

# Prestressed Girder Fundamentals: Losses and Service Stresses

Precast, pretensioned concrete girders carry a large share of the short- and medium-span bridges in the United States. Designing, checking, rating or inspecting them takes three skills: understanding how the precompression gets into the concrete, estimating how much of it is lost over time, and checking concrete stresses at transfer and in service against the code limits. This course covers each in turn. It explains pretensioning and post-tensioning, defines the components of prestress loss, and works through the elastic shortening and approximate time-dependent loss equations of the AASHTO LRFD Bridge Design Specifications. It then reviews the stress limits for strand and concrete and explains why the Service III load combination, rather than Service I, governs tension in the bottom of a girder. The main sources are two public worked examples. One is the FHWA Comprehensive Design Example for Prestressed Concrete Girder Superstructure (FHWA-NHI-04-043), which uses an earlier LRFD edition. The other is Illinois DOT Bridge Design Guide 3.5, LRFD PPC Deck Beam Design (May 2019), which applies the 2017 eighth edition. Measured-versus-predicted loss data come from a 2021 PCI Journal study of AASHTO Type II–VI girders. AASHTO LRFD and the PCI Bridge Design Manual are not freely available. Equations and limits are therefore quoted as these public sources reproduce them, with their article numbers, and any example that is not taken from those sources is labeled illustrative.

## Learning objectives

- Explain pretensioning and post-tensioning
- Identify prestress loss components
- Check service stress limits
- Recognize Service III tension

## How prestressing works

Concrete is strong in compression and weak, and unreliable, in tension. Prestressing puts the concrete into compression ahead of time, so that the tension caused by dead and live load first cancels that stored compression before it can crack the concrete. In a simply supported girder the strands sit low in the section, below the centroid. Their force acts at an eccentricity  $e$ , so it both compresses the section ( $P/A$ ) and bends it upward ( $P \cdot e/S$ ). Before any external load is applied, the bottom fiber is in high compression and the top fiber is in low compression or slight tension. That is why a girder cambers upward when the strands are released.

## Pretensioning

In a pretensioned member, the strands are stressed between abutments of a casting bed before the concrete is placed. Once the concrete reaches the required release strength ( $f'_{ci}$ ), the strands are cut or detensioned. The steel tries to shorten back to its unstressed length and pulls on the concrete it is bonded to. The force builds up over a **transfer length** at each end. Pretensioning is the standard method for plant-cast AASHTO I-girders, bulb-tees, box beams and deck beams. In FHWA-NHI-04-043 the strands are straight, some are debonded near the ends to control end stresses, and the transfer length is 2.5 ft (60 strand diameters for 0.5-in. strand). Other designs drape (harp) part of the strands with hold-down devices so their eccentricity is smaller at the ends.

## Post-tensioning

In a post-tensioned member, ducts are cast into the concrete, and tendons are threaded through them and stressed against anchorages after the concrete has hardened. In bonded systems the ducts are then grouted. Post-tensioning is used for cast-in-place box girders, segmental construction, spliced girders and many retrofits. Because the tendon is pulled through a duct, friction along it causes losses that grow with distance from the jack, and wedges seating in the anchorage cause an anchorage-set loss near the jacking end. Tendon grouting, corrosion and inspection are covered in BR-P03. This course focuses on the pretensioned girder.

### Why losses matter

The force a strand delivers to the concrete is never the force the jack applied. The concrete shortens elastically at transfer, then creeps and shrinks for years, and the steel relaxes. Each of these reduces strand stress. The FHWA design example reached a total loss of about 19.6% of the stress immediately before transfer. The IDOT deck-beam example reached 14.8%. Underestimating losses leaves less precompression than the designer assumed, which can lead to cracking in the bottom flange under service loads. Overestimating them leads to more strands than needed, higher stresses at transfer and more camber than expected, a trade-off the PCI Journal study describes directly.

### Strand stress limits

AASHTO LRFD limits the stress in prestressing steel at two stages. For low-relaxation strand with tensile strength  $f_{pu} = 270$  ksi (yield  $f_{py} = 0.9f_{pu} = 243$  ksi), the FHWA example applies:

| Stage                                   | Limit (pretensioned, low-relaxation strand) | Value for 270-ksi strand |
|-----------------------------------------|---------------------------------------------|--------------------------|
| Immediately prior to transfer           | $0.75 f_{pu}$                               | 202.5 ksi                |
| At service limit state after all losses | $0.80 f_{py}$                               | 194.4 ksi                |

The FHWA example points out a detail that often causes confusion: the  $0.75f_{pu}$  limit applies to the stress *immediately before transfer*, not the jacking stress. Between jacking and release, the strand relaxes while the concrete is placed and cures ( $\Delta f_{pR1}$ ), and there are also jacking and seating losses at the bed. In the example's words, "the fabricator is responsible for determining the jacking force by adding the relaxation loss at transfer, jacking losses and seating losses to the Engineer-determined stress immediately prior to transfer." The example notes that state practice varies on which strand stress is shown on the plans. For reference, it estimates relaxation at transfer for one day at 2.08 ksi, which gives a jacking stress of about 204.6 ksi.

After all losses the example's effective stress was  $f_{pe} = 162.83$  ksi, well below the 194.4 ksi limit. The after-loss limit rarely governs a normal pretensioned girder. The stage that matters is the one before transfer, where the strand is stressed to near  $0.75f_{pu}$ .

