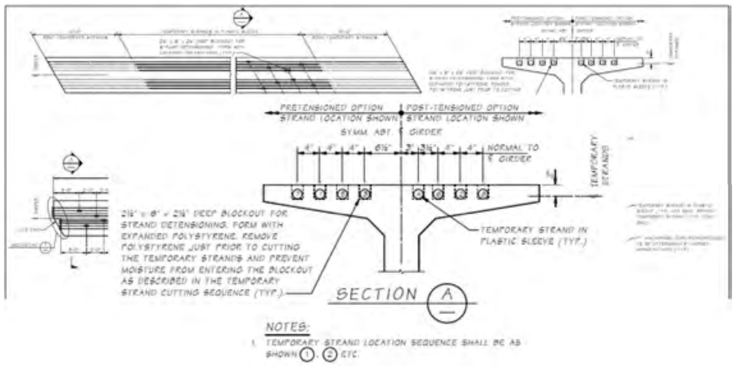
NOT RECOMMENDED – Ports were at both ends

- Single access pocket located near midspan
  - Stagger locations across flange width
  - Size port to allow proper detensioning procedures
- Protect access pocket from intrusion of water if girder may be exposed to freezing

Temporary Top Strands

Details from a WSDOT standard drawing

- Post-tensioned and pretensioned options provided

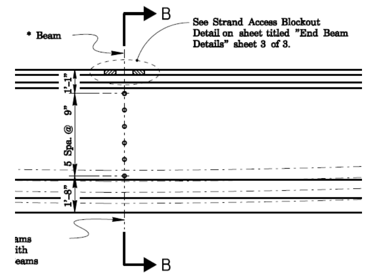


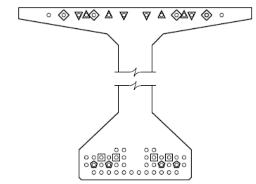
Source: http://www.wsdot.wa.gov/publications/fulltext/Bridge/Web_BSD/5.6_A4_5.pdf

Temporary Top Strands

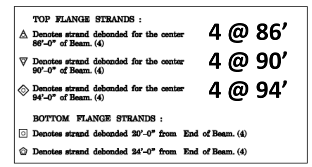
Some suggested details

Access blockout detail near midspan for strand detensioning





STRAND DEBONDING DETAIL



TOP FLANGE STRANDS :

△ Denotes strand debonded for the center 86'–0" of Beam. (4) — **4 @ 86'**

▽ Denotes strand debonded for the center 90'–0" of Beam. (4) — **4 @ 90'**

◇ Denotes strand debonded for the center 94'–0" of Beam. (4) — **4 @ 94'**

BOTTOM FLANGE STRANDS :

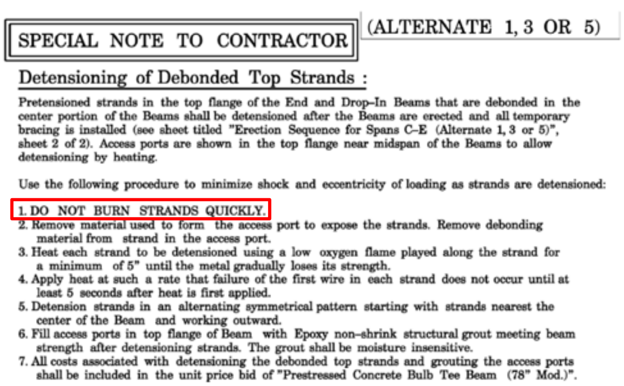
□ Denotes strand debonded 20'–0" from End of Beam. (4)

⬠ Denotes strand debonded 24'–0" from End of Beam. (4)

For 178'-6" girder

Temporary Top Strands

Provide plan note for field detensioning of debonded top strands based on strand detensioning procedures in construction specifications



SPECIAL NOTE TO CONTRACTOR (ALTERNATE 1, 3 OR 5)

Detensioning of Debonded Top Strands :

Pretensioned strands in the top flange of the End and Drop–In Beams that are debonded in the center portion of the Beams shall be detensioned after the Beams are erected and all temporary bracing is installed (see sheet titled "Erection Sequence for Spans C–E (Alternate 1, 3 or 5)", sheet 2 of 2). Access ports are shown in the top flange near midspan of the Beams to allow detensioning by heating.

Use the following procedure to minimize shock and eccentricity of loading as strands are detensioned:

1. DO NOT BURN STRANDS QUICKLY.
2. Remove material used to form the access port to expose the strands. Remove debonding material from strand in the access port.
3. Heat each strand to be detensioned using a low oxygen flame played along the strand for a minimum of 5" until the metal gradually loses its strength.
4. Apply heat at such a rate that failure of the first wire in each strand does not occur until at least 5 seconds after heat is first applied.
5. Detension strands in an alternating symmetrical pattern starting with strands nearest the center of the Beam and working outward.
6. Fill access ports in top flange of Beam with Epoxy non-shrink structural grout meeting beam strength after detensioning strands. The grout shall be moisture insensitive.
7. All costs associated with detensioning the debonded top strands and grouting the access ports shall be included in the unit price bid of "Prestressed Concrete Bulb Tee Beam (78" Mod.)".

Temporary Top Strands

Detensioning of top strands should be discussed with contractor in the preconstruction meeting

Contractor must understand that strands should be detensioned, not cut

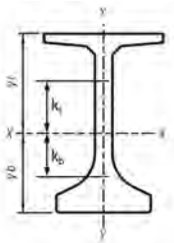
- Results of cutting strands should not be disastrous, but may crack the top flange by energy in strand being released too quickly

4. Adding Strands near Midheight

Strands added between the top and bottom flanges can also be useful

Concept of the kern:

- **A strand added at the limits of the kern will have no effect on the stresses at the opposite face**
- **The location of the limits of the kern are measured from the centroid of the cross section**



$k_t = S_b/A$

$k_b = S_t/A$

where,

$S_{t,b}$ = section modulus top, bottom

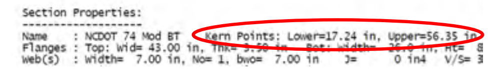
A = area of cross section

Adding Midheight Strands

Kern points for sections

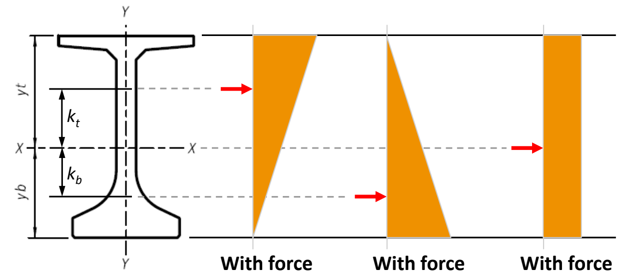
	Mod BT-72	FIB-72	FIB-63
k_t (in.)	18.61	21.90	19.05
k_b (in.)	19.44	17.46	15.20
$k_t/y_t = k_b/y_b$	0.528	0.547	0.544

From PSBeam output:



```
Section Properties:
--------------------
Name     : NCDOT 74 Mod BT     Kern Points: Lower=17.24 in, Upper=56.35 in
Flanges  : Top: Wid= 43.00 in, Thk= 3.50 in    Bot: Width= 26.0 in, Ht= 8
Web(s)   : Width=  7.00 in, No= 1, bwo=  7.00 in     J=      0 in4   V/S= 3
```

For bulb tee shapes, about 0.5 of y_b or y_t

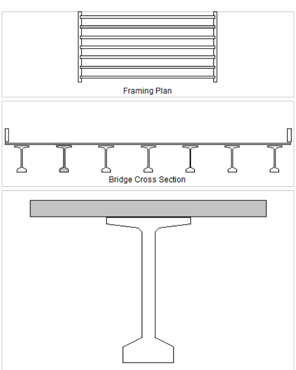


With force @ k_t **With force @ k_b** **With force @ cg**

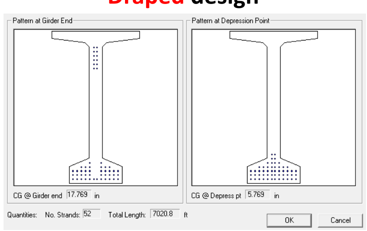
Design Example

L = 135 ft; Mod BT-74; Interior girder

Spacing = 10 ft; width = 68 ft



Draped design



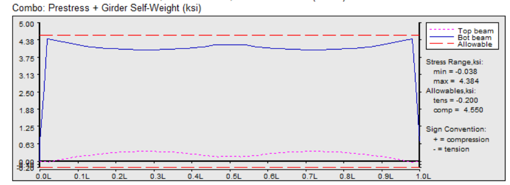
52 strands, 12 draped

No strands in top flange

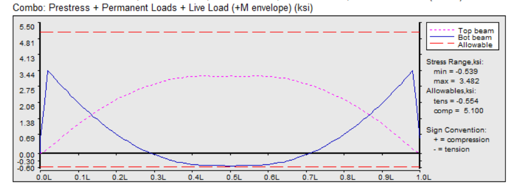
Design Example

Results of Draped Strand Design – stress limits are satisfied at all locations

Release Stresses
Prestress Losses: ES= 23.127 ksi, RE= 0.000 ksi, Total= 23.127 ksi (11.4%)
Combo: Prestress + Girder Self-Weight (ksi)



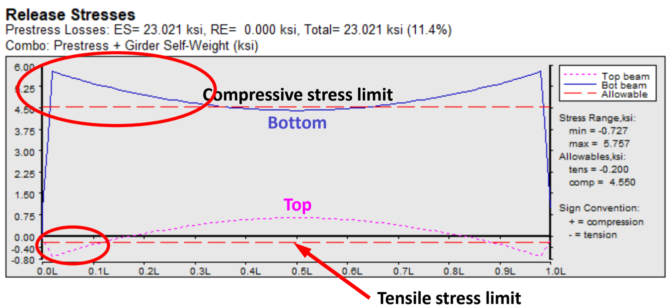
Final Stresses
Final losses: ES= 23.127 ksi, RE= 0.000 ksi, SH= 0.000 ksi, CR= 0.000 ksi, Total= 47.393 ksi (23.4%)
Combo: Prestress + Permanent Loads + Live Load (+M envelope) (ksi)



Design Example

Try to develop a **straight** strand design

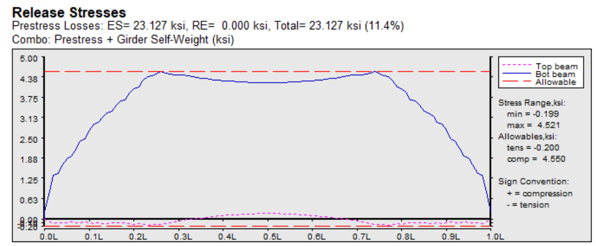
- 50 strands, no debonding
  - Both compressive and tensile limits exceeded

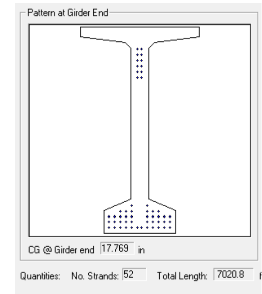


Design Example

Try to develop a straight strand design

- 50 strands, with debonding
  - Stress limits satisfied
  - Debonding limits were exceeded
    - 36 total strands = 72% ⇒ 25 to 30% is limit
    - 12 strands in a row = 100% ⇒ 40% is limit





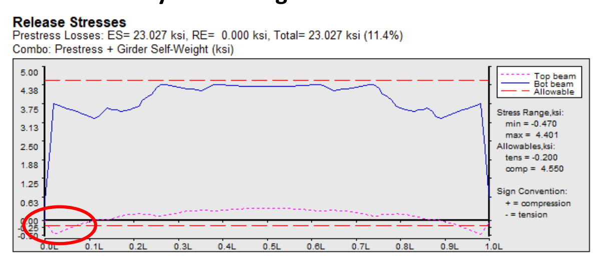
Design Example

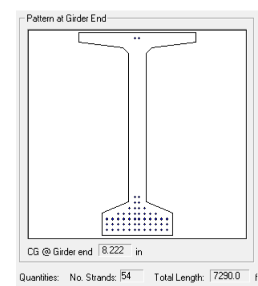
- 50 strands, with debonding
  - Debonding limits were greatly exceeded
- Next try
  - Adding strands only in bottom flange will not be successful
  - Add 2 top strands & 2 bottom strands = 54 strands

Design Example

Try to develop a straight strand design

- 54 strands, with debonding (2 top)
  - Stress limits satisfied at midspan service
  - Cannot satisfy debonding limits and tensile stress limit





  - Debonding pattern is roughly estimated

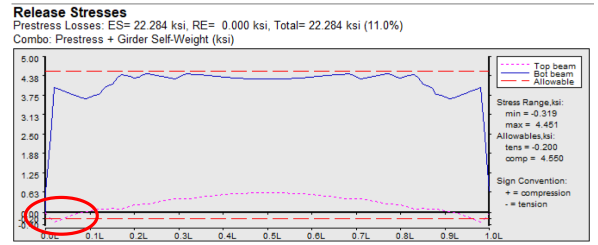
Design Example

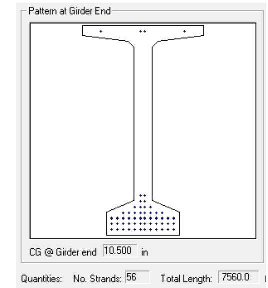
- 54 strands, with debonding and 2 top strands
  - Can't debond enough within limits
- Next try
  - Add 2 more top strands = 58 strands

Design Example

Try a straight strand design

- 56 strands with debonding; 4 strands added in top flange
- Stress limits satisfied at midspan service
  - Cannot satisfy debonding limits and tensile stress limit





- Debonding pattern is roughly estimated

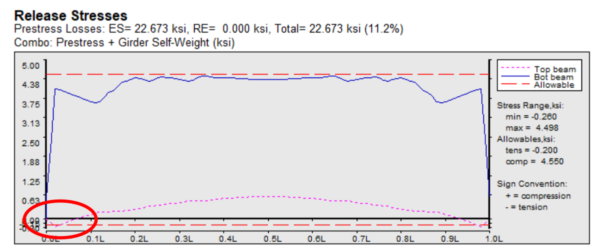
Design Example

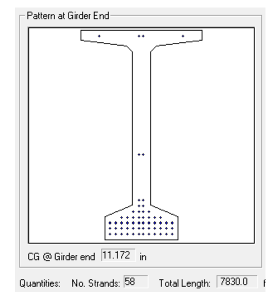
- 56 strands, with debonding and 4 top strands
  - Can't debond enough within current limits
    - Can only debond 14 strands
- Next try
  - Don't want to add 2 more at top – if did, would have to make them temporary because of midspan service
  - Only option is to increase precompression – add strands within the kern
  - Add 2 strands in lower part of kern = 58 strands

Design Example

Try a straight strand design

- 58 strands, with debonding; add 4 strands in top flange
  - Stress limits satisfied at midspan service
  - Cannot satisfy debonding limits and tensile stress limit





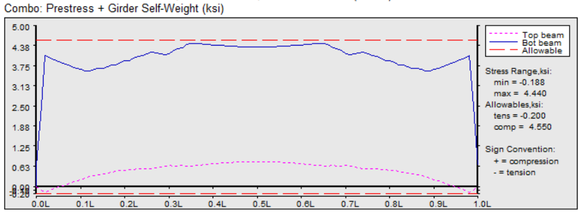
  - Can address compressive stress with debonding, so will raise the mid-height strands to deal with tension

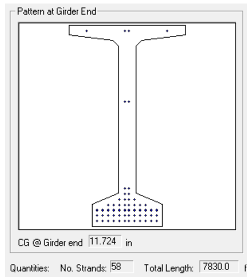
Design Example

Try a straight strand design – adding 2 more strands near midheight

- **58 strands with debonding; add 4 strands in top flange, 2 in web**
  - **Stress limits satisfied at midspan service**
  - **Debonding and tensile stress limits satisfied**

Release Stresses
Prestress Losses: ES= 22.284 ksi, RE= 0.000 ksi, Total= 22.284 ksi (11.0%)
Combo: Prestress + Girder Self-Weight (ksi)

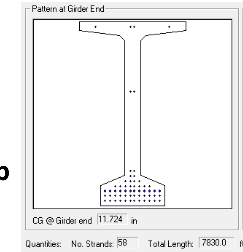




  - **Debonded solution with midheight strands required 6 more strands than draped**

Design Example

Some lessons learned



- **There are many solutions!**
  - **This is just one, and was done quickly**
  - **Won't be an automated solution – requires designer input**
- **Use concept of kern to assist in placing strands in web**
- **Top strands can be used without debonding if midspan service is still satisfied**
- **Adding top strands also adds compression (P/A), so effect of cancelling bottom strands is mitigated**
- **PCI BTs are very limited in no. of strands that can be debonded**

