Lateral stability and concrete strength requirements for precast, prestressed concrete components

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■ This paper is intended to reconcile the interaction between the temporary concrete compressive stresses traditionally used to determine the required concrete strength and the requirements for lateral stability, primarily the additional stresses due to lateral bending, for precast, prestressed concrete components.

■ This paper applies primarily to I-girders and bulb tees used in transportation structures and provides proposed revisions to articles related to temporary stresses from the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.

■ Washington State Department of Transportation policies are also discussed within this paper, and a design example is provided based on a viaduct replacement project in Seattle, Wash.

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The allowable level of temporary concrete compressive stress in precast, prestressed concrete components has been a source of debate in the concrete industry for many years. Traditionally, these stresses have been considered to originate only from the effects of prestress combined with the self-weight of a plumb component evaluated about the major axis. The maximum compressive stress divided by the coefficient of the compressive stress limit determines the required concrete strength. Although these temporary stresses can occur at any time from fabrication through erection into the structure, the critical case is usually at transfer of prestress and subsequent lifting from the form. At this stage, the prestress force is higher and the concrete strength is lower than at any other point in the life of the component. At this early age, concrete is also more susceptible to damage from high compressive stress.

As materials and fabrication capabilities in the precast, prestressed concrete industry advance, components are becoming longer and slenderer, particularly within the transportation sector. Such components require serious consideration of lateral stability during handling and shipping, which introduces bending about the minor axis. This lateral bending will increase maximum tensile and compressive stresses at the extremities of the component. These localized stresses have traditionally not been used to determine the required concrete strength, and doing so at current stress limits can significantly increase the required concrete strength.

This paper is intended to reconcile the interaction between the temporary concrete compressive stresses traditionally used to determine the required concrete strength and the requirements for lateral stability, primarily the additional stresses due to lateral bending. The assumption is made that compressive stresses govern the determination of the required concrete strength. It is normally not efficient to determine the required concrete strength based on tensile stresses, which can be satisfied in a different manner. Because this paper applies primarily to I-girders and bulb tees used in transportation structures, the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[1] terminology and notation will be used where appropriate. Washington State Department of Transportation (WSDOT) policies are also discussed within this paper, and a design example is provided based on a viaduct replacement project in Seattle, Wash.

Compressive stress limits

At transfer of prestress, both AASHTO and the American Concrete Institute (ACI) permitted concrete compressive stresses up to $0.60 f'_{ci}$ for decades. However, as discussed in Birrcher et al.,[2] a preponderance of research evidence suggests that this compressive stress limit is overly conservative. As a result, in its eighth edition (2017), the AASHTO LRFD specifications increased the concrete compressive stress limit at transfer of prestress to $0.65 f'_{ci}$ for the entire length of the component. Beginning with *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08)*,[3] the concrete compressive stress limit at transfer of prestress was increased to $0.70 f'_{ci}$ at the ends of simply supported components, while $0.60 f'_{ci}$ was maintained for the remainder of the component length. Presumably this was done to reduce the need for debonding or draping of pretensioned strands. The revisions made to the AASHTO LRFD specifications[1] and ACI 318[3] were consistent with the conclusions of past research.

It is important to note that the limit on concrete compressive stress is based on performance of the component in service, not the potential crushing of concrete due to temporary stresses. High compressive stress can cause microcracking in early-age concrete, potentially increasing the camber in an unpredictable fashion and reducing the modulus of rupture, leading to premature flexural cracking in service.[2] The referenced research opined that the prediction of camber was generally not affected by an increase in allowable compressive stress. With respect to the reduction in modulus of rupture, **Fig. 1** (reproduced from Birrcher et al.[2]) shows a comparison of the predicted flexural cracking load with the level of allowable compressive stress for a variety of section shapes tested in the lab. On the vertical axis, an accuracy of zero indicates that the predicted cracking load matched the measured cracking load. Negative accuracy values indicate that the specimens cracked before reaching the predicted cracking load. The red vertical dashed line indicates a compressive stress limit of $0.65 f'_{ci}$, which comfortably fits the data, but an argument can be made that the solid red vertical line at $0.70 f'_{ci}$ is also justifiable.

It is also important to note that the compressive stress limits proposed in this paper apply to temporary stresses from transfer of prestress through erection into the structure. Temporary stresses are distinguished from transient stresses in that temporary stresses occur only a handful of times during the pre-service life of a component. Transient stresses, which include the effects of live loads in service, may occur on a daily basis and logically should have a lower compressive stress limit ($0.60 f'_c$).

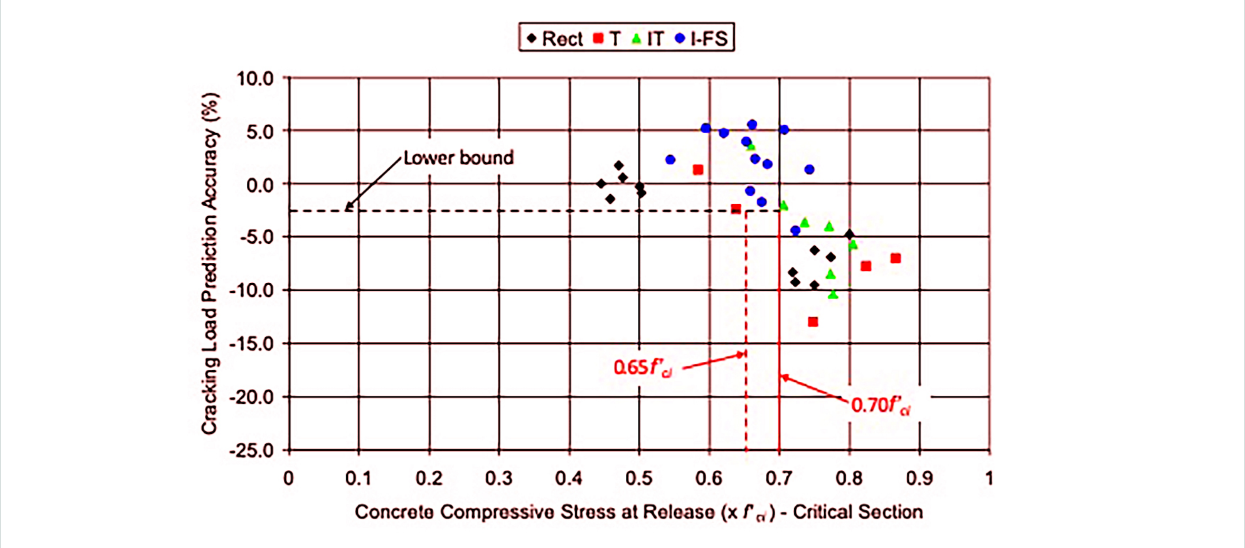

Figure 1. Comparison of measured and estimated cracking loads using AASHTO LRFD specifications losses. Source: Reproduced by permission from Birrcher, Bayrak, and Kreger (2010). Note: f'_{ci} = design compressive strength of concrete at transfer of prestress for pretensioned members and at time of application of tendon force for post-tensioned members.

The most significant stages to consider temporary stresses are at transfer of prestress, lifting after transfer, and shipping. If components are erected in the same manner as they are handled in the plant, a check at that stage is generally not necessary. The specified concrete strength at the stage considered should apply, as well as the prestress losses accumulated up to that time. At transfer of prestress only elastic shortening losses apply, but some long-term losses accumulate at later ages, such as at shipping and erection. Concrete matures over time and becomes less susceptible to damage from high compressive stress.

Lateral stability

Procedures for checking lateral stability were developed by Mast[4,5] and are summarized in detail in PCI's *Recommended Practice for Lateral Stability of Precast, Prestressed Concrete Bridge Girders*.[6] It should be noted that in the PCI recommended practice, published in 2016, the compressive stress limit for temporary stresses is given as $0.60f_c'$, where the concrete strength at the time under consideration is used. Because this preceded AASHTO's adoption of $0.65f_{ci}'$ in 2017, the next edition of the PCI recommended practice should be updated accordingly.