### Components of prestress loss

The FHWA example divides losses into two groups: "Losses due to anchorage set, friction and elastic shortening are instantaneous. Losses due to creep, shrinkage and relaxation are time-dependent."

| Loss | Type | Cause | Applies to |
|------|------|-------|------------|
|------|------|-------|------------|

|                         |                |                                                                                           |                                                                                                                                     |
|-------------------------|----------------|-------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------|
| Anchorage set           | Instantaneous  | Wedges slip as they seat in the anchor                                                    | Mainly post-tensioning. In pretensioning it is handled by the fabricator at the bed                                                 |
| Friction                | Instantaneous  | Tendon drags on the duct (wobble and curvature)                                           | Post-tensioning. In pretensioning, only at hold-down devices for harped strands                                                     |
| Elastic shortening (ES) | Instantaneous  | Concrete shortens under the prestress at transfer, and the bonded strand shortens with it | Both. In post-tensioning, it comes from tendons stressed later shortening the member and reducing force in tendons already anchored |
| Shrinkage               | Time-dependent | Concrete loses moisture and shortens                                                      | Both                                                                                                                                |
| Creep                   | Time-dependent | Concrete continues to shorten under sustained compression                                 | Both                                                                                                                                |
| Relaxation              | Time-dependent | Stress in the steel drops under constant strain                                           | Both. Low-relaxation strand greatly reduces it                                                                                      |

For a pretensioned member stressed and placed in a single stage, the earlier-edition specification quoted in the FHWA example gives the total loss as  $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$ . The 2017 edition used by IDOT writes the same idea as  $\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$ , where  $\Delta f_{pLT}$  combines the long-term creep, shrinkage and relaxation losses.

The FHWA example adds that creep and relaxation losses depend on each other, and that a rigorous calculation "is seldom warranted or even possible at the design stage, since many of the factors are either unknown or beyond the designer's control." For that reason the specifications offer simpler estimates as well as refined ones.

## Elastic shortening

When the strands are released, the concrete at the level of the strand centroid shortens by  $f_{cgp}/E_{ci}$ . Because the strand is bonded, it shortens by the same strain, and its stress drops by that strain times  $E_p$ :

$$\Delta f_{pES} = (E_p / E_{ci}) \cdot f_{cgp}$$

Here  $f_{cgp}$  is the concrete stress at the center of gravity of the strands caused by the prestressing force at transfer plus the member's self-weight at the section of maximum moment.  $E_p$  is the strand modulus (28,500 ksi in both examples), and  $E_{ci}$  is the concrete modulus at transfer.

## The circularity, and how to handle it

$f_{cgp}$  depends on the force after transfer, which depends on  $\Delta f_{pES}$ . IDOT's design guide uses iteration. Following commentary C5.9.3.2.3a, it starts by assuming the force immediately after transfer is 90% of the force before transfer, computes  $f_{cgp} = F_t/A + F_t e^2/I - M_{be}/I$ , then  $\Delta f_{pES}$ , and checks the ratio  $(f_{pbt} - \Delta f_{pES})/f_{pbt}$ . In its deck-beam example the first pass gave 7.73 ksi and a ratio of 0.96, not the assumed 0.90. The second pass at 96% gave  $f_{cgp} = 1.30$  ksi and  $\Delta f_{pES} = (28,500/4,351)(1.30) = 8.52$  ksi, which agreed with the assumption.

The FHWA example instead uses the closed-form alternative in the commentary, which solves for  $\Delta f_{pES}$  directly from  $A_{ps}$ ,  $f_{pbt}$ ,  $I_g$ ,  $e$ ,  $A_g$ ,  $M_g$ ,  $E_{ci}$  and  $E_p$ . For its 44-strand girder it obtained 13.7 ksi. The strand stress after transfer was  $202.5 - 13.7 = 188.8$  ksi, and the force at transfer was 1,271 kips, an initial loss of 6.77%.

## Gross versus transformed sections

The FHWA example notes that calculating  $f_{cgp}$  from transformed section properties, which count the strand area at the modular ratio, is more accurate. Using gross concrete properties "is also acceptable, but will result in a higher concrete stress and, consequently, higher calculated losses." It also shows a consistency check: since the concrete stress before transfer is zero,  $f_{cgp}$  must equal  $\Delta f_{pES}/(E_p/E_{ci}) = 13.7/(28,500/4,200) \approx 2.02$  ksi, which matches its direct calculation. Some newer specification editions build the elastic gain or loss into the transformed-section approach. This course does not cover those provisions; see the flag about the current edition.

## Time-dependent losses: the approximate method

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For standard precast, pretensioned members, AASHTO LRFD provides an approximate lump estimate of the long-term loss. IDOT's guide reproduces it as Eq. 5.9.3.3-1 (2017 edition):

$$\Delta f_{pLT} = 10.0 (f_{pbt} A_{ps} / A_g) \gamma_h \gamma_{st} + 12.0 \gamma_h \gamma_{st} + \Delta f_{pR}$$

- $\gamma_h = 1.7 - 0.01H$ , the correction for average annual ambient relative humidity  $H$  (%)
- $\gamma_{st} = 5 / (1 + f'_{ci})$ , the correction for concrete strength at transfer (ksi)
- $\Delta f_{pR}$  = relaxation, taken as 2.4 ksi for low-relaxation strand

The first term is creep, scaled by how heavily the section is prestressed ( $f_{pbt}A_{ps}/A_g$  is the average precompression). The second term is mainly shrinkage. Both terms go down in humid climates ( $\gamma_h$  falls as  $H$  rises) and with stronger release concrete ( $\gamma_{st}$  falls as  $f'_{ci}$  rises).