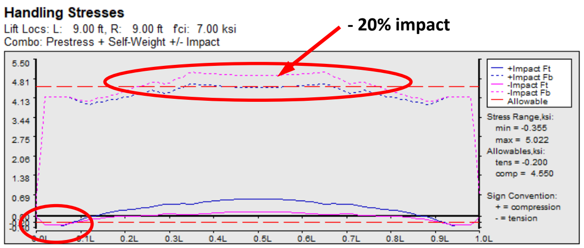
Should also check the stress conditions for the girder when lifting

Design Example

Evaluate girder stresses when lifted

- **Place lifting loops at 1.5 x height of girder, say 9 ft**
- **May use +20% impact for handling in plant, but -0%**

Handling Stresses
Lift Locs: L: 9.00 ft, R: 9.00 ft f'ci: 7.00 ksi
Combo: Prestress + Self-Weight +/- Impact

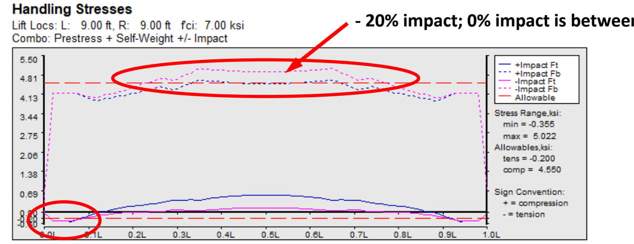


- **Stresses are not satisfied – needs further analysis**
  - **Check stability to move in lifting loops, or add strands**

Design Example

Possible further analysis for lifted girder

Handling Stresses
Lift Locs: L: 9.00 ft, R: 9.00 ft f'ci: 7.00 ksi
Combo: Prestress + Self-Weight +/- Impact



- **Max compression is 4.8 ksi - exceeds limit of 4.55 ksi**
  - **Could handle with f'_{ci} = 7.4 ksi**
- **Max tension is -0.36 ksi > -0.2 ksi; cracking ≈ -0.63 ksi**
  - **Can evaluate tension force and provide reinforcement**

Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

Stress Control Approaches for Pretensioned Concrete Girders

Full Length Debonding of Strands

To allow casting of prestressed girders, box beams, or cored slabs with different strand patterns in the same bed, full length debonding of unneeded strands can be used

Full-length debonded strands can be left in place or removed from sheath

NCDOT includes this in standard plans for cored slabs and box beams

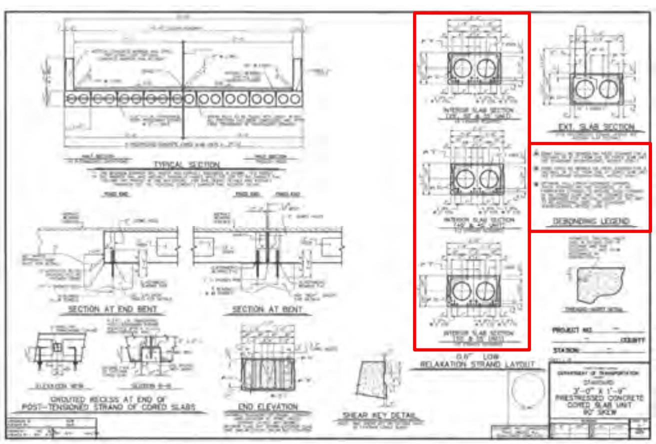
- Note allowing full length debonding appears on plans so fabricators can bid project this way
- Max. no. of full length debonded strands is 10

NCDOT has also approved this practice for other sections on case by case basis

- Savings are only available to the DOT if a note appears on plans so project can be bid using full length debonding

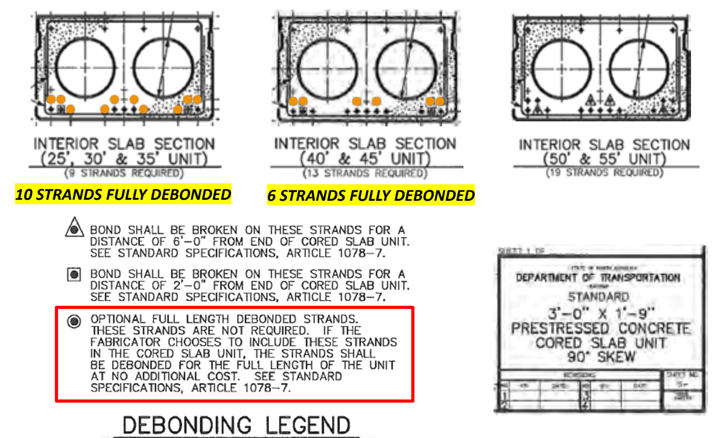
Full Length Debonding of Strands

NCDOT Cored Slab Standards



Full Length Debonding of Strands

NCDOT Cored Slab Standards



Effect of Full Length Debonding of Strands

Full length debonding creates a void in the member cross section since strand is not bonded

- Effect on section properties was evaluated
  - Used a 1-in.-diameter hole at full-length debonded strand locations
  - A conservative estimate of the size of strand with debonding material
- Sections evaluated: 18 in. cored slab, Type III girder, Mod BT-72 girder
- Used 10 strands and 4 or 6 strands debonded to give a range of results
- Composite section properties were also evaluated using 6 ft and 10 ft deck width

Effect of Full Length Debonding of Strands

Change (%) from gross section properties to section with 1-in.-diameter holes at full-length debonded strand locations

Type of Girder	No. Strands	Area	y_b	Y_t	I_{xx}	S_b	S_t
Type III	10	-1.4%	0.4%	-0.3%	-1.8%	-2.2%	-1.5%
	4	-0.6%	0.4%	-0.3%	-1.2%	-1.6%	-0.8%
Mod BT-72	10	-0.9%	0.8%	-0.9%	-2.8%	-3.6%	-2.0%
	4	-0.4%	0.3%	-0.3%	-1.1%	-1.4%	-0.8%
18" CS x 3 ft	10	-1.6%	1.0%	-1.0%	-3.3%	-4.3%	-2.2%
	6	-1.0%	0.5%	-0.5%	-1.4%	-2.0%	-0.9%

Changes in section properties are minor in most cases

Changes for composite section properties were very similar and were always equal or less (except for y_t)

Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

Stress Control Approaches for Pretensioned Concrete Girders

Prestressed Concrete Bridge Design Seminar

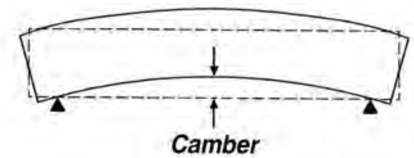
Session 2 – November 3, 2022

Camber

Camber

Definition of camber in the *Manual for Quality Control for Plants and Production of Structural Precast Concrete Products* (PCI MNL-116):

"(1) the deflection that occurs in prestressed concrete elements due to the net bending resulting from application of a prestressing force (It does not include dimensional inaccuracies); (2) A built-in curvature to improve appearance."



For prestressed girders, definition (1) is almost always used

- It is the result of the interaction of prestress force, material properties, and environmental conditions

Camber

As designers, we estimate both initial and long-term camber for girders because there are variables involved that cannot be controlled

Methods for estimating camber at erection and after all losses (long-term)

- Multiplier Methods
- Improved Multiplier Methods – Factors in estimates of prestress loss
- Detailed Analytical Methods – Numerical, time-step evaluation

Factors Affecting Camber

Prestress

- Total no. of strands = Force (*P*)
- Strand pattern (*e*)
- Method for stress control (draped or straight with debonding)

Geometry

- Beam length
- Support locations
- Girder type → section properties
- Girder spacing and deck dimensions

These factors are well known and can be controlled

Factors Affecting Camber

Materials properties – Specified and actual

- f'_{ci} and f'_c
- E_{ci} and E_c
- w_c of girder
- Prestress losses

Fabrication and construction timing

- Age at transfer of prestress
- Age at erection
- Age at deck placement and establishing continuity

Environmental conditions

These factors are based on estimates and some cannot be controlled

Multiplier Method

Most popular method was developed by Martin in an article in the Jan.-Feb. 1977 issue of *PCI Journal* as a rough approximation

- Straightforward calculations
- Apply multipliers to each component of elastic deflection to predict long-term behavior
  - Prestress uplift
  - Self-weight deflections

Assumptions for Elastic Deflections

Use appropriate concrete properties for stage being considered

- Use E_{ci} and f_{pi} for initial camber
- Use E_c at ages > 28 days (final after losses)
- Some DOTs are using "expected" values for f'_{ci} and f'_c for better estimates of cambers and deflections

Girder remains uncracked at all load stages

- Gross (uncracked) section properties
- Transform deck to compute composite section properties
- Transformed prestressing strand may be included

Initial Camber of Bare Beam

$$(\Delta_{\max})_{rel} = (\Delta_{ps})_{rel} \uparrow + (\Delta_{gdl})_{rel} \downarrow$$

Factors affecting initial camber estimate

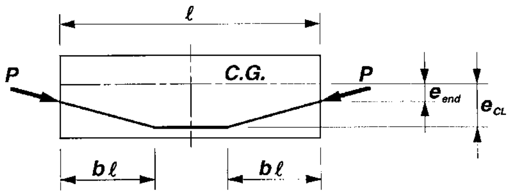
- Concrete properties (specified v. actual)
- Age at release (usually about 18 hrs, but could be over a weekend)
- Curing conditions
- Prestress losses
- Concrete temperature
- Ambient temperature
- Storage and support conditions

Elastic Deflections at Midspan

See *PCI BDM* and *PCI Handbook*

- **Dead load – use standard equation for uniformly loaded beam**
- **Two-point draped strands**

$$\Delta_{\max} = \frac{Pl^2}{24EI}\left[3e_{CL} - (e_{CL} - e_{end})4b^2\right]$$



- **There is also an equation for single point drape**

BDM Table 8.7-1 Camber & Rotations

Prestress Pattern	Equivalent Moment or Load	Equivalent Loading	Camber	End rotation	
(1)	$M = Pe$		$+\frac{M\ell^2}{16EI}$	$+\frac{M\ell}{3EI}$	$-\frac{M\ell}{6EI}$
(2)	$M = Pe$		$+\frac{M\ell^2}{16EI}$	$+\frac{M\ell}{6EI}$	$-\frac{M\ell}{3EI}$
(3)	$M = Pe$		$+\frac{M\ell^2}{8EI}$	$+\frac{M\ell}{2EI}$	$-\frac{M\ell}{2EI}$
(4)	$N = \frac{4Pe'}{\ell}$		$+\frac{N\ell^3}{48EI}$	$+\frac{N\ell^2}{16EI}$	$-\frac{N\ell^2}{16EI}$
(5)	$N = \frac{Pe'}{b\ \ell}$		$+\frac{b(3-4b^2)N\ell^3}{24EI}$	$+\frac{b(1-b)N\ell^2}{2EI}$	$-\frac{b(1-b)N\ell^2}{2EI}$
(6)	$w = \frac{8Pe'}{\ell^2}$		$+\frac{5w\ell^4}{384EI}$	$+\frac{w\ell^3}{24EI}$	$-\frac{w\ell^3}{24EI}$
(7) Debond length	$M = Pe'$		$+\frac{M\ell^2}{8EI}(1-2b_1^2-2b_2^2)$	$+\frac{M\ell}{2EI}\left[(1-2b_1)^2-b_2^2\right]$	$-\frac{M\ell}{2EI}\left[(1-2b_1)^2-b_2^2\right]$

Use superposition to combine different patterns

Final Deflection of Structure

$$(\Delta_{\max})_{fin} = (\Delta_{ps})_{fin}\uparrow + (\Delta_{gdl})_{fin}\downarrow + (\Delta_{ddl})_{fin}\downarrow + (\Delta_{ncdl})_{fin}\downarrow + (\Delta_{sdl})_{fin}\downarrow$$

Additional factors affecting final camber

- **Age of girder when deck placed**
- **Creep**
- **Differential shrinkage**
- **Environmental conditions**
- **Temperature**
- **Structural system, i.e., continuity**

PCI Multiplier Method for Estimating Camber

PS Element with Composite Deck (*PCI Handbook*)

At Erection	
Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	**1.85**
Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	**1.80**
Final	
Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	**2.40**
Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	**2.20**
Deflection (downward) component - apply to elastic deflection due to superimposed dead load only	**3.00**
Deflection (downward) component - apply to the elastic deflection caused by the composite topping	**2.30**

PCI Multiplier Method for Estimating Camber

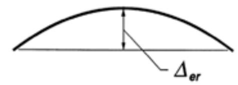
PS Element - no Composite Deck (*PCI Handbook*)

		With Composite Deck
At Erection		
Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	**1.85**	
Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	**1.80**	
Final		
Deflection (downward) component - apply to the elastic deflection due to the member weight at release of prestress	**2.70**	**2.40**
Camber (upward) component - apply to the elastic camber due to prestress at the time of release of prestress	**2.45**	**2.20**
Deflection (downward) component - apply to elastic deflection due to superimposed dead load only	**3.00**	**3.00**
Deflection (downward) component - apply to the elastic deflection caused by the composite topping	**---**	**2.30**

Computing Camber & Deflection

At Release

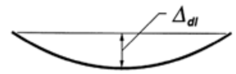
$$\Delta_{rel} = \Delta_{ps}\uparrow + \Delta_{gdl}\downarrow$$



At Erection

$$\Delta_{er} = 1.80\Delta_{ps}\uparrow + 1.85\,\Delta_{gdl}\downarrow$$

Use camber and deflection components computed in initial stage



Added Dead Loads

$$\Delta_{dl} = \Delta_{ddl}\downarrow + \Delta_{ncdl}\downarrow + \Delta_{sdl}\downarrow$$

Final

$$\Delta_{final} = \Delta_{er}\uparrow + \Delta_{dl}\downarrow$$

Multipliers for final conditions should typically not be used for composite sections

Strategy for Improving Camber Estimates

ALDOT Structural Design Manual – Section 5.2

Policies for typical prestressed concrete girder designs with composite deck

Use <u>expected</u> material properties in a separate run only for computing camber

- **Use <u>expected</u> values for f'_{ci} and f'_c, which affect E_{ci} & E_c**
- **Higher values lead to reduced camber, closer to what is being seen with girders**
- **See article in May/June 2022 issue of *PCI Journal* by Mante et al.**

9. The following shall apply for purposes of computing ***expected*** camber and deflection values to be presented in the contract plans ***only***. A second girder analysis run separate from the design run will be required to determine these values.