In checking lateral stability, a lateral offset of the girder's center of gravity—either from assumed fabrication imperfections during lifting or assumed fabrication imperfections plus roadway cross slope or superelevation during shipping—causes the girder to rotate about a roll axis passing through the lifting support locations or the roll center of the hauling vehicle, resulting in tilt. During lifting, the equilibrium tilt θ_{eq} is defined as the tilt that would be produced by the maximum extent of the permissible sweep tolerance (⅛ in. [3.175 mm] per 10 ft [3.048 m] of girder length) plus the lateral lifting embedment placement tolerance, both causing tilt in the same direction. This tilt introduces lateral bending, which increases the concrete compressive stresses at the extremities on one side of the component and reduces the concrete compressive stresses at the extremities on the other side. The lateral stability analysis itself is primarily concerned with increased tensile stresses because cracking in the component reduces lateral stiffness and could lead to progressively increasing lateral deflections and eventual failure. A factor of safety against cracking $FS \geq 1.0$ indicates that the increased maximum tensile stress due to tilt does not exceed the modulus of rupture of the concrete. However, the corresponding increased maximum compressive stress has typically not been compared with the compressive stress limit when determining the required concrete strength. The current PCI recommended practice suggests that this be done, which would lead to significant increases in required concrete strengths over past practice.

Compressive stress limits for tilted or laterally loaded components

According to descriptions of relevant research provided in Birrcher et al.,[2] the $0.65f_{ci}'$ recommendation was based on tests of specimens that either were uniformly loaded in compression or did not have lateral bending concerns as previously noted. Therefore, the experimentation represents a global stress condition in the specimens. Bending about the minor axis does not result in a net change in these global stresses but introduces peak stresses at the extremities of the component (peak compressive stress normally at the tip of the bottom flange).

As discussed, given that the 0.65 coefficient chosen by AASHTO for the compressive stress limit could also have been defensible as 0.70, it seems logical that $0.70f_{ci}'$ can be justified for stresses isolated at the extremities of the section. Because these criteria apply primarily to a potential reduction in the modulus of rupture in service, this is particularly true of designs that permit tensile stresses below the modulus of rupture (say, up to $0.19\sqrt{f_c'}$) in service. In such cases, a slight reduction of the modulus of rupture due to high early compressive stress should not result in premature cracking in service, particularly when those stresses are isolated at the extremities of the component.

Revised WSDOT criteria for lifting and shipping

Since WSDOT's adoption of new, more efficient girder sec-

Table 1. WSDOT compressive stress limits during lifting and shipping

Compressive stress limit at transfer of prestress	$0.65f_{ci}'$
Compressive stress limit when lifting a plumb girder from the form	$0.65f_{ci}'$
Compressive stress limit when lifting a girder at equilibrium tilt	$0.70f_{ci}'$
Compressive stress limit while shipping a plumb girder with ±20% impact	$0.65f_{ci}'$
Compressive stress limit while shipping a girder with ±20% impact and 2% cross slope	$0.70f_{ci}'$
Compressive stress limit while shipping a girder with no impact and 6% superelevation	$0.70f_{ci}'$

Note: f_c' = design compressive strength of concrete at time under consideration; f_{ci}' = design compressive strength of concrete at transfer of prestress for pretensioned members and at time of application of tendon force for post-tensioned members; WSDOT = Washington State Department of Transportation.

tions more than 20 years ago, spans have continued to grow longer and the girders slenderer. In order to be reasonably satisfied that safe handling of girders can occur at all stages of construction, WSDOT has implemented a policy of investigating lifting, hauling, and erection conditions during the initial design.[7] This policy also helps to minimize changes to the design after the project is awarded.

In addition, WSDOT has adopted a conservative design policy that permits no tension in the precompressed tensile zone under service loads for simple spans, even when the spans are subsequently made continuous for live loads.[8] Under such conditions, a slight reduction in the modulus of rupture due to high early compressive stress would not be expected to diminish the service life of the structure. In order to maintain the economy of current designs while mitigating the potential for overstress in girders, the analysis criteria and compressive stress limits listed in **Table 1** have been adopted for lifting and shipping bridge girders. Note that WSDOT does not include wind or centrifugal forces in the analysis.

These criteria are intended to represent realistic standard conditions that can be anticipated in practice and used for analysis during the design phase. Of course, special circumstances that are encountered on a project-specific basis should be checked separately.

Proposed revisions to AASHTO LRFD specifications

In order to implement the recommendations outlined in this paper, the following revisions to the AASHTO LRFD specifications[1] have been proposed by the authors to AASHTO Subcommittee T-10. At this writing, the processing of this proposal is in progress. The language shown on page 46 is directly from the referenced AASHTO LRFD specifications articles. Text shown in strikethrough format represents text that is proposed to be deleted from the original AASHTO LRFD specifications article. Text shown in underline format represents text that is proposed to be added to the original AASHTO LRFD specifications article.

As a proposed revision to the original AASHTO LRFD specifications articles, a definition of *temporary stresses* was added to distinguish them from stresses due to transient loads. The terminology *before losses* and *after losses* may be confusing to many engineers. The prestress loss estimation in Article 5.9.3 is time dependent and predicts losses over the lifetime of the structure. Peak stresses during pre-service conditions arise neither before any losses occur nor after all losses occur. The proposed revisions remove the terms *before losses* and *after losses* for this reason.

In addition, it is proposed that *before losses* and *after losses* be removed from Articles 5.3, 5.5.3.1, 5.6.3.1.2, 5.6.3.1.3b, 5.6.4.2, 5.6.4.4, 5.7.1.5, 5.7.2.1, and 5.8.2.4.1; Tables 5.9.2.2-1, 5.9.4.3.1, C5.9.4.3.1, 5.9.4.3.2, 5.10.6, 5.12.2.1, 5.12.3.3.6, C5.12.3.3.6, 5.12.3.4.2d, 5.12.3.4.3, and 5.12.9.4.3; and Appendixes A5.3 and C9.9.5.6.3.

Design example

The following design example is intended to illustrate WSDOT's revised handling and shipping criteria as outlined in this paper. It is based on actual WSDOT WF100G girders supplied for the Alaskan Way Viaduct replacement project in Seattle, Wash., in 2011. The results of the example calculations are slightly different from the calculations performed at the time of the project, primarily due to WSDOT's recently adopted policy of using the full permitted sweep tolerance of ⅛ in. (3.175 mm) per 10 ft (3.048 m) of girder length at initial lifting from the form rather than half that value, as previously considered. The only required change is that the lifting points were moved to 20.5 ft (6.25 m) from the ends rather than 19 ft (5.8 m), as originally done, in order to achieve the required factor of safety during lifting. All other parameters reflect what was actually done in the field during completion of the project.

The following information is based on design requirements for the structure in service, plant practices and experience, and/or criteria established in the governing specifications. The governing specifications are the WSDOT *Standard Specifications for Road, Bridge and Municipal Construction*,[9] as modified in this paper. All calculations were performed using gross section properties, though some results of calculations using transformed section properties are provided for comparison purposes.

General

- Girder type: WSDOT WF100G

- Overall girder length L: 205 ft (62.5 m)

- Time of lift: Soon after transfer of prestress
  At the time prestress is transferred into the girder, the girder spans end to end and the self-weight is most effective in countering the concrete stresses induced by prestress. Stresses at this stage will require the lowest concrete strength. However, once the girder is lifted at locations away from the ends (**Fig. 2**), the effectiveness of the self-weight in countering the stresses induced by prestress is reduced or reversed (due to negative moments at the lifting locations), resulting in a higher required concrete strength. Daily production cycles generally require lifting soon after the prestress is transferred, so it is most efficient to achieve the required concrete strength at lifting prior to transfer of prestress. Once accelerated curing is discontinued and the concrete begins to cool, it is always advisable to transfer the prestress as soon as possible to mitigate vertical restraint cracking in the girder. Under these circumstances the prestress can be transferred based on calculated stresses with the girder spanning end to end, but the girder should not be lifted until a separate set of test cylinders indicate that the required concrete strength at lifting has been achieved.

- Lifting method: Vertical cables. Lifting cables that are not at right angles to the top of the girder induce an axial force in the top flange. The effects of this force must be considered with respect to both stresses in the concrete and stability.

- Roll center above girder top flange: 0 in. (no rigid connection). Stability during lifting can be improved by rigidly attaching the lifting connection to the girder in order to raise the roll axis above the top surface of the girder. This approach is generally considered only if other means of improving stability (moving the lift points away from the ends, adding top strands, etc.) are not sufficient.

Girder gross section properties

- Girder height h: 100.00 in. (2540 mm)

- Girder area A_g: 1083 in.^{2} (698,708 mm^2)

Article 5.2

Temporary Stresses – Stresses that occur for a limited duration during pre-service conditions.