### IDOT's worked numbers

For an IDOT deck beam with  $f_{pbt} = 201.96$  ksi,  $A_{ps} = 3.06$  in.<sup>2</sup>,  $A = 569.9$  in.<sup>2</sup>,  $H = 70\%$  and  $f'_{ci} = 5$  ksi:

- $\gamma_h = 1.7 - 0.01(70) = 1.0$
- $\gamma_{st} = 5/(1+5) = 0.833$
- $\Delta f_{pLT} = 10.0(201.96)(3.06)/569.9 \times 1.0 \times 0.833 + 12.0 \times 1.0 \times 0.833 + 2.4 = 21.43$  ksi
- $\Delta f_{pT} = 8.52 + 21.43 = 29.95$  ksi, or 14.8% of  $f_{pbt}$

### Limits of the approximate method

IDOT's guide notes that Article 5.9.5.3 restricts when the approximate method may be used, and in particular requires the refined method for non-composite members. It explains IDOT's policy. Deck beams with a 5-in. concrete wearing surface may use the approximate method because the department considers them partially composite. Deck beams with no wearing surface or a bituminous one also use the approximate method, but IDOT adds a correction of 2.5 to 4.5 ksi depending on beam depth "to more closely match the intent of the refined method." This is a useful reminder that agency policy can modify the code method, and that a load rater should check what assumption the original designer used.

## Concrete stress limits: at transfer

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Concrete stresses are checked at two main stages. The first is right after transfer: the concrete is youngest and weakest ( $f'_{ci}$ ), the prestress force is highest (only elastic shortening has occurred), and the only load fighting the prestress is the girder's own

weight. The second is in service after all losses, when the full dead and live load is acting and the prestress force is at its lowest.

The IDOT design guide (2017 edition) lists the temporary stress limits before losses:

| Check       | Limit (2017 edition, as quoted by IDOT)            | IDOT example, $f'_{ci} = 5.0$ ksi |
|-------------|----------------------------------------------------|-----------------------------------|
| Compression | $0.65 f'_{ci}$ (Art. 5.9.2.3.1a)                   | 3.25 ksi                          |
| Tension     | $0.24 \lambda \sqrt{f'_{ci}}$ (Table 5.9.2.3.1b-1) | 0.537 ksi                         |

$\lambda$  is the concrete density modification factor (1.0 for normal-weight concrete), and stresses are in ksi. Note that the  $0.24\sqrt{f'_{ci}}$  tension value is the limit that applies where bonded reinforcement is provided to carry the tension. The FHWA example describes it (in its older edition) as the limit "in areas with bonded reinforcement sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section." The FHWA example then sizes top-flange longitudinal bars for that tension force. The specification has a lower tension limit where no such reinforcement is provided.

### Edition changes you will meet in older calculations

The FHWA example was written for an earlier LRFD edition and uses  $0.60f'_{ci}$  for compression at transfer ( $-2.88$  ksi at  $f'_{ci} = 4.8$  ksi) and  $0.22\sqrt{f'_{ci}}$  for tension with reinforcement. IDOT's 2019 guide, written for the 2017 edition, uses  $0.65f'_{ci}$  and  $0.24\sqrt{f'_{ci}}$ . The article numbers also changed: the FHWA example cites S5.9.3 and S5.9.5 for losses and Table S5.9.4.2.2-1 for service tension, while IDOT cites 5.9.3.2.3a, 5.9.3.3 and Table 5.9.2.3.2b-1. When checking an existing girder's original design, or comparing a rating to the as-designed values, identify which edition the designer used before calling a stress "over the limit."

### Where transfer stresses govern

At midspan, the girder's self-weight moment offsets much of the prestress moment, so transfer stresses are usually moderate there. Near the ends, the self-weight moment approaches zero but the full prestress force is present from the end of the transfer length onward. The top fiber goes into tension and the bottom fiber into high compression. The FHWA example's table of transfer stresses shows this: its top-fiber tension peaks at 0.451 ksi at 1.75 ft from the bearing, while near midspan the top fiber is in slight compression. Designers control end stresses in three main ways:

- **Debonding** some strands near the ends with plastic sheathing, so they pick up force farther into the span. The FHWA example cites the earlier-edition guidance that no more than 25% of strands should be partially debonded and no more than 40% in any row. Its design used 27.2% and argued that "should" allows some deviation.
- **Harping (draping)** some strands upward toward the ends, reducing eccentricity where self-weight moment is small.
- **Top-flange bonded reinforcement** sized for the tension, which allows the higher tension limit.

## Concrete stress limits: in service and the role of Service III

After all losses, IDOT's guide checks compression with two Service I cases and tension with Service III (2017 edition, Table 5.9.2.3.2a-1 and 5.9.2.3.2b-1):

| Check           | Load case                                                           | Limit              | IDOT example, $f'_c = 6.0$ ksi |
|-----------------|---------------------------------------------------------------------|--------------------|--------------------------------|
| Compression (a) | Effective prestress + permanent loads + transient loads (Service I) | $0.60 \phi_w f'_c$ | 3.6 ksi                        |

|                 |                                                         |                            |           |
|-----------------|---------------------------------------------------------|----------------------------|-----------|
| Compression (b) | Effective prestress + permanent loads (Service I)       | $0.45 f'_c$                | 2.7 ksi   |
| Tension         | Service III, precompressed tensile zone, bonded tendons | $0.19 \lambda \sqrt{f'_c}$ | 0.465 ksi |

$\phi_w$  is a reduction factor for slender hollow sections and equals 1.0 for solid webs and for IDOT's standard sections. The FHWA example also checks a third compression case, live load plus one-half of effective prestress and permanent loads, at  $0.40f'_c$  under its edition. IDOT's guide applies  $0.40f'_c$  to the Fatigue I combination plus one-half the effective prestress and permanent loads (Art. 5.5.3.1).