   - Use *AASHTO LRFD, Article 5.9.3.4, Refined Estimate of Time-Dependent Losses*.
   - Time at strand release: 0.75 days.
   - Time from release of strands to pouring of the bridge deck: 120 days.
   - Relative humidity: 75%.
   - Final age: 27,500 days.
   - Concrete strengths: Use ***expected*** concrete strengths computed as follows:
     - At prestress transfer, f^*_{ci}:
       For 4 ksi $\le f'_{ci} \le$ 5 ksi, $f^*_{ci} = f'_{ci} + 1.95$ ksi
       For 5 ksi $< f'_{ci} \le$ 9 ksi, $f^*_{ci} = 0.9f'_{ci} + 2.45$ ksi
     - At 28 days, f^*_c:
       For $f'_{ci} \le$ 9 ksi, $f^*_c = 1.3f'_{ci} + 3.5$ ksi

Factors Affecting Camber

ALDOT *Structural Design Manual* – Section 5.2

Camber policy using expected concrete compressive strengths is based on research at Auburn

- **Article in May/June 2022 issue of *PCI Journal***
- **Using expected values reduces predicted camber by over 1 in.**
- **Much closer to measured camber**

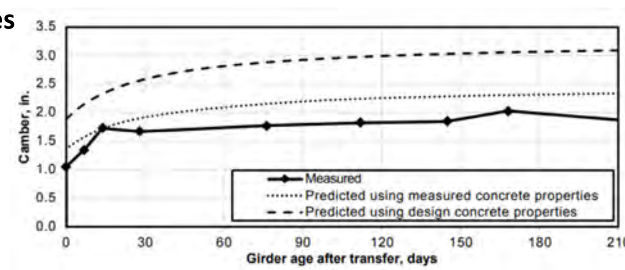


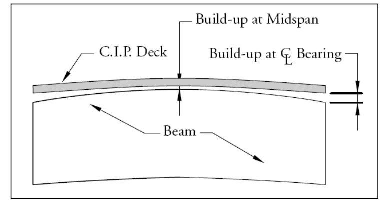
Fig. 1 in *PCI Journal* article

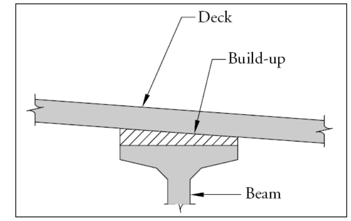
MDOT Best Practices for Estimating Camber of Bulb T and Florida Girders

- This Mississippi DOT research project "Best Practices for Estimating Camber of Bulb T and Florida Girders", State Study 288, was completed in April 2019
- David Tomley with Thompson Engineering (now with Gulf Coast Prestress) was the Principal Investigator
- A copy of the final report can be downloaded from MDOT's Research Division website:
  - https://mdot.ms.gov/portal/research
  - https://mdot.ms.gov/documents/Research/Reports/Interim%20&%20Final/State%20Study%20288%20-%20Best%20Practices%20for%20Estimating%20Camber%20of%20Bulb%20T%20and%20Florida%20Girders.pdf
  - https://mdot.ms.gov/documents/Research/Manuals/Supplemental%20Materials/Technical%20Brief%20-%20Best%20Practices%20for%20Estimating%20Camber%20of%20Bulb%20T%20and%20Florida%20Girders.pdf
- Findings are similar to the ALDOT policy of using expected concrete strengths

thompson ENGINEERING

Determining Specified Build-Up at CL Bearings





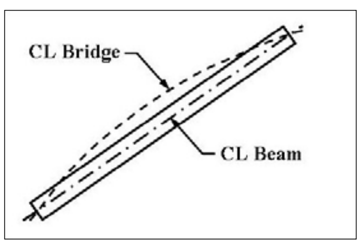
Consider cross-slope and curve effect when establishing build-up at CL bearings in design

- **Design for minimum build-up**

Contractor should determine girder top flange elevations before setting deck and screed elevations for deck using predicted deflection

Bearing seat elevations or roadway grade may be adjusted to accommodate significant differences in camber

Horizontal Curve Effect on Required Build-up

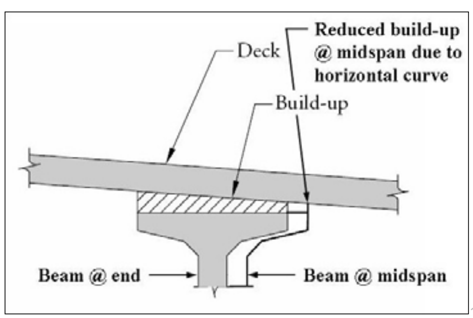


Plan view of beam on horizontal curve

At midspan, girder is offset to inside of curve

Variation of build-up across top flange from cross-slope or super elevation

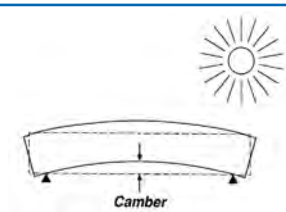
Roadway curvature requires an increased build-up at bearings to maintain minimum build-up at midspan



Other Camber Issues

Thermal camber

- **Sun exposure increases camber**
- **Measure camber early in day**



Bearing location during storage

- **Moving support locations in from end reduces span and increases camber**
- **Moving supports in also improves stability**

Differential camber

- **Complicates fit up for adjacent members**
- **Minimize effect with pre-assembly in plant for adjacent members**

Camber - Summary

Camber predictions are estimates

So-called “more exact methods” are only as good as assumptions

- Use of expected values of concrete strengths can help to provide better cambers for use in design

Girder fabricators often have a good understanding of their materials and processes so may have better estimate of expected cambers

The plant generally can do little to control or modify cambers

Some variation in camber between girders of the same design is normal

Structure should be detailed to accommodate variation in camber

Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

Camber

Prestressed Concrete Bridge Design Seminar

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Some Other Girder Shapes

Different Girder Shapes

For years standard girder shapes have been used across the country

- DOTs have used different reinforcement details and design practices

In recent years, new standard shapes have been developed

- Some designers and owners contend that details and design practices from the agency that developed the sections must be used
- This is not necessary, unless there is some special justification
- However, consistency is desirable for economical design & fabrication

FIB sections are becoming popular to get longer spans for given depth

- But are much heavier – not ideal for all cases
- Standard details for other DOTs are needed for efficient production
- Comparison to PCI BTs follow

Different Girder Shapes

Fabricators may have challenges when girder shapes are changed

- **Cost of new forms is significant**
- **If the pallet width is different, a new pallet must be installed when changing section types which takes time**
- **Existing forms can be modified in some cases**
  - **Spread side forms for wider web – but requires new pallet or filler plates**
    - **Has been used when adding post-tensioning ducts to sections to provide required side cover**
  - **Raise side forms to increase depth of bottom flange**
    - **If raised 2”, can add a row of strands**
    - **Used for PCI BTs in some areas since they have a relatively small bottom flange**

FIB Resources from FDOT

The Florida I-Beam (FIB) section was developed by FDOT

- **The FIB girders are just another girder shape**
- **FDOT has standard design details**

FDOT began requiring use of FIBs for projects in 2009

- **Temporary Design Bulletin C01-09 (01-21-09)**

Design standards released in July 2009

- **Temporary Design Bulletin C03-09 (06-02-09)**

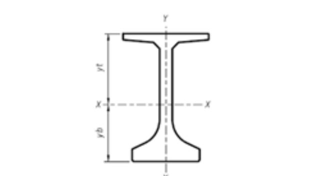
Design Standard Instructions

- **Index 450-010 Series Prestressed Florida-I Beams (2018-19)**
  - **Design guidelines, section properties, details & maximum span charts**

FIB Details

From current Design Standards Instructions

- **Family of sections – same top and bottom flange**



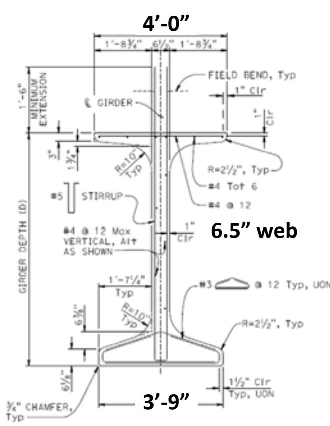
FLORIDA-I BEAM (FIB) SECTION
(FIB-72 INDEX 450-072 SHOWN, OTHERS SIMILAR)

Section Properties						
Beam Size	Area (in.²)	Perimeter (in.)	Ixx (in.⁴)	Iyy (in.⁴)	yt (in.)	yb (in.)
FIB-36	806.58	206.57	127,545	81,070	19.51	16.49
FIB-45	869.58	224.57	226,581	81,327	24.79	20.21
FIB-54	932.58	242.57	359,929	81,584	29.96	24.04
FIB-63	995.58	260.57	530,313	81,842	35.04	27.96
FIB-72	1,058.58	278.57	740,416	82,099	40.06	31.94
FIB-78	1,100.58	290.57	903,861	82,270	43.37	34.63
FIB-84	1,142.58	302.57	1.087×10^6	82,442	46.66	37.34
FIB-96	1,226.58	326.57	1.515×10^6	82,785	53.18	42.82

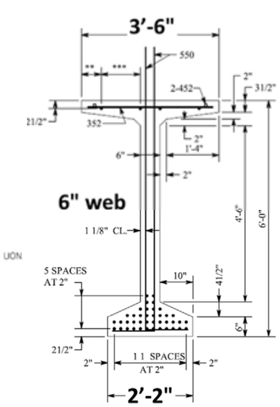
ALDOT projects have used the FIBs

- **Designs currently conform to FDOT standards, with few exceptions**

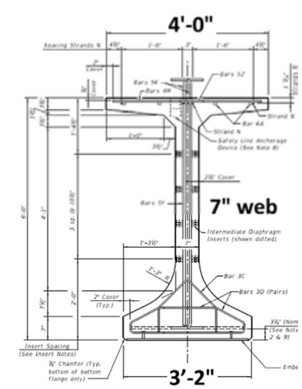
Compare CA Wide Flange Dimensions to PCI BT & FIB



Caltrans Wide Flange Girder



PCI BT-72

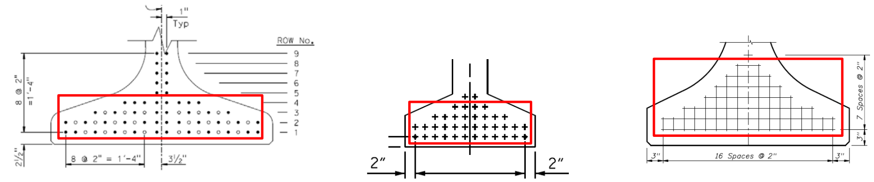


FDOT FIB-72

Compare CA Wide Flange Dimensions to PCI BT & FIB

All dimensions (in.)	CA WF	PCI BT	FIB
Top flange: Width	48.00	42.00	48.00
Edge thickness	3.00	3.50	3.50
Slope height	1.75	2.00	1.50
Fillet height	NA	2.00	3.50
Bottom flange: Width	45.00	26.00	38.00
Edge thickness	6.125	6.00	7.00
Slope height	6.375	4.50	7.50
Web: Width	6.50	6.00	7.00

Compare Strand Patterns



	CA WF	PCI BT	FIB
No. of rows with > 2 strands (red box)	4	4	7
Total no. of strands in these rows	62	36	69
No. of strands in widest row	20	12	17

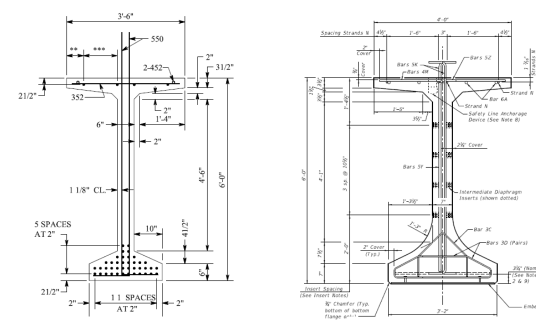
FIB: Only 1 column of strands in web – draped strands are not used
Bottom row at 3 in. above bottom

Compare Properties: PCI BT-72 & FIB-72

	PCI BT-72	FIB-72	FIB/BT
Area (in^2)	767.0	1,058.6	1.38
I_{xx} (in^4)	545,894	740,416	1.36
I_{yy} (in^4)	37,634	82,099	2.18
y_t (in.)	35.40	40.06	1.13
y_b (in.)	36.60	31.94	0.87
S_t (in^3)	15,421	18,483	1.20
S_b (in^3)	14,915	23,181	1.55
Weight / foot (lb/ft)	799	1,103	1.38

- **36% greater moment of inertia, I_{xx}**
- **Weak axis moment of inertia, I_{yy}, is over 2 times greater**
- **FIB is 38% heavier**

Compare Rebar Details: PCI BT-72 & FIB-72

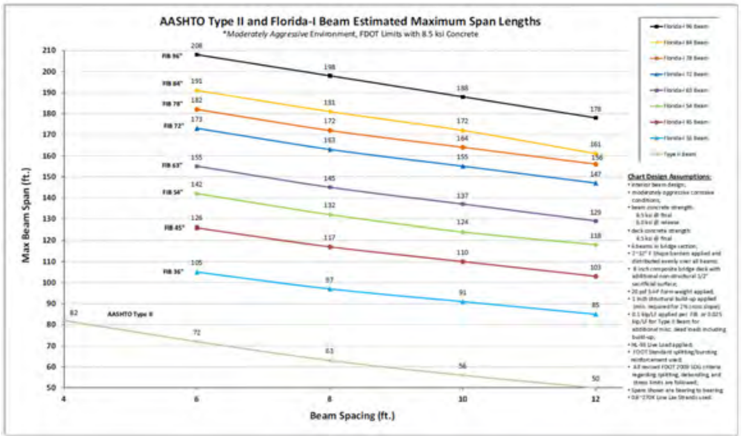


PCI BT-72 GDOT **FIB-72**

Side cover is greater for FIB because stirrups are placed on either side of the center column of strands instead of on each side of a pair of strands in BT

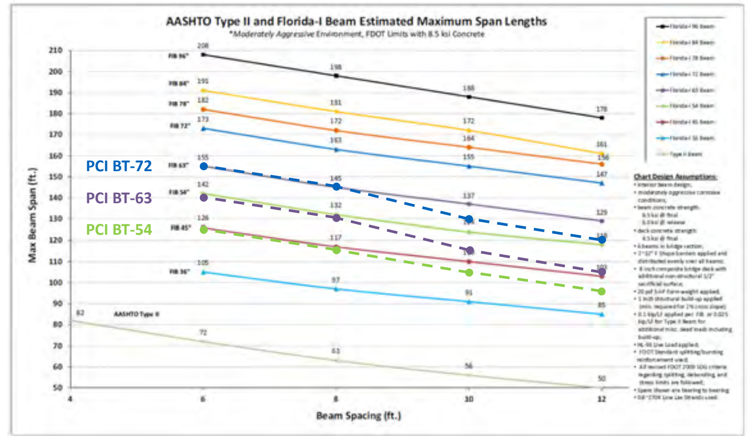
	PCI BT	FDOT
Cover to bottom flange reinforcement	**~1.5 in.**	**2 in.**
Side cover to stirrups	**1.125 in.**	**2.25 in.**

Compare Maximum Design Spans



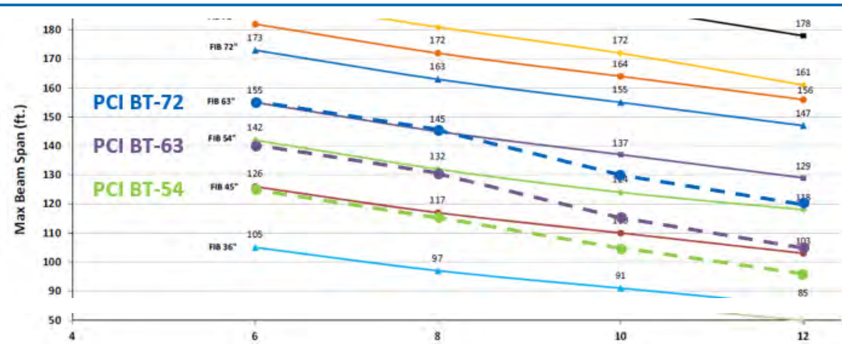
From FDOT Design Standard Instructions

Compare Maximum Design Spans



From FDOT Design Standard Instructions & PCI BDM Chap 6

Compare Maximum Design Spans



- At closer spacings (≤ 8 ft), PCI BT spans ≈ next smaller FIB
- At wider spacings (> 8 ft), PCI BT spans approach 2 sizes smaller FIB

From FDOT Design Standard Instructions & PCI BDM Chap 6

Compare Properties: PCI BT-72 & FIB-63

	PCI BT-72	FIB-63	FIB/BT
Area (in^2)	767.0	995.6	1.30
I_{xx} (in^4)	545,894	530,313	0.97
I_{yy} (in^4)	37,634	81,842	2.17
y_t (in.)	35.40	35.04	0.99
y_b (in.)	36.60	27.96	0.76
S_t (in^3)	15,421	15,135	0.98
S_b (in^3)	14,915	18,967	1.27
Weight / foot (lb/ft)	799	1,037	1.30

- **Slightly smaller moment of inertia, I_{xx}**
- **Weak axis moment of inertia, I_{yy}, is still over 2 times greater**
- **FIB is 30% heavier even though 9 in. shallower**

Other Comparisons – Weight

Weight of FIBs

- For 135 ft span (136.33 ft girder):
  - PCI BT-72: 108.9 kips
  - FIB-72: 150.4 kips (+38%)
  - FIB-63: 141.4 kips (+30%)
  - FIB-54: 132.4 kips (+22%)
- Shipping costs would be higher for FIBs due to greater weight
- A larger crane may be required for erection at site
- But savings from shallower superstructure may more than offset the added costs

Other Comparisons – Required No. of Strands

Number of strands – from a design comparison

For 135 ft span designs (straight strands, UNO):

- Mod BT-74: 60 strands x 7 girders = 420
- FIB-72: 66 strands x 6 girders = 396 (-6%)
- FIB-63: 69 strands x 7 girders = 483 (+15%)
- Mod BT-74: 50 strands x 7 girders = 350 (draped) (-17%)

Results shown are for a modified BT-72 (7 in. web – NCDOT standard)

Results for a PCI BT-72 should be similar

Other Comparisons – Max. PS Force & Draping

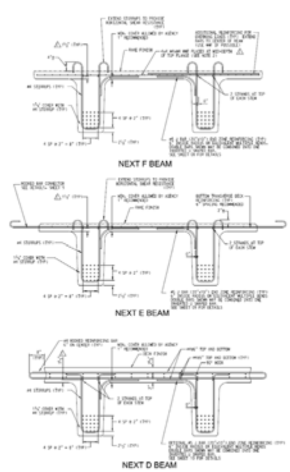
Maximum prestress force can be larger for FIBs because of greater number of potential strand positions

- PCI BT: 48 strands x 44 kips = 2,112 kips
- PCI BT + 2": 60 strands x 44 kips = 2,640 kips (+25%)
- FIB: 74 strands x 44 kips = 3,256 kips (+54%)
  - May exceed capacity of some existing prestress beds

FIB standards use straight strands

Draping could be used for FIBs with either the single center column draped, or shifting the strand pattern to allow 2 columns to be draped (like PCI BTs)