Article 5.9.2.3.1

5.9.2.3.1 – ~~For~~ Temporary Stresses ~~before Losses~~

5.9.2.3.1a – Compressive Stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0.65 f'_{ci} (ksi), except when lateral bending due to tilt, wind, or centrifugal force occurring during transportation of precast elements is explicitly considered, the compressive stress limit at the component extremities is permitted to be increased to 0.70f'_{ci} (ksi). These stress limits shall also apply for temporary pre-service load stages, such as lifting, hauling, and erection, with concrete strength at the time of loading substituted for f'_{ci} in the stress limits.

Article C5.9.2.3.1a

C5.9.2.3.1a

~~Previous research (Hale and Russell, 2006) suggests that the concrete stress limit for prestressed concrete components can safely exceed 0.60 f'_{ci}. However, c~~Concrete in the precompressed tensile zone subjected to compressive stresses at release greater than 0.70~~65~~ f'_{ci} can experience microcracking, leading to unconservative predictions of the external load to cause cracking (Birrcher and Bayrak, 2007; Heckmann and Bayrak, 2008; Schnittker and Bayrak, 2008; and Birrcher et al., 2010). While the coefficient 0.65 has been selected for the general case, an increase to 0.70 is permitted for peak compressive stresses, generally caused by lateral bending due to tilt or lateral loads during fabrication and construction including wind and centrifugal forces occurring during transportation of precast elements, since such stresses are isolated at the extremities of the cross section and do not represent the global stress state in the component. Figure 5.9.2.3.1a-1 illustrates the application of the compressive stress limits.

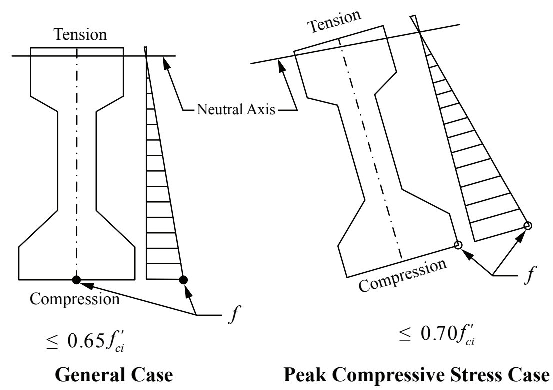

Figure 5.9.2.3.1a-1

Article 5.9.2.3.2

5.9.2.3.2 – ~~For~~ Stresses at Service Limit State ~~after Losses~~

Service Limit States may be investigated assuming all time-dependent losses have occurred.

AASHTO Table 5.9.2.3.2a-1. Compressive Stress Limits in Prestressed Concrete at Service Limit State ~~after Losses~~

Location	Stress Limit
Due to the sum of effective prestress and permanent loads	0.45 f'_c (ksi)
Due to the sum of effective prestress, permanent loads, and transient loads ~~as well as during shipping and handling~~	0.60$\Phi_w f'_c$ (ksi)

Figure 2. A 205 ft long girder for the Alaskan Way Viaduct replacement project is being lifted from the form. Note: 1 ft = 0.305 m.

- Top flange width b_{tf}: 49.00 in. (1244.6 mm)
- Bottom flange width b_{bf}: 38.375 in. (974.725 mm)
- Distance from top to center of gravity of girder section y_{gt}: 51.73 in. (1313.942 mm)
- Distance from bottom to center of gravity of girder section y_{gb}: 48.27 in. (1226.058 mm)
- Horizontal axis moment of inertia I_{gx}: 1,524,912 in.^{4} (6.347 × 10^{11} mm^4)
- Horizontal axis top-section modulus S_{gxt}: 29,481 in.^{3} (4.83 × 10^8 mm^3)
- Horizontal axis bottom-section modulus S_{gxb}: 31,589 in.^{3} (5.18 × 10^8 mm^3)
- Vertical axis moment of inertia I_{gy}: 72,516 in.^{4} (3.02 × 10^{10} mm^4)
- Vertical axis top-section modulus S_{gyt}: 2960 in.^{3} (4.85 × 10^7 mm^3)
- Vertical axis bottom-section modulus S_{gyb}: 3779 in.^{3} (6.2 × 10^7 mm^3)

WSDOT criteria for lifting girder from casting bed

- Compressive stress limit: In accordance with Table 1
- Tensile stress limit without bonded reinforcement sufficient to resist total tensile force in concrete:

  $$0.0948\lambda\sqrt{f'_{ci}} \le 0.200 \text{ ksi (1.379 MPa)}$$

  where λ = lightweight concrete factor
- Tensile stress limit with bonded reinforcement sufficient to resist total tensile force in concrete:

  $$0.24\lambda\sqrt{f'_{ci}}$$
- Minimum factor of safety against cracking FS: 1.0
- Minimum factor of safety against failure FS': 1.5
- Impact: 0%
- Wind load: No wind

Concrete

- Specified concrete design strength f'_c: 10.6 ksi (73 MPa)

- Density of plain concrete w_c: 0.155 kip/ft^3 (24.345 kN/m^3)
- Density of reinforced concrete w_{cr}: 0.165 kip/ft^3 (25.915 kN/m^3)
- Correction factor for source of aggregate K_1: 1.0
- Lightweight concrete factor λ: 1.0 for normalweight concrete

Lifting eccentricities due to fabrication tolerances

- Girder lateral sweep tolerance e_{sweep}: 2.56 in. (65.024 mm) (⅛ in. [3.175 mm] per 10 ft [3.048 m] of girder length)
- Lifting embedment placement tolerance e_{conn}: 0.25 in. (6.35 mm)

Permanent prestress

During design, the configuration of the permanent strands can be optimized[10] to reduce the demand on the prestressing bed while minimizing the required concrete strength at transfer of prestress and maintaining adequate stability during lifting.

The following information is calculated based on iteration. The primary variables manipulated to comply with stress limits and to achieve sufficient lateral stability are the distance of the lifting locations from the girder ends a_l, the number of temporary top strands N_t, and the required concrete strength at transfer of prestress f'_{ci}. From the fabrication perspective, the optimal goals are to minimize f'_{ci} to facilitate a daily production cycle and to minimize N_t to mitigate cost. The calculations that follow show the last cycle of iteration.

- Number of straight 0.6 in. (15.24 mm) diameter strands N_s: 46
- Number of harped 0.6 in. diameter strands N_h: 28
- Area of one strand A_p: 0.217 in.^{2} (140 mm^2)
- Harp point locations at 0.4L from girder ends a_h: 82 ft (25 m)
- Eccentricity of permanent strands between harp points e_{ph}: 43.79 in. (1112.27 mm)
- Eccentricity of permanent strands at ends e_{pe}: 14.33 in. (363.98 mm)
- Tensile strength of strand f_{pu}: 270 ksi (1862 MPa)
- Yield strength of strand f_{py}: 243 ksi (1675 MPa)
- Modulus of elasticity of strand E_p: 28,500 ksi (196,508 MPa)
- Initial seated stress in strand after jacking f_{pj}: 202.5 ksi (1396 MPa) (0.75f_{pu})

Lifting configuration

- Distance of the lifting locations from the girder ends a_l: 20.5 ft (6.25 m)
- Distance between lifting locations L_l: 164 ft (50 m)

Temporary top prestress

This example considers the top strands to be pretensioned and released at the same time as the permanent strands, but the top strands may also be post-tensioned at the appropriate stage of fabrication. The negative eccentricity indicates strands above the center of gravity of the girder cross section. Temporary top strands are normally detensioned after the beam is erected and braced at the ends. Top strands may be permanent as long as they are included in the design of the girders in service.