## Why Service III?

The FHWA example puts it simply: "Maximum compression is checked under Service I limit state and maximum tension is checked under Service III limit state. The difference between Service I and Service III limit states is that Service I has a load factor of 1.0 for live load while Service III has a load factor of 0.8." Its load combination is Service III =  $1.0(\text{DC} + \text{DW}) + 0.8(\text{LL} + \text{IM})$ . The reduced live-load factor reflects the purpose of the check. It limits tensile stress in the precompressed tensile zone, the bottom flange of a simple span, under frequently occurring loads, so that cracking (and corrosion that might follow) is controlled. It is not meant to keep the girder crack-free under the rare heaviest truck, which the strength limit state covers.

Because tension in the bottom flange is usually the controlling design check for a pretensioned simple-span girder, the strand count is often set by Service III, not by strength. The FHWA example's Service III bottom-fiber stress at midspan was +0.267 ksi tension against a 0.465 ksi limit. IDOT's deck beam came to 0.286 ksi against 0.465 ksi.

## Corrosion exposure and agency policy

The  $0.19\sqrt{f'_c}$  value in both examples applies, in the FHWA example's words, to components "which include bonded prestressing tendons and are subjected to not worse than moderate corrosion conditions." The specification has stricter limits for severe corrosion exposure and for unbonded tendons. The FHWA example adds that many jurisdictions do not include creep and shrinkage restraint moments in continuous pretensioned girder design, and that "jurisdictions that do not include creep and shrinkage typically design the girders for a reduced tensile stress limit or for zero tension at final condition." Owners in coastal and deicing-salt environments often adopt zero-tension or reduced-tension policies, so check the agency bridge manual before applying the national limit.

## Composite section bookkeeping

Service stresses must be built up in the order the loads arrive. In the FHWA example's Service III calculation, the bottom-fiber stress is the sum of:

- Prestress after losses on the girder (non-composite) section:  $-P/A - P \cdot e/S_b$
- Girder, deck and haunch dead load, carried by the non-composite girder:  $+M_{\text{DNC}}/S_b$
- Barriers and future wearing surface, carried by the composite section:  $+M_{\text{DC}}/S_{bc}$
- Live load plus impact on the composite section with the 0.8 factor:  $+0.8M_{\text{LL+IM}}/S_{bc}$

Its numbers were  $-1.010 - 1.707 + 2.091 + 0.173 + 0.720 = +0.267$  ksi. A common mistake is applying deck weight to the composite section. The deck is wet and carries no load when it is placed, so its weight goes on the girder alone.

# Time-dependent losses: refined methods

The approximate method gives one number for the final long-term loss. The refined method estimates each mechanism separately and at specific stages, which is needed for camber at deck placement, for non-composite members (as IDOT notes) and for unusual cases.

## The 2017 refined method

The PCI Journal study quotes the 2017 refined method (Eq. 5.9.3.4.1-1), which splits the long-term loss at the time the deck is placed:

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

- Subscript *id*: from transfer to deck placement. Girder shrinkage, creep and strand relaxation.
- Subscript *df*: from deck placement to final time. Girder shrinkage, creep and relaxation, with the girder now acting compositely.
- $\Delta f_{pSS}$ : a *gain* in strand stress caused by shrinkage of the cast-in-place deck. As the deck shrinks it pulls the top of the composite section shorter, bending the girder so the bottom fiber lengthens slightly.

The study explains the physical effect of deck placement: "the weight of the plastic concrete causes girder deflection and increases tension in the strands (reduces prestress losses). When the concrete cures, the shrinkage of the hardened concrete deck also compensates for a small portion of the prestress losses."

This method is based on NCHRP Report 496 (Tadros et al., 2003), which the PCI paper cites. The detailed shrinkage and creep coefficient equations, with their factors for volume-to-surface ratio, humidity, strength and loading age, are in the specification. This course does not reproduce them.

## The earlier refined method (for reading older designs)

Many bridges in service were designed with the earlier LRFD refined equations, which the FHWA example uses. Rating engineers will see these in older calculations:

- Shrinkage:  $\Delta f_{pSR} = 17.0 - 0.15H$  (ksi). At  $H = 70\%$  (the example's Atlanta site): 6.5 ksi.
- Creep:  $\Delta f_{pCR} = 12.0f_{cgp} - 7.0\Delta f_{cdp} \geq 0$ , where  $\Delta f_{cdp}$  is the change in concrete stress at the strand centroid from permanent loads applied after transfer (diaphragms, slab, haunch, parapets, wearing surface). Example:  $12.0(2.016) - 7.0(1.10) = 16.49$  ksi.
- Relaxation after transfer, stress-relieved strand:  $\Delta f_{pR2} = 20.0 - 0.4\Delta f_{pES} - 0.2(\Delta f_{pSR} + \Delta f_{pCR})$ . Use 30% of that for low-relaxation strand. Example:  $0.3 \times 9.92 = 2.98$  ksi.
- Total:  $13.7 + 6.5 + 16.49 + 2.98 = 39.67$  ksi.  $f_{pe} = 202.5 - 39.67 = 162.83$  ksi.  $P_e = 44(0.153)(162.83) = 1,096$  kips, a 19.59% total loss.