NEXT Beams



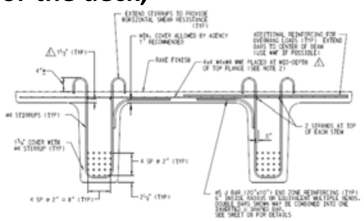
This family of sections was developed by the New England PCI Technical Committee to address needs in their system

- See article by Culmo and Seraderian in Summer 2010 issue of *ASPIRE*
- Developed to provide solutions for bridges where voided slabs and box beams do not work as well
- The double-tee shape works well for situations where utilities have to be carried under the bridge
- Excellent solution for ABC construction

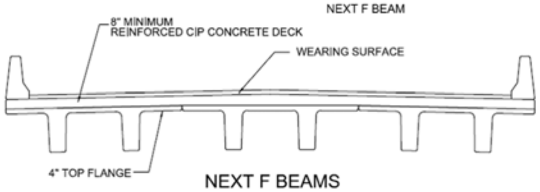
NEXT Beams

The **NEXT F** beam series is designed to have a full depth deck installed

- The top flange is intended to be the form for the deck, hence the **F** designation for "form"
- Eliminates deck formwork
- Parapet reinforcement can be incorporated in the deck slab
- No connection is made between flanges
- Flange width is varied to match bridge width



NEXT F BEAM

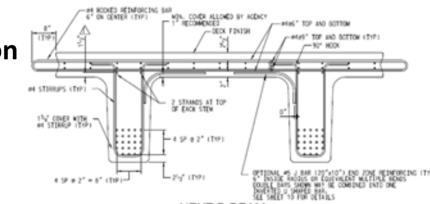


NEXT F BEAMS

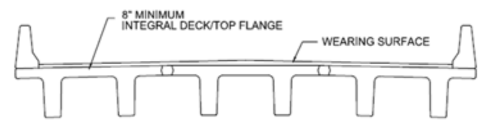
NEXT Beams

The **NEXT D** beam series is designed to provide the full deck section as part of the beam

- Since the top flange is the structural deck, the section has the **D** designation for "deck"
- Deck concrete is plant-cast so there is no deck placement in the field
- Connection between flanges is usually UHPC
- Flange width is varied to match bridge width



NEXT D BEAM

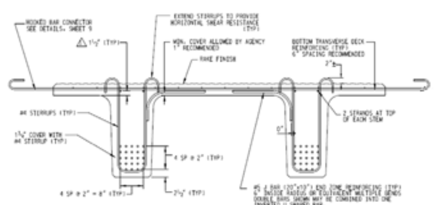


NEXT D BEAMS

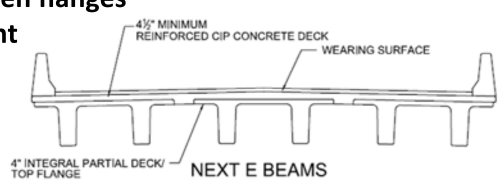
NEXT Beams

The **NEXT E** beam series is designed to provide a partial depth form for the deck section as part of the beam

- The designation **E** was given because it was an intermediate solution between the D and F sections
- A partial thickness deck placement is made in the field
- A connection is made between flanges as part of the deck placement
- Flange width is varied to match bridge width

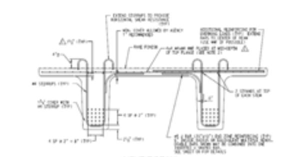


NEXT E BEAM

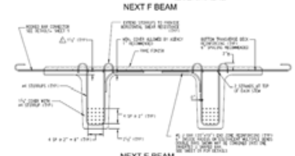


NEXT E BEAMS

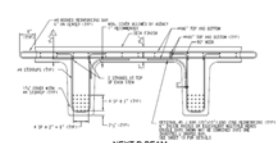
NEXT Beams



NEXT F BEAM



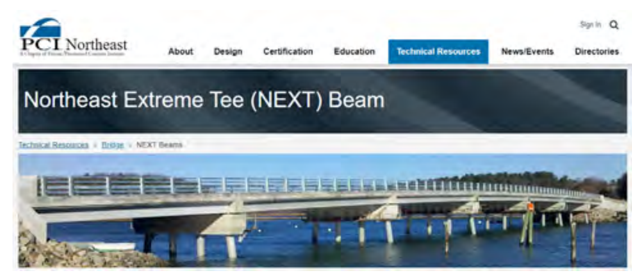
NEXT E BEAM



NEXT D BEAM

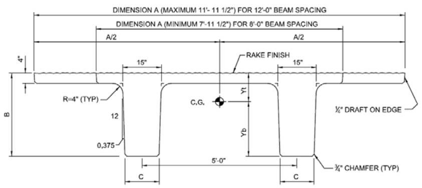
Design guidelines and other information are available on the PCINE website:

https://www.pci.org/PCINE/Technical_Resources/Bridge_Resources/NEXT_Beam.aspx

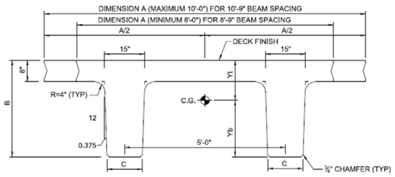


NEXT Beams

Range of sections available:



NEXT F BEAM



NEXT D BEAM

NEXT BEAM - SECTION PROPERTIES

BEAM DESIGNATION	BEAM WIDTH INCHES	BEAM DEPTH INCHES	BASE STEM WIDTH INCHES	AREA IN^2	I IN^4	Yb INCHES	Yt INCHES	St IN^3	Sb IN^3	WEIGHT PLF
	A	B	C			D	E			
MINIMUM WIDTH BEAMS										
NEXT 36 F	95.50	36.00	13.00	1287	160240	21.77	14.23	11261	7361	1341
NEXT 32 F	95.50	32.00	13.25	1182	115813	19.51	12.49	9272	5936	1231
NEXT 28 F	95.50	28.00	13.50	1075	79901	17.24	10.76	7426	4635	1120
NEXT 24 F	95.50	24.00	13.75	966	51823	14.96	9.05	5726	3466	1006
NEXT 36 E	96.00	36.00	13.00	1289	160546	21.79	14.21	11298	7368	1343
NEXT 32 E	96.00	32.00	13.25	1184	116028	19.53	12.47	9305	5941	1233
NEXT 28 E	96.00	28.00	13.50	1078	80042	17.26	10.74	7453	4637	1123
NEXT 24 E	96.00	24.00	13.75	969	51906	14.97	9.03	5748	3467	1009
NEXT 40 D	96.00	40.00	13.00	1667	238087	25.47	14.53	16381	9349	1736
NEXT 36 D	96.00	36.00	13.25	1562	176727	23.03	12.97	13630	7672	1627
NEXT 32 D	96.00	32.00	13.50	1456	126155	20.57	11.43	11039	6132	1517
NEXT 28 D	96.00	28.00	13.75	1347	85684	18.07	9.93	8626	4743	1403
MAXIMUM WIDTH BEAMS										
NEXT 36 F	143.50	36.00	13.00	1479	185525	23.36	12.64	14678	7942	1541
NEXT 32 F	143.50	32.00	13.25	1374	134258	20.98	11.02	12183	6399	1431
NEXT 28 F	143.50	28.00	13.50	1267	92661	18.57	9.43	9826	4990	1320
NEXT 24 F	143.50	24.00	13.75	1158	60045	16.12	7.88	7620	3725	1206
NEXT 36 E	114.00	36.00	13.00	1361	170830	22.44	13.56	12598	7613	1418
NEXT 32 E	114.00	32.00	13.25	1256	123575	20.14	11.86	10419	6136	1308
NEXT 28 E	114.00	28.00	13.50	1150	85300	17.81	10.19	8371	4789	1198
NEXT 24 E	114.00	24.00	13.75	1041	55322	15.45	8.55	6470	3581	1084
NEXT 40 D	120.00	40.00	13.00	1859	258217	26.55	13.45	19204	9724	1936
NEXT 36 D	120.00	36.00	13.25	1754	191497	24.02	11.99	15978	7974	1827
NEXT 32 D	120.00	32.00	13.50	1648	136539	21.44	10.56	12926	6369	1717
NEXT 28 D	120.00	28.00	13.75	1539	92622	18.80	9.20	10072	4926	1603

Prestressed Concrete Bridge Design Seminar

Session 2 – November 3, 2022

Some Other Girder Shapes

NEVADA DOT SAFE AND CONNECTED

Caltrans

PCI WEST

ACCELERATED BRIDGE CONSTRUCTION UTILIZING PRECAST ELEMENTS and RESEARCH BASED CONNECTIONS FOR SEISMIC PERFORMANCE

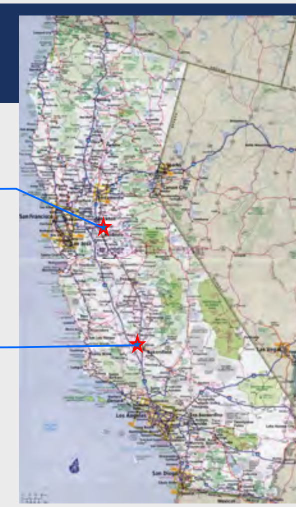
PCI WEST PRECAST PRESTRESSED CONCRETE BRIDGE DESIGN WORKSHOP

BRENT R. KOCH, P.E.

November 3, 2022

Con-Fab California, LLC

INTRODUCTION



INTRODUCTION

Lathrop Plant (est. 1984)

- Caltrans Audited Facility
- PCI "B4" Plant Certification
- PCI "C4" Plant Certification

INTRODUCTION

Shafter Plant (est. 2008)

- Caltrans Audited Facility
- PCI "B4" Plant Certification
- PCI "C4" Plant Certification

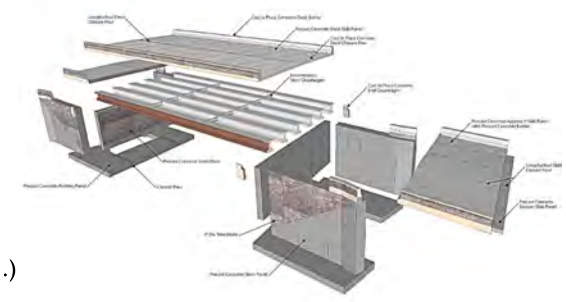
INTRODUCTION

CON-FAB has:

- Broad experience in furnishing and installing a wide range of precast members the California and Nevada bridge construction markets since 1984.
- Fabrication experience for emergency replacement and emergency repair projects.
- Fabricated the full depth deck panels for the deck reconstruction on the James E. Roberts memorial bridge.
- Fabricated and installed precast concrete bridge structure elements for many of the Caltrans ABC projects utilizing UHPC connections between precast elements.

ABC CONSTRUCTION WITH PRECAST CONCRETE ELEMENTS & SYSTEMS

- **Substructure Elements**
  - Columns and Bent Caps
  - Footings & Abutments
  - Wing Walls
  - Arched Bridges
- **Superstructure Elements**
  - Deck Panels: Partial & Full-Depth
  - Precast Girders (Flanged, Box, Tub…)
  - Total Superstructure Systems

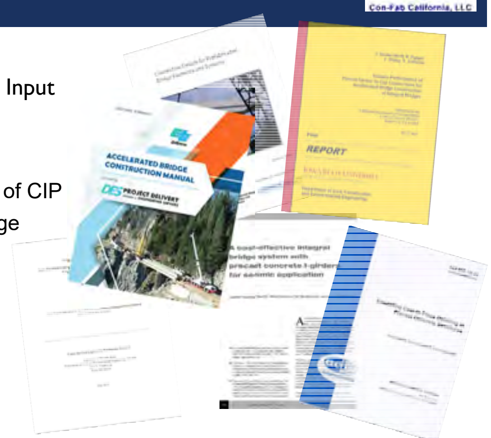


PBES: Prefabricated Bridge Elements & Systems

ABC CONSTRUCTION WITH PRECAST CONCRETE ELEMENTS & SYSTEMS

DESIGN

- Design development benefits from Industry Input
  - Forming and member sizing efficiencies.
  - Lessons from previous fabrication experience.
- Most details are based on emulative design of CIP
- Basic design of the structure does not change
- Convert construction joints to connections
- Methods to emulate a construction joint
  - Research Based Connections
  - Caltrans ABC Manual
  - FHWA manual No. FHWA-IF-09-010



ABC CONSTRUCTION WITH PRECAST CONCRETE ELEMENTS & SYSTEMS

MANUFACTURING & TRANSPORATION CONSTRAINTS

- Maximum weight consideration of fabricator & jobsite lifting limitations
- Maximum height, width & length to accommodate fabrication and shipping limitations.
- Effect of site access limitations on delivery point, cane sizing.
- Attention to abutment and bent cap member sizing.
- Repetition and standardization whenever possible to realize cost efficiencies.
- Fabrication and assembly tolerances to be accounted for during design.

ABC CONSTRUCTION WITH PRECAST CONCRETE ELEMENTS & SYSTEMS

ERECTION WORK PLANS – provide detailed information on the means and methods for incorporating the precast elements into the work. Preconstruction erection plan development meetings involving all parties are critical.

Examples of some of the information in the erection plan:

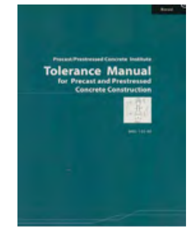
- Shop drawings of all elements with accurate weight determinations.
- Worksite preparations for crane placement and truck access.
- Load staging areas and truck access to the crane.
- Methods of adjusting alignment and securing the element after placement.
- Procedures for controlling tolerances.
- Procedures for constructing connections (forming, placing and curing closures)
- Detailed activity schedule (often to the minute).

ABC CONSTRUCTION WITH PRECAST CONCRETE ELEMENTS & SYSTEMS

TOLERANCES – Accommodation of fabrication and field assembly tolerances must be considered during design and accounted for in the preparation of the erection work plan.

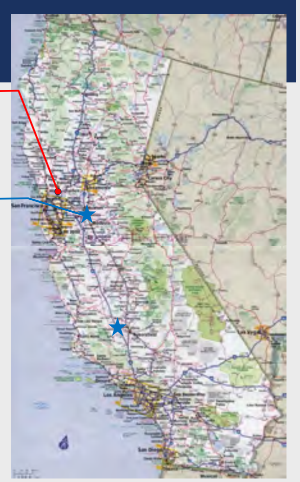
Guidelines for Prefabricated Bridge Elements and Systems Tolerance NCHRP 12-98

- Tolerances of Member Size and Shape
  - Forming
  - Elastic Sortening / Creep / Shrinkage
  - Bar bends
- Location of Inserts & Voids
- Installation Tolerances (Horiz. & Vert.)