- Number of 0.6 in. (15.2 mm) temporary top strands N_t: 8
- Eccentricity of temporary top strands e_t: -49.73 in. (-1252.982 mm) (2 in. [50.8 mm] from top of girder)
- Initial seated stress in strand after jacking f_{pj}: 202.5 ksi (1396 MPa) (same as for permanent strands)

Concrete properties at transfer of prestress

- Required concrete strength at lifting f'_{ci}: 8.0 ksi (55 MPa) (to be achieved at transfer of prestress)
- Modulus of elasticity at transfer of prestress E_{ci}:

$$E_{ci} = 120{,}000 K_1 w_c^2 f_{ci}'^{0.33} = 120{,}000(1.0)(0.155)^2(8.0)^{0.33}$$

$$= 5{,}726 \text{ ksi } (39{,}480 \text{ MPa})$$

Prestress losses at harp point

- Relaxation between jacking and transfer of prestress[11] (t = 1 day) Δf_{pR0}:

$$\Delta f_{pR0} = \frac{\log(24t)}{45}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

$$= \frac{\log(24(1))}{45}\left[\frac{202.5}{243} - 0.55\right](202.5)$$

$$= 1.76 \text{ ksi } (12.135 \text{ MPa})$$

Stress in strand immediately prior to transfer of prestress f_{pbt}:

$$f_{pbt} = f_{pj} - \Delta f_{pR0} = 202.5 - 1.76 = 200.74 \text{ ksi } (1384 \text{ MPa})$$

Elastic shortening at transfer of prestress: The girder spans end-to-end upon release of prestress, so the moment at the harp point M_{hr} is calculated accordingly. Although the temporary top strands are unbonded for most of their length and do not conform to strain compatibility, the elastic shortening estimate should be reasonably close. For comparison purposes, elastic shortening loss using transformed section analyses is 19.43 ksi (133.97 MPa).

Total area of strands in girder A_{ps}:

$$A_{ps} = A_p\left(N_s + N_h + N_t\right) = 0.217\left(46+28+8\right)$$

$= 17.794$ in.² (11,480 mm²)

Weight of girder per unit length w_{gird}:

$$w_{gird} = w_{cr}A_g = \frac{0.165\left(1083\right)}{144} = 1.241 \text{ kip/ft (18.1 kN/m)}$$

Eccentricity of all strands between harp points e_h:

$$e_h = \frac{\left(N_s + N_h\right)e_{ph} + N_t e_t}{\left(N_s + N_h + N_t\right)}$$

$$= \frac{\left(46+28\right)43.79 + 8\left(-49.73\right)}{\left(46+28+8\right)} = 34.67 \text{ in. (880.62 mm)}$$

Moment at harp point at transfer of prestress M_{hr}:

$$M_{hr} = \frac{w_{gird}a_h\left(L - a_h\right)}{2} = \frac{1.241\left(82\right)\left(205-82\right)\left(12\right)}{2}$$

= 75,100 kip-in. (8487 kN-m)

Prestress loss due to elastic shortening of girder at transfer of prestress Δf_{pES}:

$$\Delta f_{pES} = \frac{A_{ps}f_{pbt}\left(I_{gx} + e_h^2A_g\right) - e_hM_{hr}A_g}{A_{ps}\left(I_{gx} + e_h^2A_g\right) + \dfrac{A_gI_{gx}E_{ci}}{E_p}}$$

$$= \frac{17.794\left(200.74\right)\left(1{,}524{,}912 + \left(34.67\right)^2\left(1083\right)\right) - 34.67\left(75{,}083\right)\left(1083\right)}{17.794\left(1{,}524{,}912 + \left(34.67\right)^2\left(1083\right)\right) + \dfrac{1083\left(1{,}524{,}912\right)\left(5726\right)}{28{,}500}}$$

= 19.05 ksi (131.35 MPa)

Concrete stresses at harp point during initial lifting, immediately after transfer of prestress

Prestress force in girder immediately after transfer of prestress P_i:

$$P_i = A_p\left(N_s + N_h + N_t\right)\left(f_{pbt} - \Delta f_{pES}\right)$$

$$= 0.217\left(46+28+8\right)\left(200.74 - 19.05\right)$$

= 3233 kip (14,380 kN)

Distance from lifting point to harp point x_l:

$$x_l = a_h - a_l = 82 - 20.5 = 61.5 \text{ ft (18.75 m)}$$

Moment at harp point during lifting M_{hl}:

$$M_{hl} = \frac{w_{gird}}{2}\left(L_lx_l - x_l^2 - a_l^2\right)$$

$$= \frac{1.241}{2}\left(164\left(61.5\right) - \left(61.5\right)^2 - \left(20.5\right)^2\right)\left(12\right)$$

= 43,809 kip-in. (4949 kN-m)

Stress in top concrete fibers at harp point during lifting plumb girder immediately after transfer of prestress f_{xt}:

$$f_{xt} = \frac{P_i}{A_g} - \frac{P_ie_h}{S_{gxt}} + \frac{M_{hl}}{S_{gxt}} = \frac{3233}{1083} - \frac{3233\left(34.67\right)}{29{,}481} + \frac{43{,}809}{29{,}481}$$

= 0.669 ksi (4.61 MPa) (compression)

Stress in bottom concrete fibers at harp point during lifting plumb girder immediately after transfer of prestress f_{xb}:

$$f_{xb} = \frac{P_i}{A_g} + \frac{P_ie_h}{S_{gxb}} - \frac{M_{hl}}{S_{gxb}} = \frac{3233}{1083} + \frac{3233\left(34.67\right)}{31{,}589} - \frac{43{,}809}{31{,}589}$$

= 5.147 ksi (35.50 MPa) (compression)

Required concrete strength at transfer:

$$f'_{ci} = \frac{5.147}{0.65} = 7.92 \text{ ksi} < 8.0 \text{ ksi}$$

OK during initial lifting, immediately after transfer of prestress

For comparison purposes, concrete stresses at the top and bottom of the girder using transformed section analysis are 0.746 and 5.110 ksi (5.14 and 35.2 MPa), respectively. For girders with harped strands, the controlling location for concrete stresses during lifting from the form is most likely either the lifting point or the harp point, depending on the prestressing configuration and the lifting locations. In this case the controlling location is the harp point. For girders with only straight and/or debonded strands, the controlling location can be at the end of the transfer length or the lifting location. Girders with debonded strands at the ends can have several locations where strands fully transfer their stress. Note that the compressive stress limit for this case is $0.65f'_{ci}$.

Concrete stresses at harp point during initial lifting, immediately after transfer of prestress, including θ_{eq}

- Initial eccentricity e_i (assuming full magnitude of ⅛ in. [3.175 mm] per 10 ft [3.048 m] sweep tolerance):

$$e_{sweep} = \frac{L}{10}\left(\frac{1}{8}\right) = \frac{205}{10}\left(\frac{1}{8}\right) = 2.56 \text{ in. (65.024 mm)}$$

Offset factor that determines distance between the roll axis and center of gravity of arc of a curved girder F_{offset}:

$$F_{offset} = \left(\frac{L_l}{L}\right)^2 - \frac{1}{3} = \left(\frac{164}{205}\right)^2 - \frac{1}{3} = 0.307$$