Notice the  $-7.0\Delta f_{cdp}$  term. Superimposed dead load acts against the prestress at the strand level and reduces the creep loss, a simple version of the same physics as the deck-placement gain in the newer method.

## Comparing the two examples

|  |                                                   |                                           |
|--|---------------------------------------------------|-------------------------------------------|
|  | FHWA-NHI-04-043 (Type VI girder, earlier refined) | IDOT DG 3.5 (deck beam, 2017 approximate) |
|--|---------------------------------------------------|-------------------------------------------|

|                        |                                |                   |
|------------------------|--------------------------------|-------------------|
| Stress before transfer | 202.5 ksi                      | 201.96 ksi        |
| Elastic shortening     | 13.7 ksi                       | 8.52 ksi          |
| Long-term              | 25.97 ksi (6.5 + 16.49 + 2.98) | 21.43 ksi         |
| Total loss             | 39.67 ksi (19.6%)              | 29.95 ksi (14.8%) |

The difference comes mainly from the amount of precompression. The FHWA girder has 44 strands in a 1,085 in.<sup>2</sup> section, so  $f_{cgp}$  is about 2.0 ksi. The IDOT deck beam has about 1.30 ksi at the strand centroid. Elastic shortening and creep both scale with that stress.

## Predicted versus measured losses

Loss equations are estimates, and field data show they can be far off in either direction. The 2021 PCI Journal study instrumented nine full-scale AASHTO Type II, III, IV and VI girders from two Arkansas and Mississippi precast plants and tracked strand strain through fabrication, storage, erection and deck placement. Its findings:

- **Concrete was much stronger than specified.** Measured strength at transfer and at 28 days exceeded design strength by as much as 73%. The authors note that "fabricators overdesign the concrete mixture proportions to ensure that the required (design) compressive strength is achieved on time." The measured modulus at transfer averaged 32% above the value predicted by AASHTO Eq. C5.4.2.4-2.
- **Elastic shortening was overestimated** with design properties, because stiffer concrete shortens less. When actual properties were used, measured elastic shortening exceeded prediction by 2% to 18%.
- **Shrinkage varied.** It was close to prediction for Types II and III, but 48% and 44% above prediction for Types IV and VI. The authors observe that steam-cured girders shrink much less than water-cured ones, and the 2017 specification does not account for curing type.
- **Creep** measured on sealed cylinders was predicted well.
- **Total losses were overestimated.** At deck placement, design losses from the refined method exceeded measured losses by 146%, 86%, 71% and 14% for Types II, III, IV and VI. Using measured properties, the refined method still predicted about 61% and 33% more than measured for Types II and III, and 23% and 9% more for Types IV and VI.

The authors' recommendation is to coordinate with local precasters and use anticipated, not minimum specified, concrete properties when estimating losses and camber. They add that "the specified compressive strength in the shop drawings should not be adjusted." Applying their recommended properties cut the average overestimation to 30%, from 79% with design properties.

## What this means in practice

- **Camber.** Overestimated losses lead to underestimated prestress, so the expected camber is too low. Actual girders can arrive with more camber than predicted, which affects haunch depths and deck profile. The PCI paper notes that camber prediction "is further complicated" by differences in losses.
- **Design.** Overestimating losses is conservative for Service III bottom tension but unconservative for transfer stresses and camber.
- **Rating and evaluation.** A rater analyzing an existing girder usually uses the design losses. That is generally conservative for tension checks, but any rating based on measured behavior (such as load testing or camber surveys) should allow for this variability.

## Worked example: losses and stresses for a simple-span girder (illustrative)

All inputs in this example are illustrative values chosen for this course. They do not describe a standard section or a real bridge, and the arithmetic was done for this course. The equations are the 2017-edition forms quoted by IDOT.

### Given

- 40 low-relaxation 0.5-in. strands, 0.153 in.<sup>2</sup> each, so  $A_{ps} = 6.12 \text{ in}^2$ .  $f_{pbt} = 0.75(270) = 202.5 \text{ ksi}$ .  $E_p = 28,500 \text{ ksi}$ .
- Girder:  $A = 800 \text{ in}^2$ ,  $I = 260,000 \text{ in}^4$ ,  $y_b = 24 \text{ in}$ . ( $S_b = 10,833 \text{ in}^3$ ),  $y_t = 30 \text{ in}$ . ( $S_t = 8,667 \text{ in}^3$ ), strand eccentricity at midspan  $e = 20 \text{ in}$ .
- $f'_{ci} = 5.0 \text{ ksi}$ ,  $E_{ci} = 4,000 \text{ ksi}$ .  $f'_c = 8.0 \text{ ksi}$ .  $H = 60\%$ .
- Midspan moments: girder self-weight  $M_g = 1,200 \text{ k-ft}$ . Total non-composite dead load (girder + deck + haunch) = 2,600 k-ft. Composite superimposed dead load = 300 k-ft. LL+IM = 1,500 k-ft. Composite  $S_{bc} = 16,000 \text{ in}^3$ .