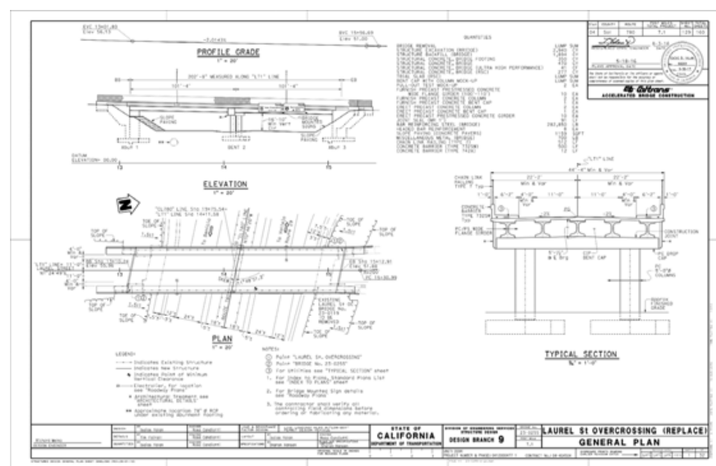
Tolerance Manual for Precast and Prestressed Concrete Construction (MNL-135)

LAUREL STREET OC (REPLACE)
04-4G4504



- First Caltrans ABC contract to implement precast columns and a precast drop cap connected with a grouted UHPC cap to column connection.
- Cap to Column connection follows Caltrans sponsored research of performed by the University of Nevada, Reno

LAUREL STREET OC



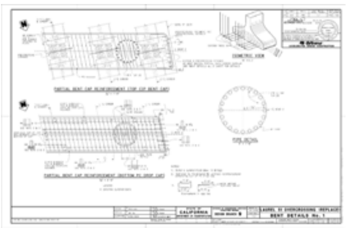
precast elements

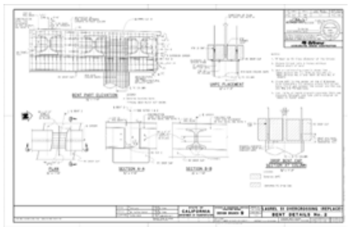
GIRDERS
(10) CAWF48
2 Spans x
5 Girder Lines
98 ft Long
95,000 lbs/ea

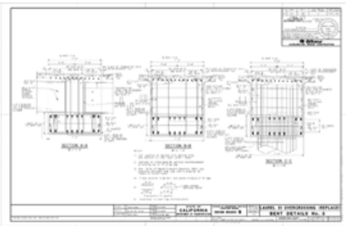
COLUMNS
(2) 5'-0" x 19'
59,000 lbs/ea

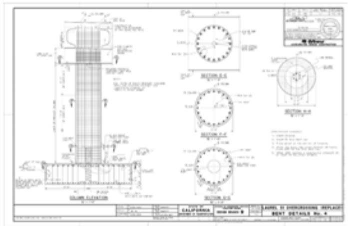
DROP CAP
7 ft Wide
3 ft Deep
44 ft Long
149,000 lbs

LAUREL STREET OC







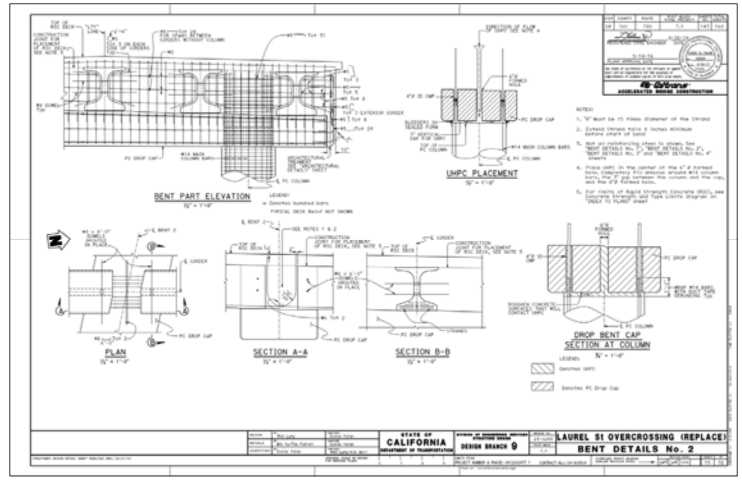


design drawings

BENT DESIGN DETAILS

- Cap to Column Connection with UHPC Grouted Sleeves

LAUREL STREET OC

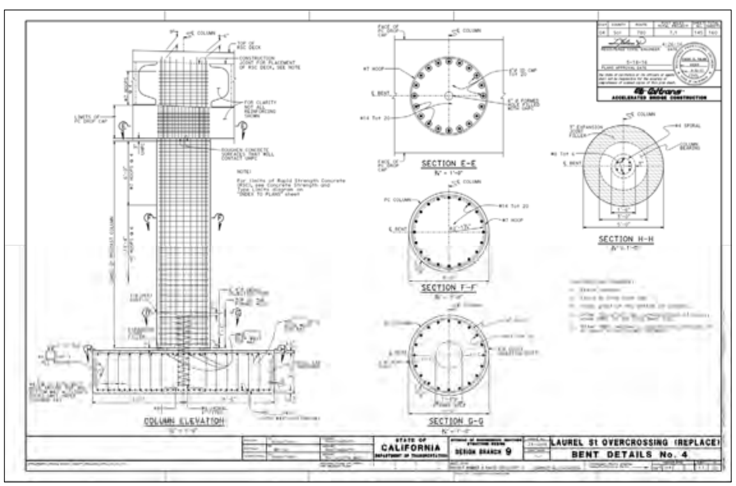


design drawings

BENT DESIGN DETAILS

- Cap to Column Connection with UHPC Grouted Sleeves

LAUREL STREET OC

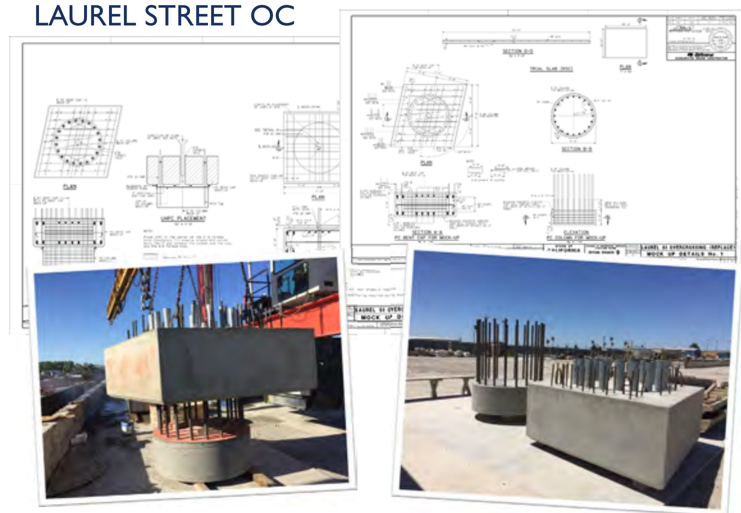


design drawings

BENT DESIGN DETAILS

- Cap to Column Connection with UHPC Grouted Sleeves

LAUREL STREET OC

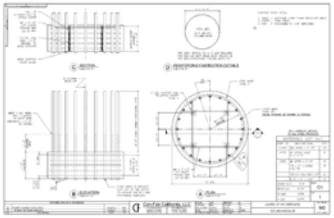


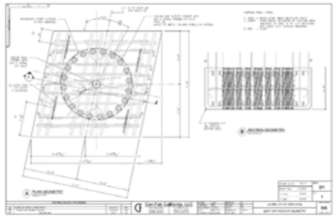
mockups

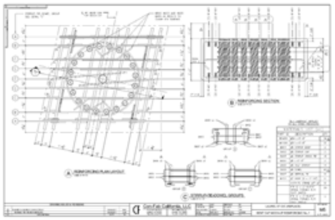
CAP/COLUMN

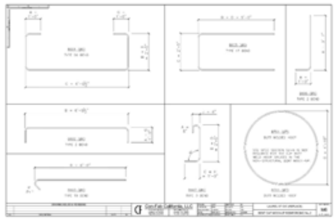
- UHPC interconnection of cap and column mockup components performed at precast plant
- Specimen sawcutting

LAUREL STREET OC









mockups

Bent Cap To Column Connection Mockup

- Quality verification of UHPC material, joint forming and placement procedure

LAUREL STREET OC

mockups

PREPARATION FOR SAW CUTS

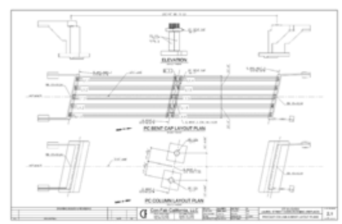
- Challenging assembly handling for safe sawcutting

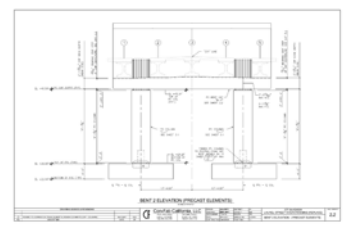
LAUREL STREET OC

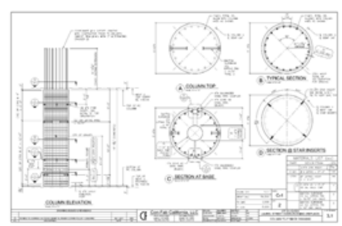
mockups

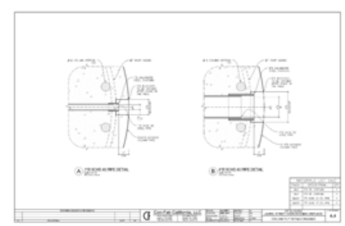
- UHPC placement quality verification

LAUREL STREET OC





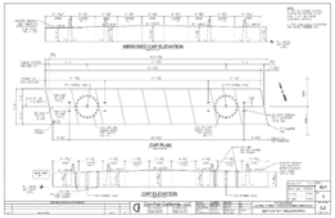


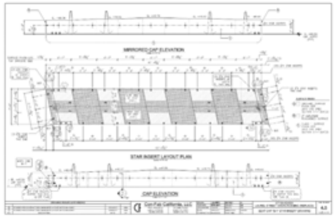


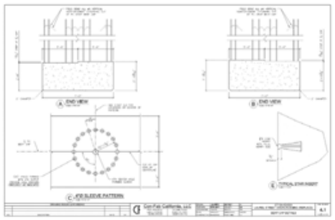
shop drawings

PRECAST DROP CAP and COLUMN SHOP DRAWINGS

LAUREL STREET OC



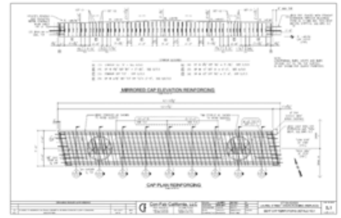


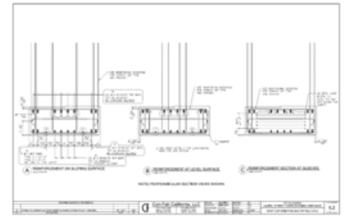


shop drawings

PRECAST DROP CAP and COLUMN SHOP DRAWINGS

LAUREL STREET OC





shop drawings

PRECAST DROP CAP and COLUMN SHOP DRAWINGS

LAUREL STREET OC

plant precast fabrication

COLUMN FABRICATION

- Laser cut bar positioning ring templates
- Cast Vertical
- 5’-0” dia
- 19 ft tall
- 59,000 lbs

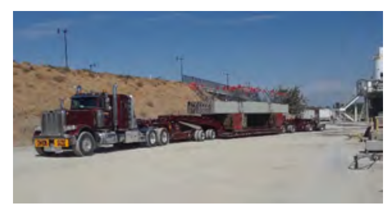
LAUREL STREET OC

Precaged Drop Cap Reinforcement

Cap Pour Setup

Concrete Placement

Stripping From Form



Loaded for Transport to Jobsite

LAUREL STREET OC

fit test

- Revised fit test procedure authorized
- Stub columns cast adjacent to cap form using same rebar ring templates from column fab
- Cap fit tested to stub columns immediately upon stripping

LAUREL STREET OC

Load:______

Con-Fab California LLC
(209) 249-4700 Fax (209) 249-4725

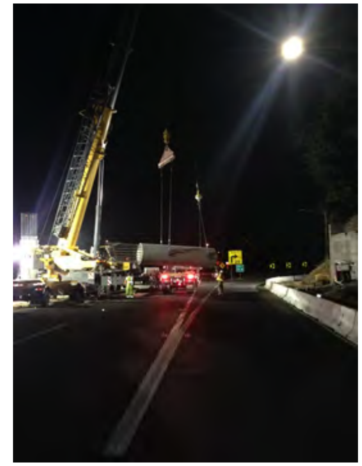
Load List
Laurel Street O/C (Replace)
From Lathrop Plant

Precaster: Con-Fab / Alan 209-969-2307
General Contractor: RNR / 916-379-0957
Crane Contractor: Maxim / Dean / 916-997-6566
Trucking Company: Reeve / Jason - 209-625-6224
Bent Overhang: Front: 4 Ft. Rear: 4 Ft.

Job No.: L16-115
Job Name: Laurel St. Overcrossing Replace
Product: PC Columns
Location: Vallejo, CA
Revised: 8/22/17

Load No.	Delivery Date	Schedule and Pick Times	Mark No.	Location	Aprox. Col. Length	Weight (lbs)	Mark end on Truck	QC	Notes
	29-Aug-17	10:00 PM	Close WB 780						
EB780	Tuesday Night	11:00 PM	RNR to move k-rail and set out plates as required						
		11:30 PM	Rig up Crane						
		1:00 AM	Precast delivery trucks cannot arrive prior to 1:00 am.						
		1:30 AM	Safety Meeting by RNR						
1	Column	2:00 AM	C-1	West	27'-2"	59,300	Bot. of Col. To Cab		
2	Column	2:30 AM	C-1	East	27'-2"	59,300	Bot. of Col. To Cab		
3	Bent Cap	3:00 AM	B-1	Bent	43'-11"	148,900			
		4:00 AM	Rig out Crane						

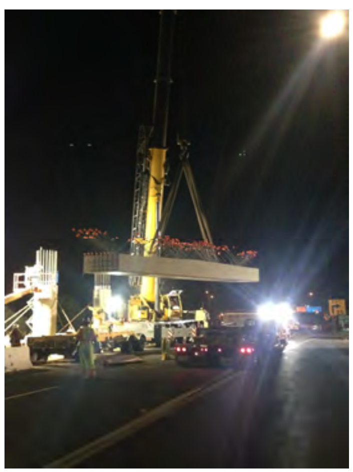
Overall length includes rebar extending out of top of columns.



precast erection

- Columns and cap erected in one shift
- Offload columns and trip to vertical

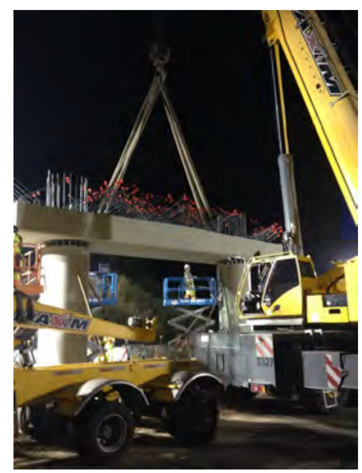
LAUREL STREET OC



precast erection

- Grouted pin connection to footing
- Column spacing and rotational alignment is critical for assembly fitup

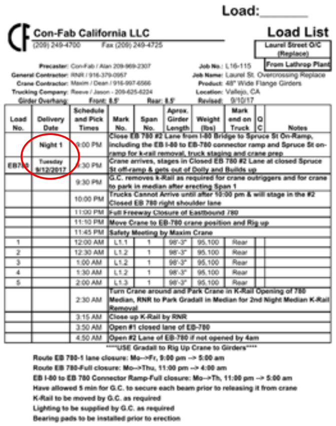
LAUREL STREET OC



precast erection

- Column bar / cap sleeve alignment
- Column to column bar spacing template
- Field measurement confirmation with layout calculations

LAUREL STREET OC



Load:______

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Load List
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From Lathrop Plant

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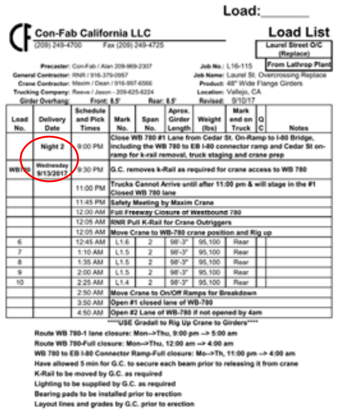
General Contractor: RNR / 916-379-0957
Crane Contractor: Maxim / Dean / 916-997-6566
Trucking Company: Reeve / Jason - 209-625-6224
Girder Overhang: Front: 8.5' Rear: 8.5'

Job No.: L16-115
Job Name: Laurel St. Overcrossing Replace
Product: 48" Wide Flange Girders
Location: Vallejo, CA
Revised: 9/10/17

Load No.	Delivery Date	Schedule and Pick Times	Mark No.	Span No.	Aprox. Girder Length	Weight (lbs)	Mark end on Truck	QC	Notes
	Night 1	9:00 PM	Close EB 780 #2 Lane from I-80 Bridge to Spruce St On-Ramp, including the EB I-80 to EB-780 connector ramp and Spruce St on-ramp for k-rail removal, truck staging and crane prep						
EB780	Tuesday 9/12/2017	9:30 PM	Crane arrives, stages in Closed EB 780 #2 Lane at closed Spruce St off-ramp & gets out of Dolly and Builds up						
		9:30 PM	G.C. removes k-Rail as required for crane outriggers and for crane to park in median after erecting Span 1						
		10:00 PM	Trucks Cannot Arrive until after 10:00 pm & will stage in the #2 Closed EB 780 right shoulder lane						
		11:00 PM	Full Freeway Closure of Eastbound 780						
		11:10 PM	Move Crane to EB-780 crane position and Rig up						
		11:45 PM	Safety Meeting by Maxim Crane						
1		12:00 AM	L1.1	1	98'-3"	95,100	Rear		
2		12:30 AM	L1.2	1	98'-3"	95,100	Rear		
3		1:00 AM	L1.2	1	98'-3"	95,100	Rear		
4		1:30 AM	L1.2	1	98'-3"	95,100	Rear		
5		2:00 AM	L1.3	1	98'-3"	95,100	Rear		
		2:30 AM	Turn Crane around and Park Crane in K-Rail Opening of 780 Median, RNR to Park Gradall in Median for 2nd Night Median K-Rail Removal						
		3:15 AM	Close up K-Rail by RNR						
		3:50 AM	Open #1 closed lane of EB-780						
		4:50 AM	Open #2 Lane of EB-780 if not opened by 4am						

****USE Gradall to Rig Up Crane to Girders****

Route EB 780-1 lane closure: Mo-->Fr, 9:00 pm --> 5:00 am
Route EB 780-Full closure: Mo-->Thu, 11:00 pm --> 4:00 am
EB I-80 to EB 780 Connector Ramp-Full closure: Mo-->Th, 11:00 pm --> 5:00 am
Have allowed 5 min for G.C. to secure each beam prior to releasing it from crane
K-Rail to be moved by G.C. as required
Lighting to be supplied by G.C. as required
Bearing pads to be installed prior to erection