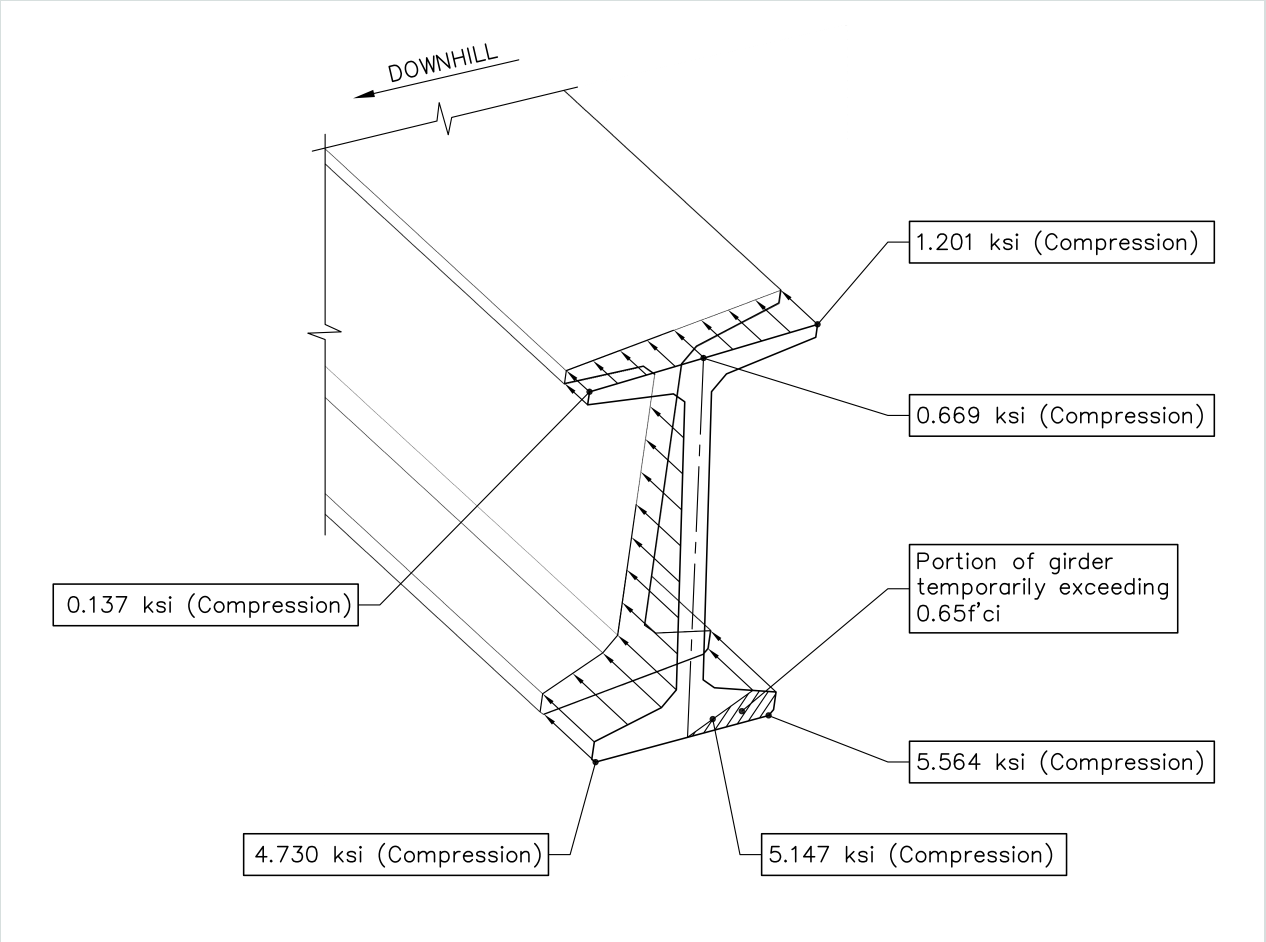

Figure 3. Stress profile of lifted girder hanging at equilibrium tilt. Note: f_{ci}' = design compressive strength of concrete at transfer of prestress for pretensioned members and at time of application of tendon force for post-tensioned members. 1 ksi = 6.895 MPa.

e_{conn} = 0.25 in. (6.35 mm)

$$e_i = e_{sweep}F_{offset} + e_{conn} = 2.56(0.307) + 0.25$$

$$= 1.04 \text{ in. } (26.416 \text{ mm})$$

- Estimate initial camber to adjust height of roll axis above center of gravity of hanging girder y_{lift}:

Component of downward deflection due to self-weight Δ_{self}:

$$\Delta_{self} = \frac{5w_{gird}L^4}{384E_{ci}I_{gx}} = \frac{5(1.241)(205)^4(12)^3}{384(5726)(1{,}524{,}912)}$$

$$= 5.65 \text{ in. } (143.51 \text{ mm}) \downarrow$$

Eccentricity of all strands at ends e_e:

$$e_e = \frac{(N_s + N_h)e_{pe} + N_t e_t}{(N_s + N_h + N_t)} = \frac{(46+28)(14.33)+8(-49.73)}{(46+28+8)}$$

$$= 8.08 \text{ in. } (205.23 \text{ mm})$$

Change in eccentricity of all strands between harp point and end of girder e':

$$e' = e_h - e_e = 34.67 - 8.08 = 26.59 \text{ in. } (675.39 \text{ mm})$$

Component of upward deflection due to prestress Δ_{ps}:

$$\Delta_{ps} = \frac{P_i e_e L^2}{8E_{ci}I_{gx}} + \frac{P_i e'}{E_{ci}I_{gx}}\left(\frac{L^2}{8} - \frac{a_h^2}{6}\right)$$

$$= \frac{3233(8.08)(205)^2(12)^2}{8(5726)(1{,}524{,}912)}$$

$$+ \frac{3233(26.59)(12)^2}{(5726)(1{,}524{,}912)}\left(\frac{(205)^2}{8} - \frac{(82)^2}{6}\right)$$

$$= 8.12 \text{ in. } (206.25 \text{ mm}) \uparrow$$

Additional component of upward deflection due to girder overhangs beyond lift points Δ_{ohang}:

$$\Delta_{ohang} = \frac{w_{gird} a_l L^3}{16 E_{ci} I_{gx}} = \frac{1.241(20.5)(205)^3(12)^3}{16(5726)(1,524,912)}$$

$= 2.71$ in. (68.83 mm) ↑

Total camber during lifting immediately after transfer of prestress Δ_{tot}:

$$\Delta_{tot} = -\Delta_{self} + \Delta_{ps} + \Delta_{ohang} = -5.65 + 8.12 + 2.71$$

$= 5.18$ in. (131.57 mm) ↑

$$y_{lift} = y_{gt} - \Delta_{tot} F_{offset} = 51.73 - 5.18(0.307)$$

$= 50.14$ in. (1273.56 mm)

- Calculate theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally $\bar{z}_0$:

$$\bar{z}_0 = \frac{w_{gird}}{12 E_{ci} I_{gy} L}\left(\frac{1}{10}L_l^5 - a_l^2 L_l^3 + 3a_l^4 L_l + \frac{6}{5}a_l^5\right)$$

$$= \frac{1.241(12)^3}{12(5726)(72,516)(205)}$$

$$\left(\frac{1}{10}(164)^5 - (20.5)^2(164)^3 + 3(20.5)^4(164) + \frac{6}{5}(20.5)^5\right)$$

$= 21.20$ in. (538.48 mm)

Figure 3 shows the stress profile of the lifted girder hanging at equilibrium tilt. Note that the portion of the girder where the compressive stress exceeds the 0.65 f'_{ci} limit amounts to a small portion of the bottom flange only.

- Calculate equilibrium tilt angle θ_{eq}:

$$\theta_{eq} = \frac{e_i}{y_{lift} - \bar{z}_0} = \frac{1.04}{50.14 - 21.20} = 0.0359 \text{ radians}$$

Lateral moment at harp point when lifting at θ_{eq}, M_{eqlat}:

$$M_{eqlat} = \theta_{eq} M_{hl} = 0.0359(43,809)$$

$= 1574$ kip-in. (177.7 kN-m)

Minimum concrete stress in top flange at harp point with girder hanging at equilibrium tilt when lifting immediately after transfer of prestress f_{tt}:

$$f_{tt} = f_{xt} - \frac{M_{eqlat}}{S_{gyt}} = 0.669 - \frac{1574}{2960}$$

$= 0.137$ ksi (0.946 MPa) (compression)

Maximum concrete stress in bottom flange at harp point with girder hanging at equilibrium tilt when lifting immediately after transfer of prestress f_{bt}:

$$f_{bt} = f_{xb} + \frac{M_{eqlat}}{S_{gyb}} = 5.147 + \frac{1574}{3779}$$

$= 5.564$ ksi (38.36 MPa) (compression)

Required concrete strength at transfer:

$$f'_{ci} = \frac{5.564}{0.70} = 7.95 \text{ ksi} < 8.0 \text{ ksi}$$

OK during initial lifting, immediately after transfer of prestress, including θ_{eq}

For this case, the freshly stripped girder is assumed to be hanging at the equilibrium tilt caused by fabrication imperfections at their permissible tolerance limits causing tilt in the same direction. Because the lateral bending caused by the equilibrium tilt increases stresses only at the flange extremities, the compressive stress limit is $0.70 f'_{ci}$. Had this stress been evaluated based on a compressive stress limit of $0.65 f'_{ci}$, the required concrete strength at transfer of prestress would have increased to 8.6 ksi (59.3 MPa), which at this concrete strength level represents a very significant increase in cost and schedule.