### 1. Elastic shortening (iterate)

$$F_i = 6.12 \times 202.5 = 1,239 \text{ kips.}$$

- Trial 1,  $F_t = 0.90F_i = 1,115 \text{ kips}$ :  $f_{cgp} = 1,115/800 + 1,115(20^2)/260,000 - (1,200 \times 12)(20)/260,000 = 1.394 + 1.716 - 1.108 = 2.00 \text{ ksi}$ .  $\Delta f_{pES} = (28,500/4,000)(2.00) = 14.3 \text{ ksi}$ . Ratio =  $(202.5 - 14.3)/202.5 = 0.93$ , not 0.90, so iterate.
- Trial 2,  $F_t = 0.93F_i \approx 1,152 \text{ kips}$ :  $f_{cgp} = 1.441 + 1.773 - 1.108 = 2.11 \text{ ksi}$ .  $\Delta f_{pES} = 15.0 \text{ ksi}$ . Ratio = 0.926.
- Trial 3,  $F_t = 0.926F_i \approx 1,148 \text{ kips}$ :  $f_{cgp} \approx 2.09 \text{ ksi}$ .  $\Delta f_{pES} \approx 14.9 \text{ ksi}$ . Converged.

### 2. Long-term loss (approximate method)

- $\gamma_h = 1.7 - 0.01(60) = 1.10$ .  $\gamma_{st} = 5/(1 + 5) = 0.833$ .  $\gamma_h \gamma_{st} = 0.917$ .
- Average precompression  $f_{pbt} A_{ps}/A = 202.5(6.12)/800 = 1.549 \text{ ksi}$ .
- $\Delta f_{pLT} = 10.0(1.549)(0.917) + 12.0(0.917) + 2.4 = 14.2 + 11.0 + 2.4 = 27.6 \text{ ksi}$ .
- Total  $\Delta f_{pT} = 14.9 + 27.6 = 42.5 \text{ ksi}$  (21%).  $f_{pe} = 160.0 \text{ ksi}$ , which is below  $0.80f_{py} = 194.4 \text{ ksi}$ , so OK.  $P_e = 6.12 \times 160.0 = 979 \text{ kips}$ .

### 3. Transfer stresses

- Limits: compression  $0.65(5.0) = 3.25 \text{ ksi}$ . Tension, with bonded reinforcement,  $0.24\sqrt{5.0} = 0.537 \text{ ksi}$ .
- Midspan bottom:  $-1,148/800 - 1,148(20)/10,833 + 1,200(12)/10,833 = -1.435 - 2.119 + 1.329 = -2.22 \text{ ksi}$  compression. OK.
- End region (beyond the transfer length, where  $M_g \approx 0$ ), top fiber, with all strands bonded and straight:  $-1.435 + 1,148(20)/8,667 = -1.435 + 2.649 = +1.21 \text{ ksi}$  tension. That is more than double the 0.537 ksi limit, so the designer must debond or harp strands near the ends. The bottom fiber at the same section is  $-1.435 - 2.119 = -3.55 \text{ ksi}$ , which also exceeds 3.25 ksi in compression.

### 4. Service III bottom tension at midspan

- Prestress:  $-979/800 - 979(20)/10,833 = -1.224 - 1.807 = -3.03 \text{ ksi}$
- Non-composite DL:  $+2,600(12)/10,833 = +2.88 \text{ ksi}$
- Composite SDL:  $+300(12)/16,000 = +0.23 \text{ ksi}$

- Live load:  $+0.8 \times 1,500(12)/16,000 = +0.90$  ksi
- Total: +0.98 ksi tension, against a limit of  $0.19\sqrt{8.0} = 0.537$  ksi. **Not OK.** Under Service I ( $1.0 \times LL$ ) the stress would be +1.20 ksi, which shows why using Service I for tension would be overly severe.

## What the designer would do

Add strands or lower their centroid to increase  $P_e$  and  $e$ , choose a deeper section, or reduce girder spacing. Each extra strand also raises the transfer stresses and the creep loss, so prestressed design is iterative. That is why the FHWA example says its strand pattern was "refined using trial and error until a pattern produced stresses, at transfer and under service loads, that fell within the permissible stress limits."

## Reading prestressed girders in the field and in ratings

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### For the load rater

- **Know the design basis.** Losses, stress limits and even load combinations have changed between editions. The two public examples alone differ on transfer limits ( $0.60f'_{ci}$  versus  $0.65f'_{ci}$ ) and on the loss method. Older girders designed under the AASHTO Standard Specifications used other methods altogether.
- **Service checks in rating.** Prestressed girders are commonly checked at a service limit state for tension in the precompressed zone, alongside the strength check. Rating procedures, factors and the treatment of losses for existing members are covered in BR-L01 and BR-L04.
- **Damage changes everything.** Broken or corroded strands reduce  $P_e$  and  $e$  directly. Each lost strand removes both precompression and moment capacity. Rating a damaged girder means recalculating stresses with the remaining strands. BR-P02 covers damage evaluation and repair.

### For the inspector

The mechanics in this course explain what inspectors see:

- **Flexural cracks in the bottom flange near midspan** mean the precompressed tensile zone has gone into tension beyond its cracking stress. In a girder designed to the Service III limit, that should not happen under normal traffic. Possible causes are lost prestress (strand corrosion or damage), overloads, or design or fabrication problems. These cracks call for engineering follow-up, not just a note.
- **End-region cracking** (horizontal and inclined web cracks near the girder ends) relates to the high transfer forces at the ends. Some appears at the plant at release. Inspectors should record its width and pattern and look for rust staining.
- **Longitudinal cracks along strand lines** with rust staining in the bottom flange suggest strand corrosion. That is a direct threat to the prestress force.
- **Unusual camber or sag.** Loss of camber over time, or a visible sag, can indicate prestress loss or damage. Camber variability between girders is common at erection, as the PCI study's discussion of losses and camber suggests.