Load:______

Con-Fab California LLC
(209) 249-4700 Fax (209) 249-4725

Load List
Laurel Street O/C (Replace)
From Lathrop Plant

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Girder Overhang: Front: 8.5' Rear: 8.5'

Job No.: L16-115
Job Name: Laurel St. Overcrossing Replace
Product: 48" Wide Flange Girders
Location: Vallejo, CA
Revised: 9/10/17

Load No.	Delivery Date	Schedule and Pick Times	Mark No.	Span No.	Aprox. Girder Length	Weight (lbs)	Mark end on Truck	QC	Notes
	Night 2	9:00 PM	Close WB 780 #1 Lane from Cedar St. On-Ramp to I-80 Bridge, including the WB 780 to EB I-80 connector ramp and Cedar St on-ramp for k-rail removal, truck staging and crane prep						
WB780	Wednesday 9/13/2017	9:30 PM	G.C. removes k-Rail as required for crane access to WB 780						
		11:00 PM	Trucks Cannot Arrive until after 11:00 pm & will stage in the #1 Closed WB 780 lane						
		11:45 PM	Safety Meeting by Maxim Crane						
		12:00 AM	Full Freeway Closure of Westbound 780						
		12:05 AM	RNR Pull K-Rail for Crane Outriggers						
		12:05 AM	Move Crane to WB-780 crane position and Rig up						
6		12:45 AM	L1.6	2	98'-3"	95,100	Rear		
7		1:10 AM	L1.5	2	98'-3"	95,100	Rear		
8		1:35 AM	L1.5	2	98'-3"	95,100	Rear		
9		2:00 AM	L1.5	2	98'-3"	95,100	Rear		
10		2:25 AM	L1.4	2	98'-3"	95,100	Rear		
		2:50 AM	Move Crane to On/Off Ramps for Breakdown						
		3:50 AM	Open #1 closed lane of WB-780						
		4:50 AM	Open #2 Lane of WB-780 if not opened by 4am						

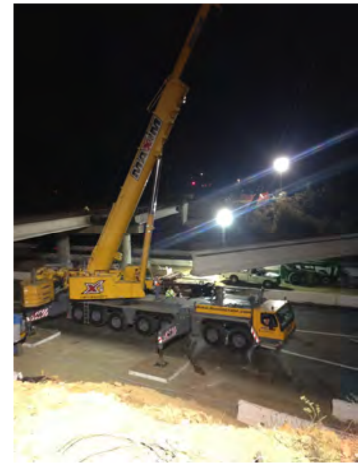
****USE Gradall to Rig Up Crane to Girders****

Route WB 780-1 lane closure: Mon-->Thu, 9:00 pm --> 5:00 am
Route WB 780-Full closure: Mon-->Thu, 12:00 am --> 4:00 am
WB 780 to EB I-80 Connector Ramp-Full closure: Mo-->Th, 11:00 pm --> 4:00 am
Have allowed 5 min for G.C. to secure each beam prior to releasing it from crane
K-Rail to be moved by G.C. as required
Lighting to be supplied by G.C. as required
Bearing pads to be installed prior to erection
Layout lines and grades by G.C. prior to erection

precast erection

- Girders installed 2 weeks after columns and cap
- 5 girders per span erected on 2 consecutive nights

LAUREL STREET OC



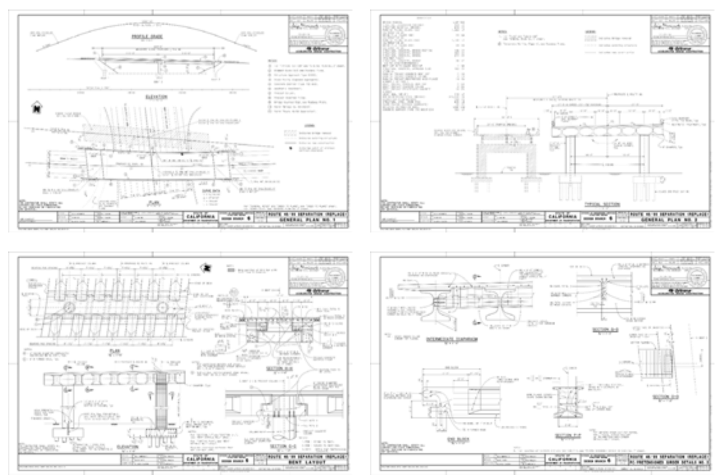
precast erection

- (5) Span 1 girders installed on 9/12/2017
- (5) Span 2 girders installed on 9/13/2017

ROUTE 46/99 SEPARATION (REPLACE)

- Second Caltrans ABC contract to implement precast columns and a precast bent cap connected with a grouted UHPC cap to column connection.
- Integral beam to inverted tee bent cap connection follows the research of Iowa State University.

ROUTE 46/99 SEPARATION



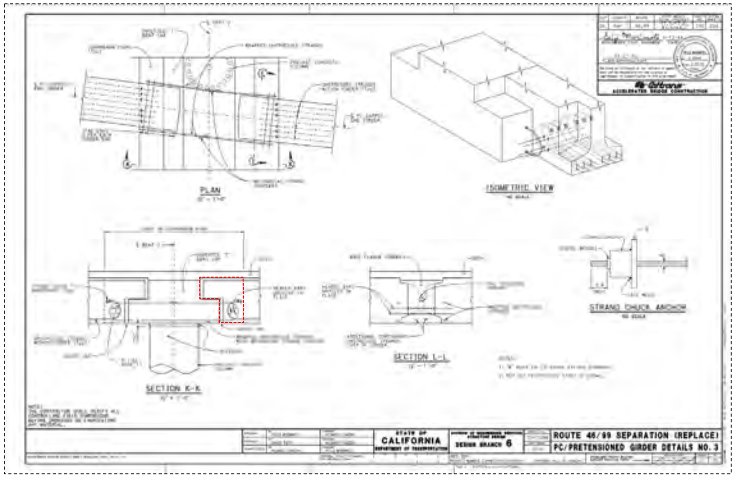
precast elements

GIRDERS
(14) CAWF48
2 Spans
7 Girder Lines
107 ft Long
115,000 lbs/ea

COLUMNS
(2) 4'-6" x 22'-23'
46,000 lbs/ea

INV. TEE CAP
8'-6" ft Wide
4 ft Deep
57 ft Long
245,000 lbs

ROUTE 46/99 SEPARATION

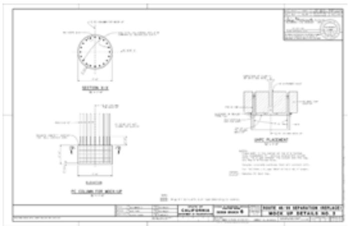


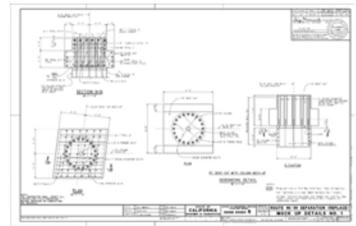
design drawings

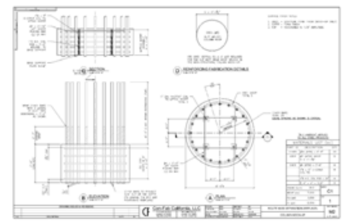
INCREASED COMPLEXITY

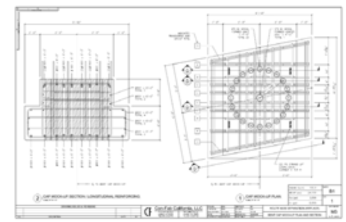
- Profile grade and cross slope geometry
- Girder to cap fixity details

ROUTE 46/99 SEPARATION









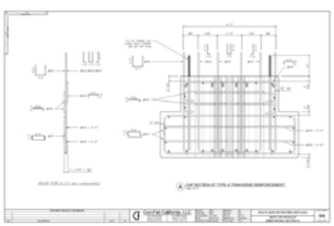
mock-up

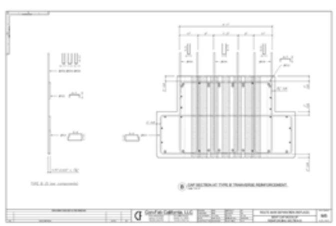
Column and Cap Mock-up

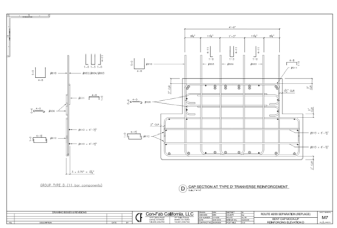
The 46/99 columns were 4'-6" diameter vs 5'-0" for Laurel

Mock-up detailing identified a fit issue for the cap longitudinal steel through the circular vertical array of cap sleeves.

ROUTE 46/99 SEPARATION







mock-up

Cap Mock-up Shop Drawings

ROUTE 46/99 SEPARATION

mock-up

Mockup Fabrication and Assembly

ROUTE 46/99 SEPARATION

mock-up

ROUTE 46/99 SEPARATION

mock-up

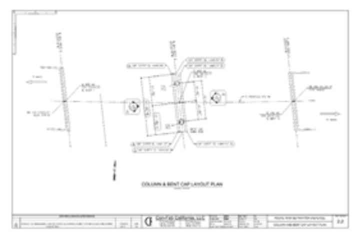
- UHPC work quality verification through sawcutting the mock-up
- Sawcut through UHPC joint between column and cap

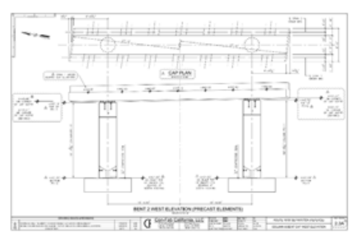
ROUTE 46/99 SEPARATION

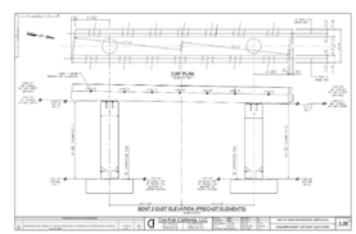
mock-up

- UHPC work quality verification through sawcutting the mock-up
- Sawcut through IT cap above bearing ledge

ROUTE 46/99 SEPARATION



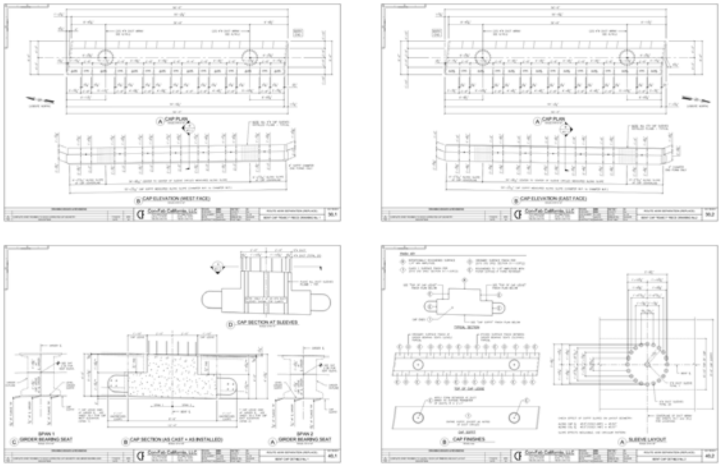




shop drawings

COLUMN and INVERTED TEE BENT CAP ASSEMBLY DRAWINGS

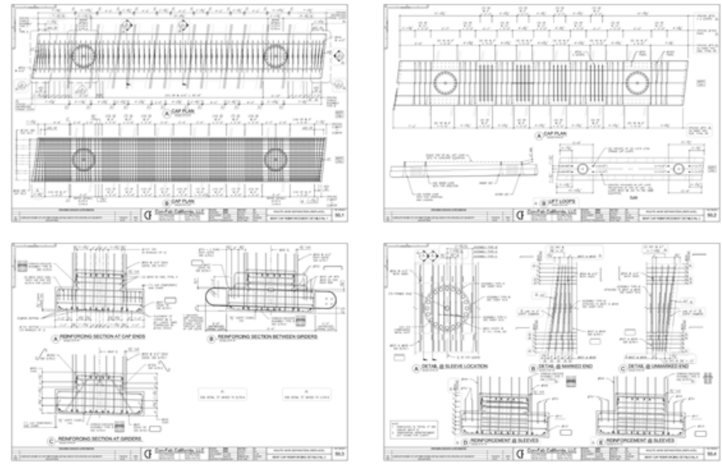
ROUTE 46/99 SEPARATION



shop drawings

INVERTED TEE BENT CAP FABRICATION DRAWINGS

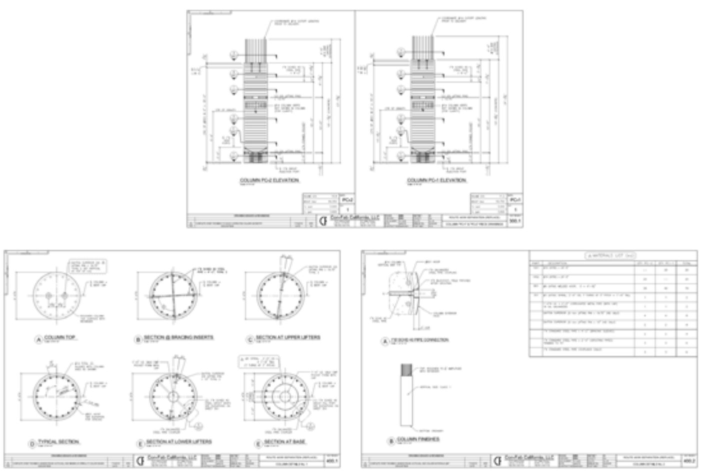
ROUTE 46/99 SEPARATION



shop drawings

INVERTED TEE BENT CAP SHOP DRAWINGS

ROUTE 46/99 SEPARATION



shop drawings

COLUMN SHOP DRAWINGS

ROUTE 46 / 99 SEPARATION

PRECAGED INVERTED TEE CAP REINFORCING **CAP POUR**

SOFFIT FORM CONSTRUCTED TO FINAL PROFILE GRADE

ROUTE 46 / 99 SEPARATION

INVERTED TEE BENT CAP
245,000 LBS

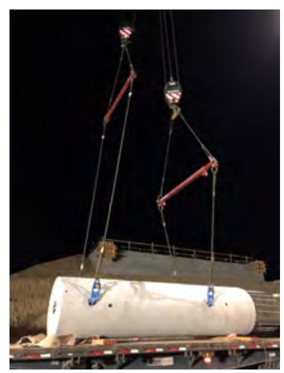
ROUTE 46 / 99 SEPARATION

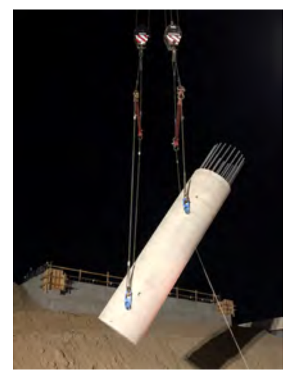
INVERTED TEE BENT CAP TO COLUMN FIT TEST

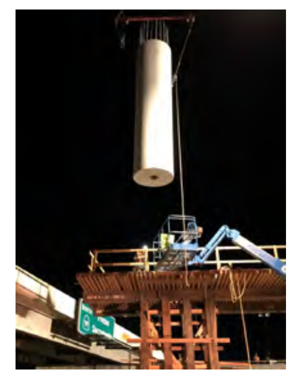
ROUTE 46 / 99 SEPARATION

INVERTED TEE CAP SECTION LOADED ONTO DUAL LANE TRAILER
(Ready for California Highway Patrol escort to jobsite)

ROUTE 46 / 99 SEPARATION



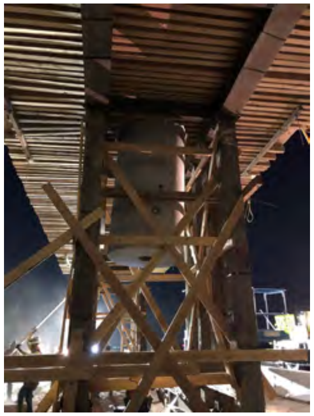




TWO-LINE COLUMNS OFFLOADING / TRIP TO VERTICAL

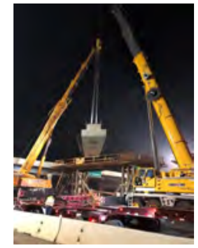
4’-6” dia x 24 ft Bridge Column x 50,000 lbs