Lateral stability

- Calculate tilt angle at cracking θ_{max}:

Modulus of rupture of concrete f_r:

$$f_r = 0.24\lambda\sqrt{f'_{ci}} = 0.24(1.0)\sqrt{8.0}$$

$= 0.679$ ksi (4.68 MPa) (tension)

Lateral bending moment at harp point of girder at cracking when lifting immediately after transfer of prestress M_{latl}:

$$M_{latl} = \frac{2(f_r + f_{xt}) I_{gy}}{b_{tf}} = \frac{2(0.679 + 0.670)(72,516)}{49.00}$$

$= 3990$ kip-in. (451 kN-m)

$$\theta_{max} = \frac{M_{latl}}{M_{hl}} = \frac{3990}{43,809} = 0.0911 \text{ radians}$$

Calculate factor of safety against cracking FS:

$$FS = \frac{y_{lift}\theta_{max}}{\bar{z}_0 \theta_{max} + e_i} = \frac{50.14(0.0911)}{21.20(0.0911) + 1.04} = 1.54 > 1.0 \text{ \textbf{OK}}$$

- Calculate factor of safety against failure:

$$\theta'_{max} = \sqrt{\frac{e_i}{2.5\bar{z}_0}} = \sqrt{\frac{1.04}{2.5(21.20)}} = 0.1401 \text{ radians}$$

$$FS' = \frac{y_{lift}\theta'_{max}}{\bar{z}_0 \theta'_{max}(1 + 2.5\theta'_{max}) + e_i}$$

$$= \frac{50.14(0.1401)}{21.20(0.1401)(1 + 2.5(0.1401)) + 1.04}$$

$= 1.39 < 1.5$ **NG**

Figure 4. Transportation of 205 ft long girders for the Alaskan Way Viaduct replacement project through the Port of Tacoma. Note: 1 ft = 0.305 m.

Mast[5] indicated that if the factor of safety against failure FS' is less than the factor of safety against cracking FS, the maximum factor of safety occurs just before cracking. In this case, FS' should be taken equal to FS, and therefore $FS' = FS = 1.54 \geq 1.5$ **OK**.

Shipping at 10 days

WSDOT requires girders to be at least 10 days old prior to shipping (**Fig. 4**). Long-term prestress losses due to shrinkage, creep, and strand relaxation calculated at 10 days using the AASHTO LRFD specifications refined method are Δ_{fpLT} = 6.32 ksi (43.57 MPa).

Girder stresses during shipping with 20% impact

Prestress force in girder at shipping P_s:

$$P_s = A_p\left(N_s + N_h + N_t\right)\left(f_{pbt} - \Delta f_{pES} - \Delta f_{pLT}\right)$$

$$= 0.217\left(46+28+8\right)\left(200.74-19.05-6.32\right)$$

$$= 3121 \text{ kip } (13{,}882 \text{ kN})$$

Figure 5 shows the hauling configuration. Girder cantilevers over the truck supports are determined to be 25 ft (7.62 m) at the tractor end, which is limited by proximity of the girder end to the cab, and 30 ft (9.144 m) at the trailer end. Stresses are checked along the length of the girder, and the critical location for compressive stress is determined to occur at the harp point closest to the trailer end with -20% (upward) impact. Impact loads the girder flanges uniformly across their width, so stresses are evaluated based on $0.65f'_c$.

Static moment at harp point during shipping M_{hs}:

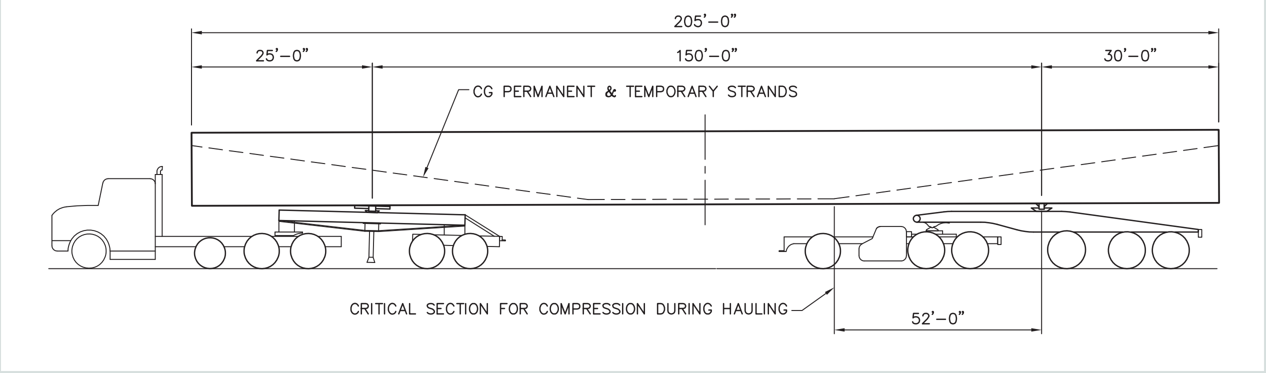

Figure 5. Hauling configuration for 205 ft long girders for the Alaskan Way Viaduct replacement project. Note: 1 ft = 0.305 m.

M_{hs} = 31,953 kip-in. (3610 kN-m)

Stress in top concrete fibers at harp point during shipping plumb girder with 20% impact f_{xtsi}:

$$f_{xtsi} = \frac{P_s}{A_g} - \frac{P_s e_h}{S_{gxt}} + \frac{(0.8)M_{hs}}{S_{gxt}}$$

$$= \frac{3121}{1083} - \frac{3121(34.67)}{29,481} + \frac{(0.8)31,953}{29,481}$$

= 0.079 ksi (0.545 MPa) (compression)

Stress in bottom concrete fibers at harp point during shipping plumb girder with 20% impact f_{xbsi}:

$$f_{xbsi} = \frac{P_s}{A_g} + \frac{P_s e_h}{S_{gxb}} - \frac{(0.8)M_{hs}}{S_{gxb}}$$

$$= \frac{3121}{1083} + \frac{3121(34.67)}{31,589} - \frac{(0.8)31,953}{31,589}$$

= 5.498 ksi (37.9 MPa) (compression)

Required concrete strength at shipping:

$$f'_{ci} = \frac{5.498}{0.65} = 8.46 \text{ ksi} < 10.6 \text{ ksi} \qquad \textbf{OK}$$

Girder stresses during shipping with 20% impact and 2% cross slope

The sweep in the girder is assumed to be the same as during lifting, e_{sweep} = 2.56 in. (65 mm)

$$F_{offset} = \left(\frac{L_l}{L}\right)^2 - \frac{1}{3} = \left(\frac{205-30-25}{205}\right)^2 - \frac{1}{3} = 0.202$$

Lateral placement tolerance on truck support e_{truck} = 1.00 in. (25.4 mm)

$$e_i = e_{sweep}F_{offset} + e_{truck} = 2.56(0.202) + 1.00$$

= 1.52 in. (38.6 mm)

The delivery vehicle had a rotational stiffness K_θ = 80,000 in.-kip/radian (9038 kN-m/radian) and a wheelbase of 96 in. (2438.4 mm) (distance from centerline of vehicle to center of dual tires z_{max} = 48 in. [1219.2 mm]), with the center of axle h_r = 24 in. (609.6 mm) above the road. Total camber during shipping Δ_{ship}, including additional upward deflection due to the overhangs, is estimated to be 8.29 in. (210.6 mm). The top-of-truck support is 72 in. (1828.8 mm) above the road, so the height of the girder center of gravity at this location is:

$$h_{cg} = 72 + y_{gb} = 72 + 48.27 = 120.27 \text{ in. (3054.9 mm)}$$

For the entire girder, the height of the center of gravity above the center of axle (center of rotation) is:

$$y_{ship} = h_{cg} - h_r + F_{offset}\Delta_{ship} = 120.27 - 24 + 0.202(8.29)$$

= 97.95 in. (2487.9 mm)

Girder weight $W = w_{gird}(L) = 1.241(205) = 254.41$ kip (1131.35 kN)

Modulus of elasticity of concrete at time considered E_c:

$$E_c = 120,000K_1 w_c^2 f_c'^{0.33} = 120,000(1.0)(0.155)^2(10.6)^{0.33}$$

= 6283 ksi (43,321 MPa)

$$\bar{z}_0 = \frac{w_{gird}}{12E_cI_{gy}L}\left(\frac{1}{10}L_t^5 - a_t^2L_t^3 + 3a_t^4L_t + \frac{6}{5}a_t^5\right)$$

where

a_t = average cantilever = 27.5 ft (8.4 m)

$$\bar{z}_0 = \frac{1.241(12)^3}{12(6283)(72,516)(205)}\left(\frac{1}{10}(150)^5 - (27.5)^2(150)^3 + 3(27.5)^4(150) + \frac{6}{5}(27.5)^5\right)$$

= 10.17 in. (258.3 mm)