Concrete deck and superstructure inspection procedures are covered in BR-I06.

## Common mistakes and quick checks

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- **Applying the  $0.75f_{pu}$  limit to the jacking stress** instead of the stress immediately before transfer. The fabricator adds relaxation and seating losses to reach the jacking stress.

- **Skipping the elastic shortening iteration** or assuming 10% without checking. The IDOT example converged at 4% (ratio 0.96), not 10%.
- **Mixing editions**, for example using a current loss equation with an older stress table or article reference.
- **Wrong section for each load**, such as putting deck weight on the composite section.
- **Using Service I for tension**, or Service III for compression.
- **Forgetting the end regions**. Midspan rarely controls transfer stresses. Check at the transfer length and at debonding termination points.
- **Ignoring agency policy** on zero tension, corrosion exposure or loss corrections, such as IDOT's deck-beam correction of 2.5 to 4.5 ksi.

## Quick sanity checks

- Total losses for typical pretensioned girders in the two public examples fell between roughly 15% and 20% of the stress before transfer. A result far outside that range deserves a second look, though unusual members can legitimately differ.
- Elastic shortening  $\approx (E_p/E_{ci}) \times f_{cgp}$ , and  $E_p/E_{ci}$  is usually about 6 to 7 for typical release strengths. With  $f_{cgp}$  of 1 to 2 ksi, expect about 7 to 15 ksi.
- Service III bottom stress is the small difference between large numbers: about  $-3$  ksi from prestress and about  $+3$  to  $+4$  ksi from loads. Small errors in section properties or moments therefore matter, and should be checked carefully.

## Transfer length, development length and the effective prestress

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Effective prestress,  $f_{pe}$ , is used for more than service stresses. It also sets how far a pretensioned strand must be embedded before it can carry its full stress. A strand in a pretensioned girder transfers its force to the concrete only through bond, so the prestress builds up gradually from the girder end.

### Transfer length

The FHWA example (earlier edition) takes the transfer length as 60 strand diameters (S5.11.4.1). For 0.5-in. strand that is  $60(0.5) = 30$  in., or 2.5 ft. The example states that  $f_{pe}$  "may be assumed to vary linearly from 0.0 at the point where bonding commences to a maximum at the transfer length." This is why the critical transfer-stress section is near the end of the transfer length, not at the girder end itself: the full prestress has just arrived there, and self-weight moment is still small. It is also why the FHWA example's transfer-stress table starts at 1.75 ft from the bearing, where its top-fiber tension was 0.451 ksi against its 0.48 ksi limit ( $0.22\sqrt{4.8}$ ).

### Development length

To develop the stress  $f_{ps}$  that the strand reaches at nominal flexural resistance, more embedment is needed. The FHWA example uses:

$$l_d \geq \kappa[f_{ps} - (2/3)f_{pe}]d_b \text{ (S5.11.4.2-1)}$$

$\kappa$  is 1.6 for fully bonded strands and 2.0 for partially debonded strands (S5.11.4.2 and S5.11.4.3, earlier edition). With  $f_{ps} = 264.4$  ksi and  $f_{pe} = 162.83$  ksi:

- Fully bonded:  $1.6[264.4 - (2/3)(162.83)](0.5) = 124.7$  in. (10.39 ft)
- Debonded:  $2.0[264.4 - (2/3)(162.83)](0.5) = 155.8$  in. (12.98 ft), measured from the point where bonding begins

Note that higher  $f_{pe}$ , meaning lower losses, shortens the development length. The losses calculation feeds directly into the strand anchorage check.

Between transfer and development length, the example says the strand stress "may be assumed to increase in a parabolic manner," and that "many jurisdictions assume that the stress increases linearly between the transfer and the development lengths." The example uses the linear assumption.

## Why this matters beyond design

- **Shear and flexure near supports.** Near the girder ends, the strands have not reached full stress. Flexural capacity and the longitudinal reinforcement check for shear must use the reduced strand stress within the development length.
- **Debonding points.** Each debonded group starts its own transfer and development length where bonding begins. The FHWA example's debonding layout staggers these points so that strand force builds up gradually, consistent with its 40%-per-row limit.
- **Damage near ends.** Strand corrosion at a girder end, often beneath leaking deck joints (see BR-B02), affects the zone where strands are still developing their force. A loss of bond or section there can reduce shear and flexural capacity more than the same damage at midspan.
- **Cut-off or damaged ends.** If a girder end has to be cut back or is damaged by impact, the transfer and development lengths start from the new bonded end. The capacity near that end needs to be re-evaluated.