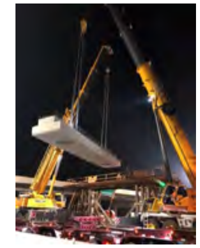
ROUTE 46 / 99 SEPARATION



COLUMN INSTALLATION **Layout lines on foundation and column**

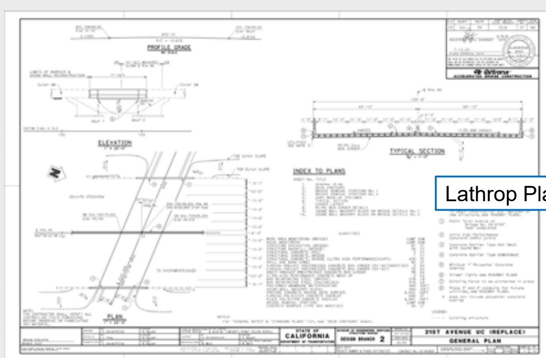
ROUTE 46 / 99 SEPARATION



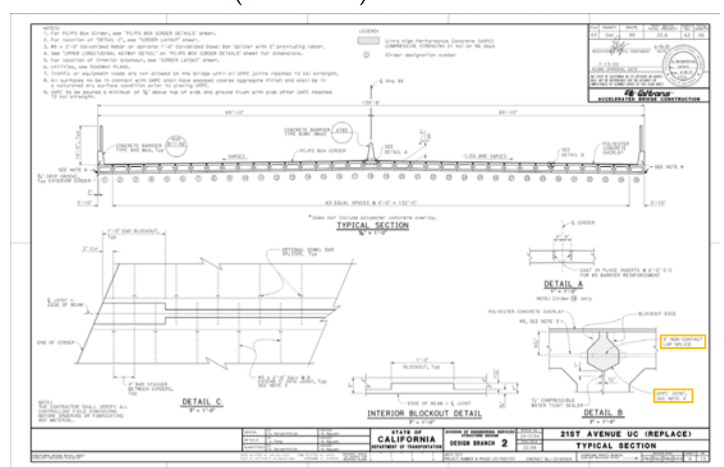


TWO CRANE CAP INSTALLATION
245,000 lbs

21ST AVENUE UC (REPLACE)



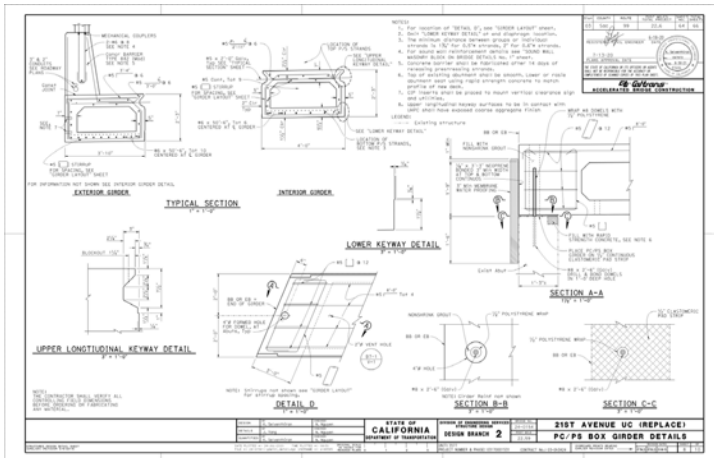
21ST AVENUE UC (REPLACE)



design drawings

5” non-contact lap splice of #5 keyway dowels spaced at 8”

21ST AVENUE UC (REPLACE)



design drawings

(33) Interior
48"W x 27"D x
51' L
x 43,400 lbs

(2) Exterior
46"W x 27"D x
51'L
with barrier
x 86,700 lbs

21ST AVENUE UC (REPLACE)

precast fabrication

- UHPC Keyway forming
- Lap spliced transverse reinforcement from adjacent box beams (photo during plant pre-assembly)

21ST AVENUE UC (REPLACE)

precast fabrication

21ST AVENUE UC (REPLACE)

fit testing

- Pre-delivery fit verification
- 139'-8" design width of (35) preassembled girders verified at precast plant

21ST AVENUE UC (REPLACE)

precast erection

- The existing superstructure of the 21st Avenue UC of CA-99 was demolished and reconstructed during a 100 hour total closure of the highway.
- The 2019 AADT at this site was 221,000 vehicles (both directions combined).
- The existing abutments were preserved.
- Con-Fab CA erected all 35 box beams using two 350 ton hydraulic truck cranes placed on Hwy 99, working outward from the bridge centerline.
- All girder erection completed within a 4 hours window on on 6/13/21.

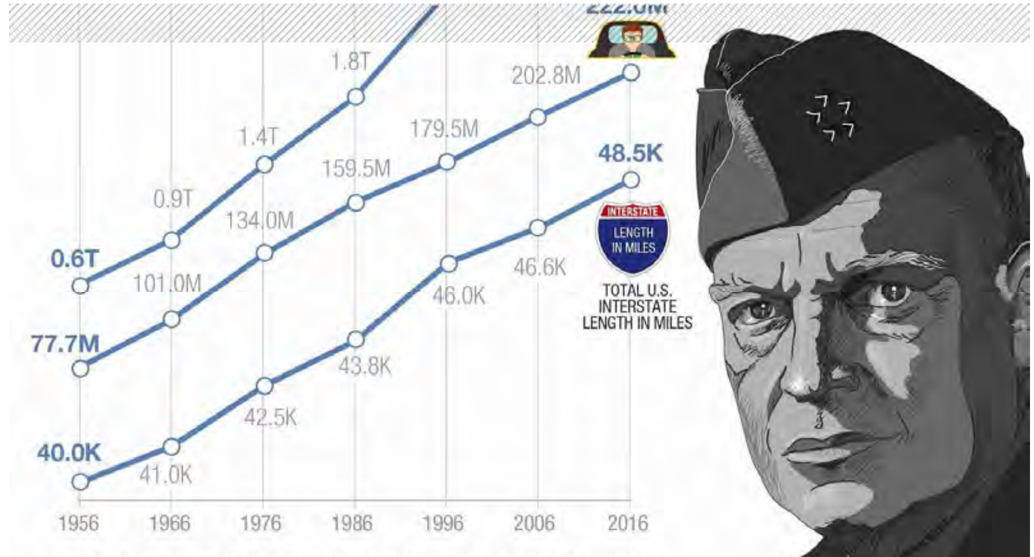
SAN MATEO BRIDGE HINGE SPAN REPLACEMENT (OCTOBER 2012)

THANK YOU! Questions?

HOW PRECAST BUILDS™

Precast-Prestressed Concrete Replacement Slabs

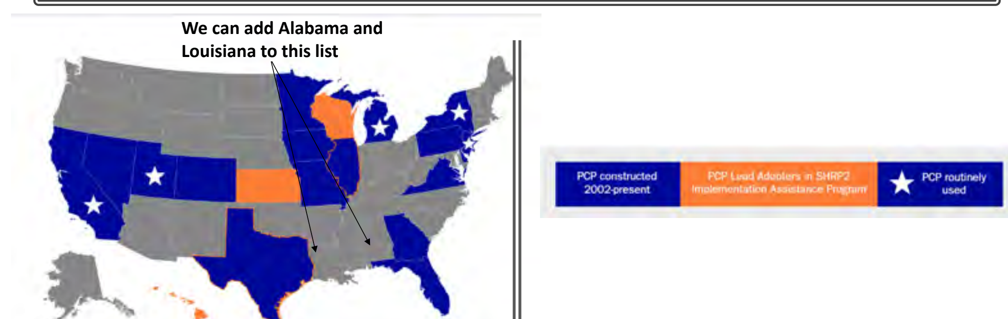
PRECAST IS VERSATILE, RESILIENT, EFFICIENT



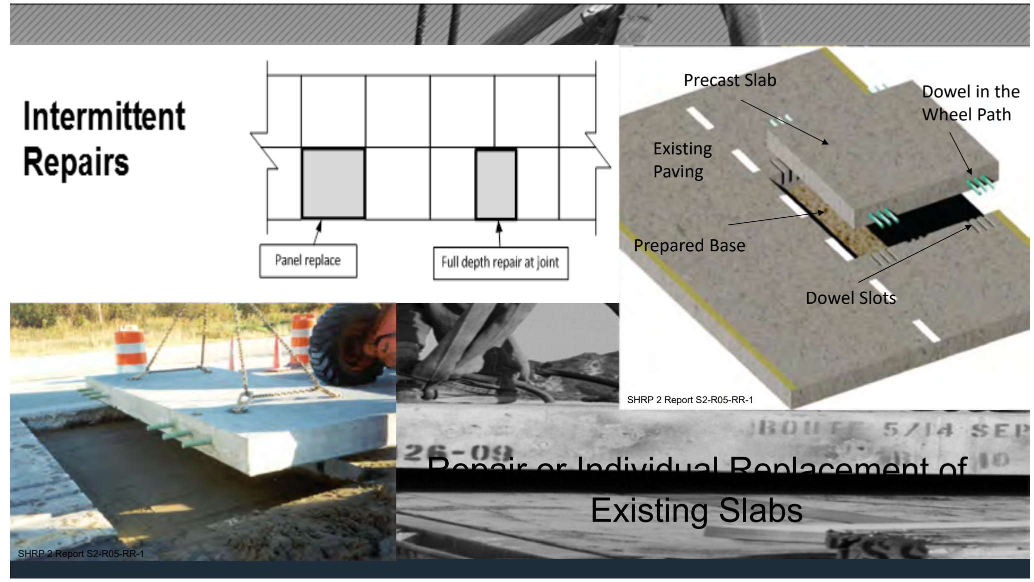
Why Precast-Prestressed Concrete Pavement?

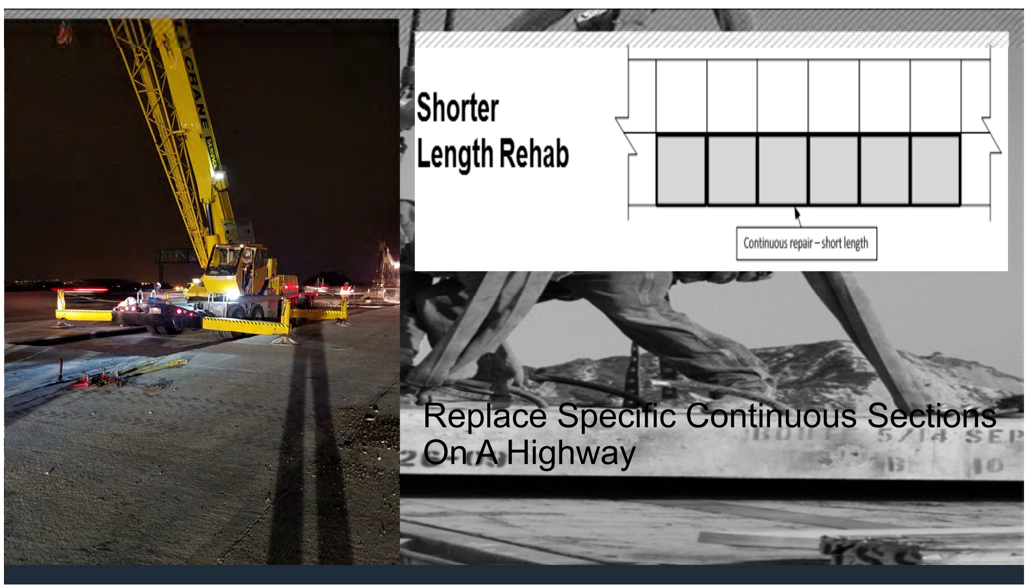
- Less maintenance over the lifespan
- Fast and rapid construction (opening to traffic) for overnight installations
- Less user delays
- Less worker exposure, less safety risks
- Inherent durability of a plant-produced concrete product in a quality-controlled environment.
- Materials savings- thinner slabs with prestressed panels-9in.
- Reduced number of working joints (with post tensioning)
- Takes up less space at the site.
- Accelerated Highway Construction
- FHWA is advocating for this technology: Get in, get out, and stay out

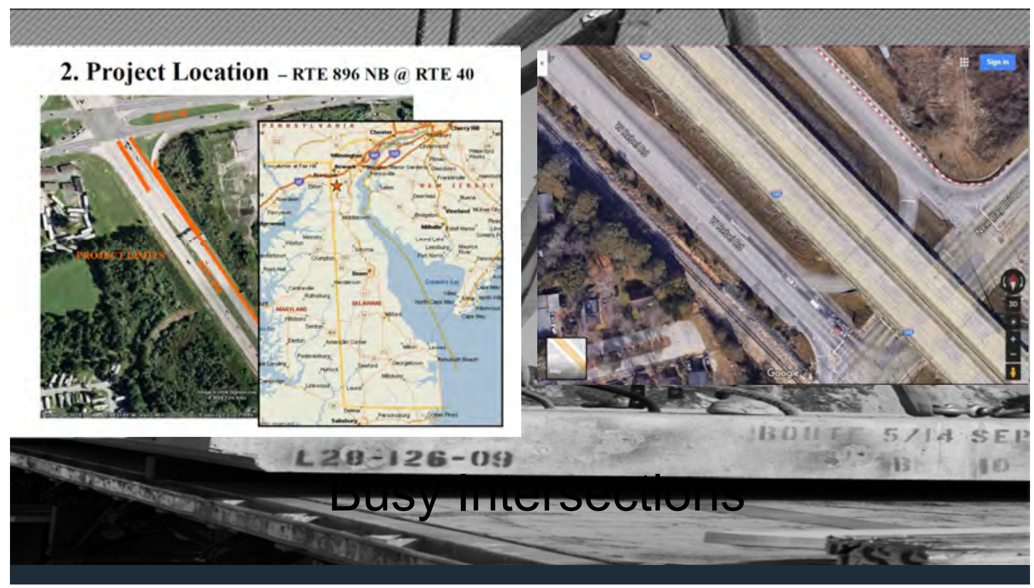
Who's Using Precast-Prestressed Concrete Pavement?




HOW PRECAST BUILDS
Precast Applications
PRECAST IS VERSATILE, RESILIENT, EFFICIENT


Intermittent Repairs
Panel replace
Full depth repair at joint
Precast Slab
Dowel in the Wheel Path
Existing Paving
Prepared Base
Dowel Slots
SHRP 2 Report S2-R05-RR-1
SHRP 2 Report S2-R05-RR-1
Repair or Individual Replacement of Existing Slabs


Shorter Length Rehab
Continuous repair – short length
Replace Specific Continuous Sections On A Highway


2. Project Location – RTE 896 NB @ RTE 40
PROJECT LIMITS
Busy Intersections


Replacement of Other Failed Material


Bridge Approaches


Multiple Lane Construction


Heavy Loads or Turn Lanes
OVERSIZE LOAD
WRONG WAY

New Construction


Photos: Tommy Nantung

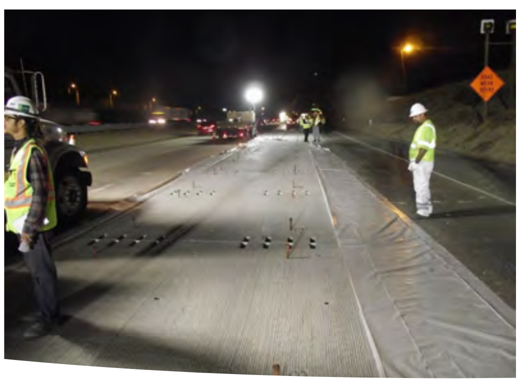
HOW PRECAST **BUILDS**

Design Considerations

PRECAST IS VERSATILE, RESILIENT, EFFICIENT

Design Considerations

Existing Conditions



- Heavy Traffic (Volume)
- Heavy Traffic (Loads)
- Existing Material
  - Concrete/Asphalt
- Sub-surface Conditions
  - Existing concrete
  - Soft soils-use of soil cement

Design Considerations

Bridges/Overpasses

Existing Structures

Design Considerations

Prestressed

Mild Steel Reinforcing

Design Considerations

Generic or Proprietary Systems

Design Considerations

Size of Panels

Panel Configuration

1 lane
2 lanes
2 lanes and a shoulder
Not longer than 40’

Design Considerations

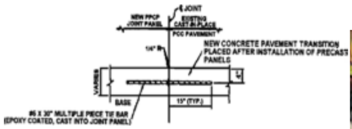
Leveling & Lifting Devices

Leveling rod

“Level Lifters” to speed installation and level exact heights

All transverse and longitudinal joints of the repair areas should be sealed in accordance with the agency's joint sealing practices.