Roll angle of major axis of girder with respect to plumb during shipping at 2% cross slope θ_2:

$$\theta_2 = \frac{K_\theta\alpha + We_i}{K_\theta - W(y_{ship} + \bar{z}_0)} = \frac{80,000(0.02) + 254.35(1.52)}{80,000 - 254.35(97.95 + 10.17)}$$

= 0.0378 radians

Lateral bending moment at harp point of girder during shipping at 2% cross slope M_{slat2}:

$$M_{slat2} = \theta_2 M_{hs} = 0.0378(31,953) = 1,209 \text{ kip-in. (136.6 kN-m)}$$

Minimum concrete stress in top flange at harp point during shipping with 20% impact and 2% cross slope f_{tls2}:

$$f_{tls2} = f_{xtsi} - \frac{M_{slat2}}{S_{gyt}} = 0.079 - \frac{1209}{2960}$$

= -0.329 ksi (-2.268 MPa) (tension)

Maximum concrete stress in bottom flange at harp point during shipping with 20% impact and 2% cross slope f_{bls2}:

$$f_{bls2} = f_{xbsi} + \frac{M_{slat2}}{S_{gyb}} = 5.498 + \frac{1209}{3779}$$

= 5.817 ksi (40.1 MPa) (compression)

Based on a compressive stress limit of $0.70f'_c$, required concrete strength at shipping:

$$f'_c = \frac{5.817}{0.70} = 8.31 \text{ ksi} < 10.6 \text{ ksi} \qquad \textbf{OK}$$

Girder stresses during shipping with no impact and 6% superelevation

Roll angle of major axis of girder with respect to plumb during shipping at 6% superelevation θ_6:

$$\theta_6 = \frac{K_\theta \alpha + We_i}{K_\theta - W\left(y_{ship} + \bar{z}_0\right)} = \frac{80{,}000(0.06) + 254.41(1.52)}{80{,}000 - 254.41(97.95 + 10.17)}$$
$$= 0.0988 \text{ radians}$$

Lateral bending moment at harp point of girder during shipping at 6% superelevation M_{slat6}:

$$M_{slat6} = \theta_6 M_{hs} = 0.0988(31{,}953) = 3{,}157 \text{ kip-in. (357 kN-m)}$$

Stress in top concrete fibers at harp point during shipping with no impact f_{xts}:

$$f_{xts} = \frac{P_s}{A_g} - \frac{P_s e_h}{S_{gxt}} + \frac{M_{hs}}{S_{gxt}} = \frac{3121}{1083} - \frac{3121(34.67)}{29{,}481} + \frac{31{,}953}{29{,}481}$$
$$= 0.295 \text{ ksi (2.04 MPa) (compression)}$$

Stress in bottom concrete fibers at harp point during shipping with no impact f_{xbs}:

$$f_{xbs} = \frac{P_s}{A_g} + \frac{P_s e_h}{S_{gxb}} - \frac{M_{hs}}{S_{gxb}} = \frac{3121}{1083} + \frac{3121(34.67)}{31{,}589} - \frac{31{,}953}{31{,}589}$$
$$= 5.296 \text{ ksi (36.5 MPa) (compression)}$$

Minimum concrete stress in top flange at harp point during shipping with no impact and 6% superelevation f_{tls6}:

$$f_{tls6} = f_{xts} - \frac{M_{slat6}}{S_{gyt}} = 0.295 - \frac{3157}{2960}$$
$$= -0.772 \text{ ksi (-5.3 MPa) (tension)}$$

Maximum concrete stress in bottom flange at harp point during shipping with no impact and 6% superelevation f_{bls6}:

$$f_{bls6} = f_{xbs} + \frac{M_{slat6}}{S_{gyb}} = 5.296 + \frac{3157}{3779}$$
$$= 6.131 \text{ ksi (42.3 MPa) (compression)}$$

Based on a compressive stress limit of $0.70 f'_c$, required concrete strength at shipping:

$$f'_c = \frac{6.130}{0.70} = 8.76 \text{ ksi} < 10.6 \text{ ksi} \quad \textbf{OK}$$

Based on tensile stress limit of $0.24\sqrt{f'_c}$, required concrete strength at shipping:

$$f'_c = \left(\frac{0.772}{0.24}\right)^2 = 10.35 \text{ ksi} < 10.6 \text{ ksi} \quad \textbf{OK}$$

In this case, tension controls the required concrete strength. This is not desirable because the required concrete strength grows rapidly with increasing tension. The addition of two more temporary top strands or the use of larger 0.62 or 0.7 in. (15.748 or 17.78 mm) temporary top strands could reduce the tension and thereby reduce the required concrete strength at shipping to 8.8 ksi (60.7 MPa).

Lateral stability

- Calculate tilt angle at cracking θ_{max}:

$$f_r = 0.24\lambda\sqrt{f'_c} = 0.24(1.0)\sqrt{10.6} = 0.781 \text{ ksi (5.4 MPa)}$$

$$f_{xts} = 0.295 \text{ ksi (2.04 MPa)}$$

Lateral bending moment at harp point of girder at cracking during shipping M_{lats}:

$$M_{lats} = \frac{2\left(f_r + f_{xts}\right)I_{gy}}{b_{tf}} = \frac{2(0.781 + 0.295)(72{,}516)}{49.00}$$
$$= 3{,}188 \text{ kip-in. (360 kN-m)}$$

$$M_{hs} = 31{,}953 \text{ kip-in. (3610 kN-m)}$$

$$\theta_{max} = \frac{M_{lats}}{M_{hs}} = \frac{3188}{31{,}953} = 0.0998 \text{ radians}$$

- Calculate factor of safety against cracking FS:

$$FS = \frac{K_\theta\left(\theta_{max} - \alpha\right)}{W\left(\left(\bar{z}_0 + y_{ship}\right)\theta_{max} + e_i\right)}$$
$$= \frac{80{,}000(0.0998 - 0.06)}{254.41\left((10.17 + 97.95)0.0998 + 1.52\right)}$$
$$= 1.02 > 1.0 \qquad \textbf{OK}$$

- Calculate factor of safety against rollover FS':

$$\theta'_{max} = \frac{W\left(z_{max} - h_r\alpha\right)}{K_\theta} + \alpha = \frac{254.41\left(48 - 24(0.06)\right)}{80{,}000} + 0.06$$
$$= 0.208 \text{ radians}$$

$$FS' = \frac{K_\theta\left(\theta'_{max} - \alpha\right)}{W\left(\left(\bar{z}_0\left(1 + 2.5\theta'_{max}\right) + y_{ship}\right)\theta'_{max} + e_i\right)}$$

$$= \frac{80{,}000(0.208 - 0.06)}{254.41\left(\left(10.17\left(1 + 2.5(0.208)\right) + 97.95\right)(0.208) + 1.52\right)}$$
$$= 1.85 > 1.5 \qquad \textbf{OK}$$

Implementing stability design practices

State-of-the-art analysis techniques, best practices, and industry recommendations have been developed and published. As the example shows, girder stresses and stability at initial lifting and hauling are integral elements of the design process. Lifting and hauling conditions are often a governing design case. Designing for optimized fabrication and girder stability involves complex, iterative analytical procedures. Properly implemented tools are essential for lateral stability design to be a common and routine practice.

WSDOT has successfully implemented the practice of lateral-stability design[7] with sophisticated software tools. WSDOT's prestressed concrete girder design software—PGSuper for pretensioned girders, PGSplice for post-tensioned spliced girders, and PGStable for general precast concrete girder stability analysis—incorporates the necessary analytical procedures to enable engineers to arrive at acceptable design solutions quickly. These tools are part of the BridgeLink suite of bridge engineering software, which can be downloaded free of charge at https://www.wsdot.wa.gov/eesc/bridge/software.

Conclusion

For many years, WSDOT has employed a policy of investigating lifting, hauling, and erection conditions during the initial design phase in order to be reasonably satisfied that safe handling of girders can occur at all stages of construction. This process has been successful in optimizing girder design to suit local fabrication and shipping capabilities, promote economy, and minimize revisions to the girder design after the project is awarded. For agencies and designers who would like to follow suit, it is important that the project specifications document the assumptions and criteria used to derive the handling and shipping information provided in the contract documents and to require calculations for any deviations from these assumptions and criteria on any given project.