## Putting it together: the design and check sequence

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Pulling together the steps in the IDOT and FHWA examples, a typical pretensioned simple-span girder check goes as follows:

1. **Section and materials.** Girder shape,  $f'_{ci}$  and  $f'_c$ , strand size and grade, and moduli. Use  $E_{ci}$  at release for elastic shortening.
2. **Loads and moments.** Separate non-composite loads (girder, deck, haunch, diaphragms) from composite loads (barriers, wearing surface, live load with distribution factors and impact).
3. **Trial strand pattern** from experience or design tables. The FHWA example notes that jurisdictions often keep span-and-spacing tables to pick the girder size.
4. **Elastic shortening** by iteration on  $F_t$ .
5. **Long-term losses** by the approximate method, or by the refined method where needed or required by the agency. Include any agency correction.
6. **Check strand stress after losses** against  $0.80f_{py}$ .
7. **Transfer stresses** at the transfer length, at debonding and harp points, and at midspan. Adjust with debonding, harping or top reinforcement.
8. **Service stresses.** Service I compression under all loads and under permanent loads, Service III tension in the precompressed zone, and the fatigue-case compression check.
9. **Strength I flexure and shear.** IDOT's combination is  $1.25DC + 1.5DW + 1.75(LL + IM)$ , with 33% dynamic load allowance on the design truck. This includes development length near the ends.
10. **Camber and deflection** for haunch and profile, recognizing the uncertainty the PCI study measured.

The sequence is iterative. A Service III failure leads to more strands, which raises transfer stresses and may require more debonding, which in turn changes development length and end-region capacity.

## Key numbers from the public sources

| Item                                            | Value                                        | Source                 |
|-------------------------------------------------|----------------------------------------------|------------------------|
| Stress before transfer (270-ksi low-relaxation) | $0.75f_{pu} \approx 202.5 \text{ ksi}$       | FHWA; IDOT             |
| After all losses                                | $\leq 0.80f_{py}$                            | FHWA; IDOT             |
| Transfer compression                            | $0.65f'_{ci}$ (0.60 in the earlier edition)  | IDOT; FHWA             |
| Transfer tension, with bonded reinforcement     | $0.24\lambda\sqrt{f'_{ci}}$ (0.22 earlier)   | IDOT; FHWA             |
| Service compression                             | $0.60\phi_w f'_c$ ; $0.45f'_c$ ; $0.40f'_c$  | IDOT                   |
| Service III tension, moderate exposure          | $0.19\lambda\sqrt{f'_c}$                     | IDOT; FHWA             |
| Transfer length                                 | $60d_b$                                      | FHWA (earlier edition) |
| Debonding guidance                              | $\leq 25\%$ of strands; $\leq 40\%$ in a row | FHWA (earlier edition) |

Confirm all values against the edition of the AASHTO LRFD Bridge Design Specifications that governs your project.

## Glossary of terms used in this course

- $f_{pbt}$ : strand stress immediately before transfer. For low-relaxation 270-ksi strand it is limited to  $0.75f_{pu}$ .
- $f_{pj}$ : jacking stress. It is higher than  $f_{pbt}$  by the seating and early-relaxation losses that happen in the bed before release.
- $f_{pe}$ : effective strand stress after all losses.
- $f_{cgp}$ : concrete stress at the strand centroid caused by prestress and self-weight just after transfer. It drives elastic shortening and creep.
- $\Delta f_{cdp}$ : change in concrete stress at the strand centroid from permanent loads added after transfer. It appears in the earlier refined creep equation.
- $\gamma_h, \gamma_{st}$ : humidity and concrete-strength correction factors in the approximate long-term loss equation.
- **Precompressed tensile zone**: the part of the section that prestress puts into compression and that service loads put into tension. For a simple span, this is the bottom flange near midspan.
- **Transfer length**: the bonded length over which the strand transfers  $f_{pe}$  to the concrete.
- **Development length**: the bonded length needed to develop  $f_{ps}$  at nominal flexural resistance.
- **Debonding (shielding)**: sheathing strands near the girder ends so they carry no force over that length.
- **Harping (draping)**: deflecting strands upward toward the ends to reduce eccentricity where self-weight moment is small.

## Key takeaways

- Prestress puts the concrete into compression in advance; strand eccentricity adds an upward moment that offsets dead and live load.
- Instantaneous losses are anchorage set, friction and elastic shortening; time-dependent losses are creep, shrinkage and relaxation.
- Elastic shortening is  $\Delta f_{pES} = (E_p/E_{ci})f_{cgp}$ ; iterate on the force after transfer or use the closed-form commentary equation; gross section properties overestimate the loss.

- The approximate long-term loss (2017 edition Eq. 5.9.3.3-1) depends on the average precompression, humidity ( $\gamma_h$ ) and release strength ( $\gamma_{st}$ ), plus 2.4 ksi relaxation for low-relaxation strand.
- Strand stress is limited to  $0.75f_{pu}$  immediately before transfer and  $0.80f_{py}$  after losses; the fabricator sets the jacking stress.
- Check transfer stresses at  $f'_{ci}$ , service compression under Service I, and bottom-fiber tension under Service III (0.8 on live load), against the limits in the governing edition.

## References

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1. AASHTO, LRFD Bridge Design Specifications, 9th Edition, 2020
2. PCI, Bridge Design Manual, 3rd Edition (with revisions)
3. FHWA, Comprehensive Design Example for Prestressed Concrete (PSC) Girder Superstructure Bridge with Commentary, US Customary Units, FHWA-NHI-04-043 (2003), [fhwa.dot.gov/bridge/lrfd/fhwanhi04043.pdf](http://fhwa.dot.gov/bridge/lrfd/fhwanhi04043.pdf)
4. Illinois DOT, Bridge Manual Design Guide 3.5 — LRFD PPC Deck Beam Design (May 2019, updated for the 2017 AASHTO LRFD 8th Edition)
5. Dang, C. N., et al., 'Investigation of measured prestress losses compared with design prestress losses in AASHTO Types II, III, IV, and VI girders,' PCI Journal 66(3), May–June 2021

Study text only; the final quiz is taken online. Verify all references against the edition your agency has adopted.