Strategic Highway Research Program 2

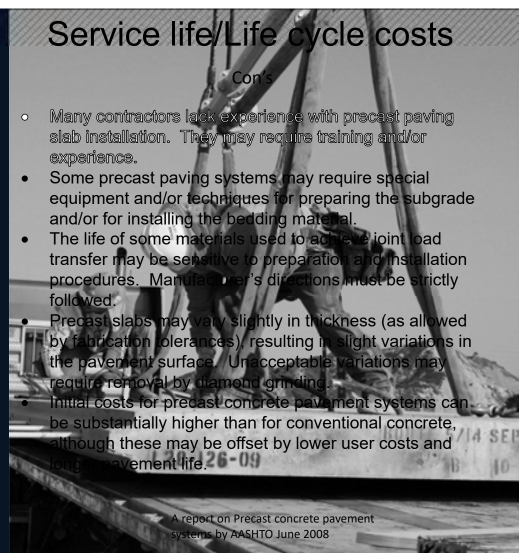


Design Considerations

Joints

Design Considerations

Grouting Pockets/Systems



Service life/Life cycle costs

Pro's

- No field cure time required for slab concrete – traffic can be carried immediately after placement.
- Slabs are fabricated and cured under controlled conditions, providing the potential for excellent durability and long service life.
- Control of curing conditions can virtually eliminate early-age cracking.
- High assurance that lanes will be opened to traffic on time because slabs are ready for traffic as soon as they are installed
- Since there is no finishing or formwork, placement requires only small crews.
- Slab installation activities are not greatly affected by weather conditions; construction can be accomplished in very hot, cold or rainy conditions without impacting slab performance or durability.
- Experience shows that short sections of precast concrete pavement can be installed in work windows of 5 hours or less.
- The potential for rapid installation and long pavement life minimizes short- and long-term user delay costs.
- Expected long pavement life and lower user costs can offset higher initial costs in life-cycle cost analyses

Con's

- Many contractors lack experience with precast paving slab installation. They may require training and/or experience.
- Some precast paving systems may require special equipment and/or techniques for preparing the subgrade and/or for installing the bedding material.
- The life of some materials used to achieve joint load transfer may be sensitive to preparation and installation procedures. Manufacturer's directions must be strictly followed.
- Precast slabs may vary slightly in thickness (as allowed by fabrication tolerances), resulting in slight variations in the pavement surface. Unacceptable variations may require removal by diamond grinding.
- Initial costs for precast concrete pavement systems can be substantially higher than for conventional concrete, although these may be offset by lower user costs and longer pavement life.

A report on Precast concrete pavement systems by AASHTO June 2008

Advantages of Precast Concrete

Sustainable

Precast prestressed products is a sustainable product produced using a durable material; concrete. So why is sustainability important?
The world population is using more of the Earth's natural resources than it can regenerate. Structures have an impact on the use of natural resources in two ways:

•The resources used to create and construct the structure
•The resources used to maintain and operate over time

Local materials
Local labor

No Waste:
Precast is manufactured using predetermined forms that reduce or eliminate concrete waste.

Resilient

Long lifespan

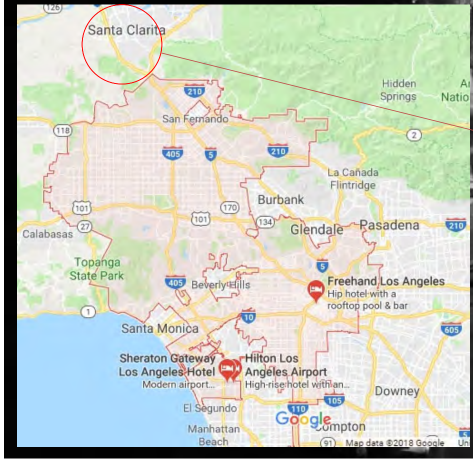
HOW PRECAST BUILDS™

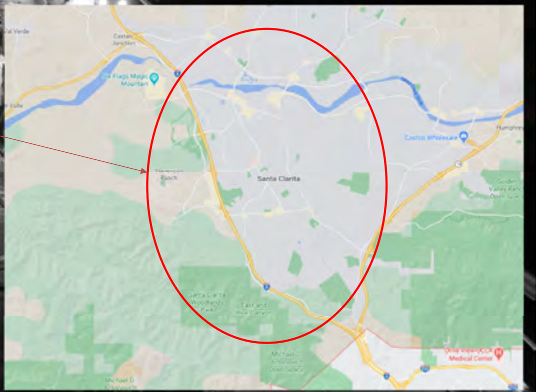
Case Study

PRECAST IS VERSATILE, RESILIENT, EFFICIENT

I-5 Precast-Prestressed Concrete Replacement Slab Project

Project site Northeast of Los Angeles

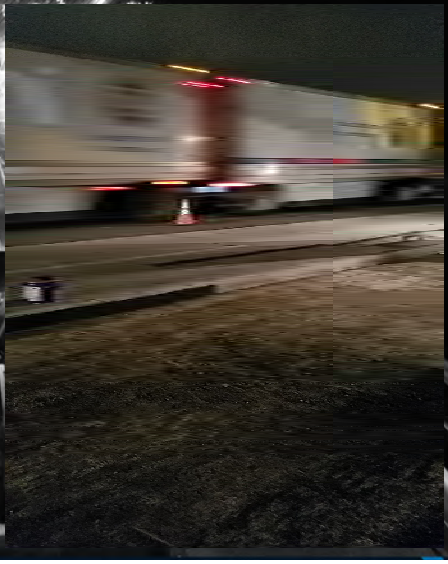




Typical Construction Sequence:

Road closure began at 8:00 PM
Sections were sawcut the night before
Removal of sections and disposal
Level/prep the base material
Slabs delivered to jobsite
Slab Installation
Slab leveling
Grouting

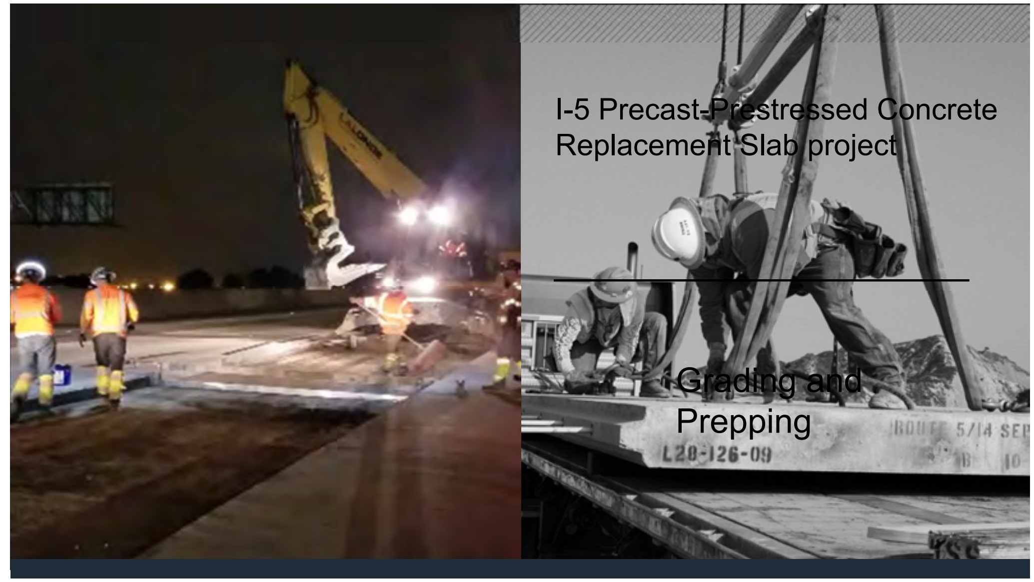
Road re-opened by 5:30 am

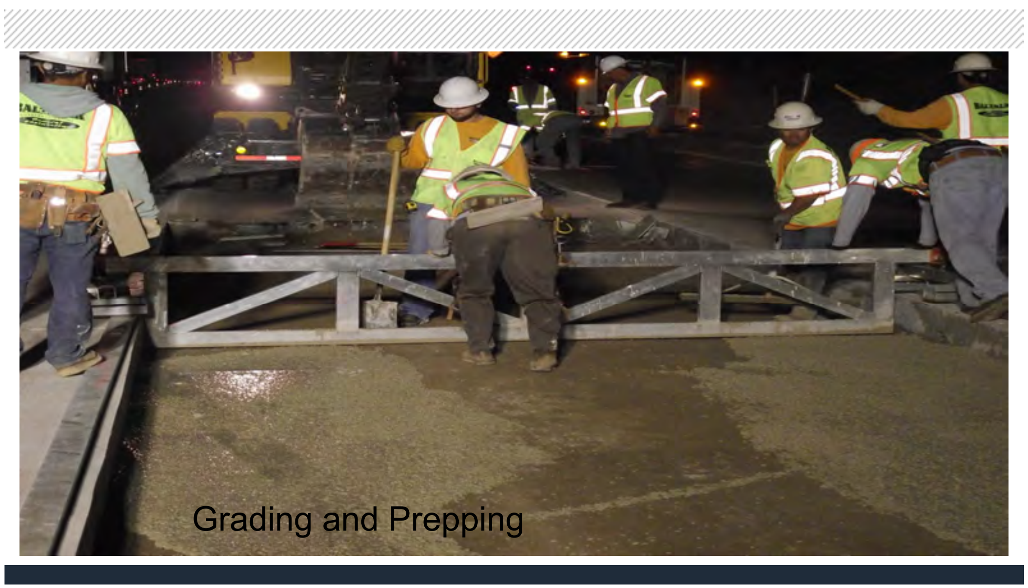


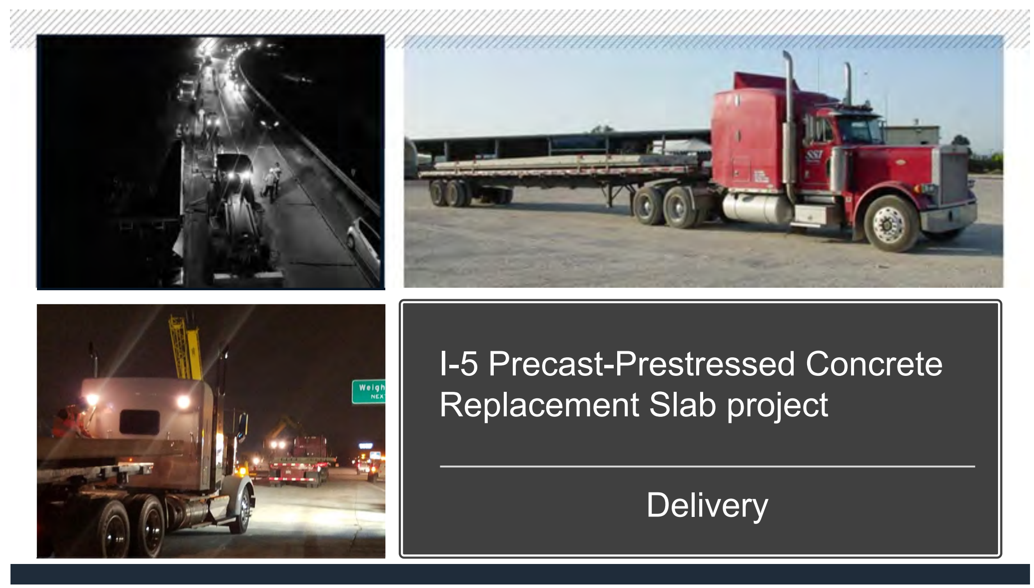
I-5 Precast-Prestressed Concrete Replacement Slab Project

Saw Cutting and Demolition


I-5 Precast-Prestressed Concrete Replacement Slab project
Removing Existing Roadway


I-5 Precast-Prestressed Concrete Replacement Slab project
Grading and Prepping

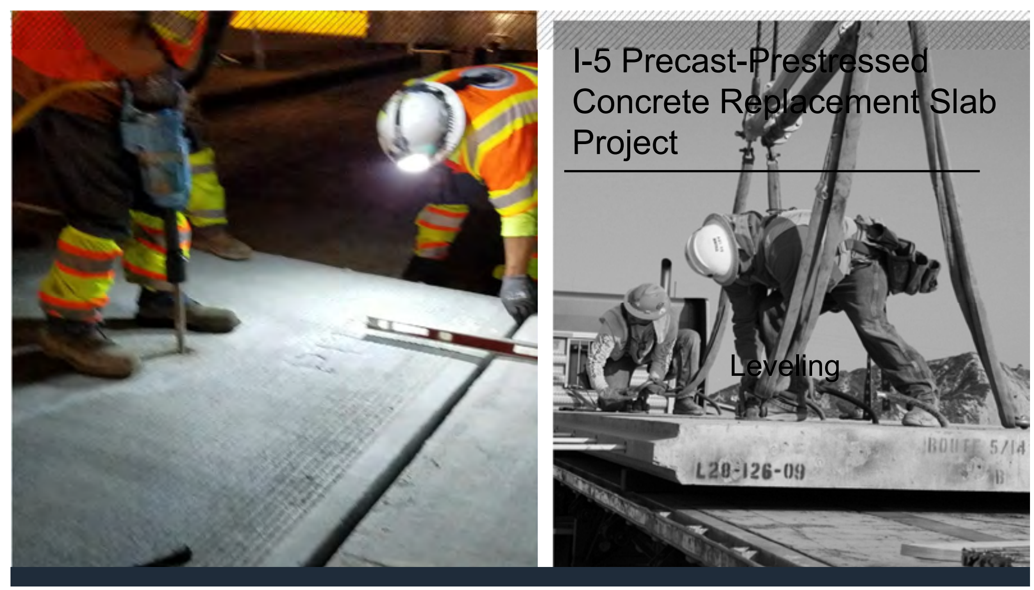

Grading and Prepping


I-5 Precast-Prestressed Concrete Replacement Slab project
Delivery


I-5 Precast-Prestressed Concrete Replacement Slab project
Setting Pavement Slabs


I-5 Precast-Prestressed Concrete Replacement Slab Project
Pavement Lifting Device


I-5 Precast-Prestressed Concrete Replacement Slab Project
Leveling


I-5 Precast-Prestressed Concrete Replacement Slab Project
Leveling



I-5 Precast-Prestressed Concrete Replacement Slab Project

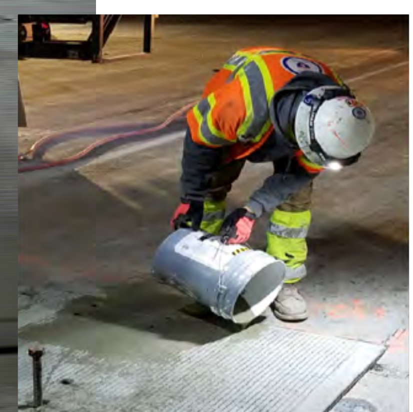
Slab Identification and Dowel Bars



I-5 Precast-Prestressed Concrete Replacement Slab Project

Grouting

I-5 Precast-Prestressed Concrete Replacement Slab Project



Grouting

I-5 Precast-Prestressed Concrete Replacement Slab Project

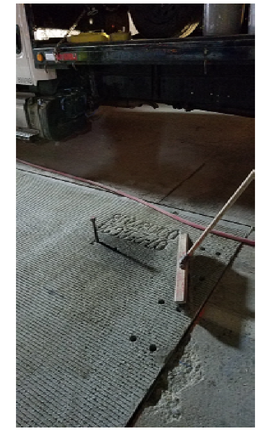
Production Rate

Typical Production Rates/Nighttime Closure

- Repairs: 15 to 20
- Continuous: 30 to 40
- Record is 60 panels (about 1000 ft)

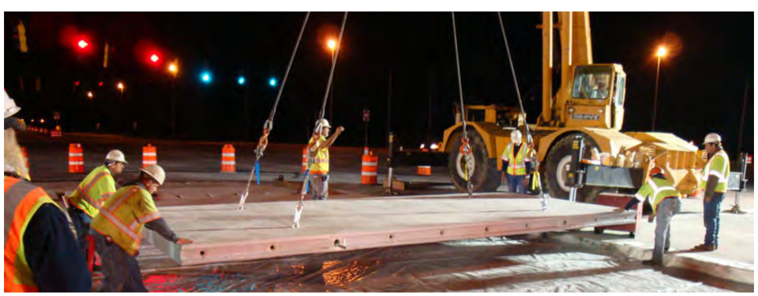
Summary

- State Highway Current Conditions- are poor, underfunded/overused aging road system
- Replacement Slab Applications:
  heavy use areas, existing concrete slab replacement, night closures-need for daytime operation
- Design Considerations:
  how many lanes/prestressed/post tensioned, leveling system, grouting system, full lane replacement,



Summary

- Construction joints, prestressing/mild steel, grouting pockets
- Nighttime closures, minimal lane closures.



"EARLY PRECASTER INVOLVEMENT"

The precaster can provide:

- Technical advice
- Engineering support
- Aesthetic guidance
- Economical solutions and product suggestions

Resources

- Visit the Prestressed/Precast Concrete Pavement Repository (sponsored by PCI, FHWA, APCA)
- www.precastconcretepavement.org

- References (PCI Guidance Documents, etc)
- PCI Pavement Committee
- www.pci.org

HOW PRECAST **BUILDS**

Thank You

Questions

PRECAST IS VERSATILE, RESILIENT, EFFICIENT