Until recently, WSDOT's criteria did not consider the additional compressive stresses induced by lateral bending due to tilt of the girder during handling and shipping. These peak stresses are isolated at the girder extremities and do not affect the global stresses in the top and bottom flanges. If evaluated based on current allowable compressive stress limits, these additional compressive stresses would increase the required concrete strength at all stages of construction significantly, affecting both cost and schedule. Based on published research, WSDOT has adopted an increased allowable compressive stress at the girder extremities under conditions that include lateral bending. These increased allowable compressive stresses have also been proposed as revisions to the AASHTO LRFD specifications.

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Notation

a_h = harp point locations at 0.4L from girder ends

a_l = distance of lifting locations from girder ends

a_t = average distance of truck supports from girder ends

A_g = gross area of girder section

A_p = area of one strand

A_{ps} = total area of strands in girder

b_{bf} = girder bottom flange width

b_{tf} = girder top flange width

e' = change in eccentricity of all strands between harp point and end of girder = $e_h - e_e$

e_{conn} = lifting embedment placement tolerance

e_e = eccentricity of all strands at ends

e_h = eccentricity of all strands between harp points

e_i = initial eccentricity of center of gravity of girder from roll axis due to maximum extent of fabrication tolerances

e_{pe} = eccentricity of permanent strands at ends

e_{ph} = eccentricity of permanent strands between harp points

e_{sweep} = girder lateral sweep tolerance

e_t = eccentricity of temporary top strands

e_{truck} = lateral placement tolerance on truck support

E_c = modulus of elasticity of concrete at time considered

E_{ci} = modulus of elasticity of concrete at transfer of prestress

E_p = modulus of elasticity of strand

f_{bls2} = maximum concrete stress in bottom flange at harp point during shipping with 20% impact and 2% cross slope

f_{bls6} = maximum concrete stress in bottom flange at harp point during shipping with no impact and 6% superelevation

f_{bt} = maximum concrete stress in bottom flange at harp point with girder hanging at equilibrium tilt when lifting immediately after transfer of prestress

f'_c = design compressive strength of concrete at time under consideration

f'_{ci} = design compressive strength of concrete at transfer of prestress for pretensioned members and at time of application of tendon force for post-tensioned members

f_{pbt} = stress in strand immediately prior to transfer of prestress

f_{pj} = initial seated stress in strand after jacking

f_{pu} = tensile strength of strand

f_{py} = yield strength of strand

f_r = modulus of rupture of concrete

f_{tt} = minimum concrete stress in top flange at harp point with girder hanging at equilibrium tilt when lifting immediately after transfer of prestress

f_{tls2} = minimum concrete stress in top flange at harp point during shipping with 20% impact and 2% cross slope

f_{tls6} = minimum concrete stress in top flange at harp point during shipping with no impact and 6% superelevation

f_{xb} = stress in bottom concrete fibers at harp point during lifting plumb girder immediately after transfer of prestress

f_{xbs} = stress in bottom concrete fibers at harp point during shipping with no impact

f_{xbsi} = stress in bottom concrete fibers at harp point during shipping plumb girder with 20% impact

f_{xt} = stress in top concrete fibers at harp point during lifting plumb girder immediately after transfer of prestress

f_{xts} = stress in top concrete fibers at harp point during shipping with no impact

f_{xtsi} = stress in top concrete fibers at harp point during shipping plumb girder with 20% impact

F_{offset} = offset factor that determines distance between the roll axis and center of gravity of arc of a curved girder

FS = factor of safety against cracking in tilted girder

FS' = factor of safety against failure in tilted girder

h = girder height

h_{cg} = height of center of gravity of girder above road

h_r = height of truck roll center above road

I_{gx} = moment of inertia of gross girder section about horizontal centroidal axis

I_{gy} = moment of inertia of gross girder section about vertical centroidal axis

K_1 = correction factor for source of aggregate

K_θ = sum of rotational spring constants of truck axles

L = overall length of component

L_l = distance between lifting locations

L_t = distance between truck supports

M_{eqlat} = lateral moment at harp point when lifting at θ_{eq}

M_{hl} = moment at harp point during lifting

M_{hr} = moment at harp point at transfer of prestress

M_{hs} = moment at harp point during shipping

M_{latl} = lateral bending moment at harp point of girder at cracking when lifting immediately after transfer of prestress

M_{lats} = lateral bending moment at harp point of girder at cracking during shipping

M_{slat2} = lateral bending moment at harp point of girder during shipping at 2% cross slope

M_{slat6} = lateral bending moment at harp point of girder during shipping at 6% superelevation

N_h = number of harped strands

N_s = number of straight strands

N_t = number of temporary top strands

P_i = prestress force in girder immediately after transfer of prestress

P_s = prestress force in girder at shipping

S_{gxb} = bottom-section modulus of gross girder section about horizontal centroidal axis

S_{gxt} = top-section modulus of gross girder section about horizontal centroidal axis

S_{gyb} = bottom-section modulus of gross girder section about vertical centroidal axis

S_{gyt} = top-section modulus of gross girder section about vertical centroidal axis

w_c = unit weight of plain concrete

w_{cr} = unit weight of reinforced concrete

w_{gird} = weight of girder per unit length

W = total weight of girder

x_l = distance from lifting point to harp point

y_{gb} = distance from bottom of girder to center of gravity of gross girder section

y_{gt} = distance from top of girder to center of gravity of gross girder section

y_{lift} = height of roll axis above center of gravity of hanging girder

y_{ship} = height of center of gravity of girder above roll axis during shipping

z_{max} = distance from centerline of vehicle to center of dual tires

$\bar{z}_0$ = theoretical lateral deflection of center of gravity of girder with full dead weight applied laterally

α = cross slope or superelevation of roadway

Δ_{ohang} = additional component of upward deflection due to girder overhangs beyond lift points

Δ_{ps} = component of upward deflection due to prestress

Δ_{self} = component of downward deflection due to self-weight

Δ_{ship} = total camber during shipping

Δ_{tot} = total camber during lifting immediately after transfer of prestress

Δf_{pES} = prestress loss due to elastic shortening of girder at transfer of prestress

Δf_{pLT} = long-term prestress losses due to shrinkage, creep, and strand relaxation, calculated at the time under consideration

Δf_{pR0} = relaxation of strand between jacking and transfer of prestress

θ_2 = roll angle of major axis of girder with respect to plumb during shipping at 2% cross slope

θ_6 = roll angle of major axis of girder with respect to plumb during shipping at 6% superelevation

θ_{eq} = equilibrium tilt of member produced by the maximum extent of the permissible sweep tolerance plus the lateral lifting embedment placement tolerance, both causing tilt in the same direction

θ_{max} = tilt angle at which cracking begins measured from plumb

θ'_{max} = tilt angle at maximum factor of safety against failure measured from plumb

λ = lightweight concrete factor

ϕ_w = hollow column reduction factor

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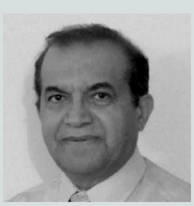

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Abstract

The allowable level of temporary concrete compressive stress in precast, prestressed concrete components has been a source of debate in the concrete industry for many years. Traditionally, these stresses have been considered to originate only from the effects of prestress combined with the self-weight of a plumb component, evaluated about the major axis. The maximum compressive stress divided by the coefficient of the compressive stress limit determines the required concrete strength. Although these temporary stresses can occur at any time from fabrication through erection into the structure, the critical case is usually at transfer of prestress and subsequent lifting from the form. At this stage, the prestress force is higher and the concrete strength is lower than at any other point in the life of the component. At this early age, concrete is also more susceptible to damage from high compressive stress.

As materials and fabrication capabilities in the precast, prestressed concrete industry advance, components are becoming longer and slenderer, particularly within the transportation sector. Such components require serious consideration of lateral stability during handling, which introduces bending about the minor axis. This lateral bending will increase maximum tensile and compressive stresses at the extremities of the component. These localized stresses traditionally have not been used to determine the required concrete strength, and doing so at current stress limits can significantly increase the required concrete strength.

This paper is intended to reconcile the interaction between the temporary concrete compressive stresses traditionally used to determine the required concrete strength and the requirements for lateral stability, primarily the additional stresses due to lateral bending. The assumption is made that compressive stresses govern the determination of the required concrete strength. It is normally not efficient to determine the required concrete strength based on tensile stresses, which can be satisfied in a different manner.

Keywords

AASHTO, American Association of State Highway and Transportation Officials, compressive stress, criteria, lateral stability, load- and resistance-factor design, LRFD, strength, Washington State Department of Transportation, WSDOT.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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