California Amendments to AASHTO LRFD Bridge Design Specifications and its Impact on Precast Bridge Design and Construction

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Abstract

As the policy makers of California Department of Transportation (Caltrans), our mission is to provide the safest transportation system in the nation and maximize transportation system performance and accessibility. The main purpose for California Amendments to AASHTO LRFD Bridge Design Specifications is not only to add adequate California standardized design, of many years, to current AASHTO LRFD Specifications, but also to modify AASHTO LRFD Specifications based on previous successful California bridge design practices to efficiently deliver transportation projects, and to promote quality services while seeking creative solutions and taking intelligent risks. Based on many research studies and past experience, Caltrans amended AASHTO LRFD Specifications every four years based on current California bridge design practices and recent research results, which include areas of live loads, load distribution factors, analysis methods, load factors for ultimate strength combinations, prestress losses, and application of modified compression field theory for shear design. Some amendments reflect the modifications or changes resulting from California precast bridge design practices. This paper reviews reasons why Caltrans make the modifications of design code in California and intends to share its design philosophy and results with other States policy makers and engineers. Additionally, this paper illustrates common design and construction practices of precast bridges in California and California Amendments' impact on them, such as allowable service limits for concrete crack control, girder deflection and comber, spliced girders, seismic connections, skew factors, debonding, diaphragms, and usage of welded wire reinforcement. The goal of this paper is to give other State policy makers and bridge engineers a general sense what California Amendments are and its impact on precast bridge design and construction. At mean time, it intends to open the discussion among the different State policy makers and PCI bridge members if some of the California Amendments are needed to be adopted by AASHTO and its pros and cons to the precast industry.

Keywords: Precast Bridge; AASHTO LRFD Bridge Design Specifications; California Amendments

INTRODUCTION

Structural concrete design has gone through numerous changes since its early inception in the late 19th century. As time progressed, concrete features had been better understood. At the same time, technology developed and concrete behavior advanced as well. In order to achieve the greater reliability during times, adaptations and alterations were made where necessary to refine the concrete design standards and concrete design process. Throughout its progressive life, concrete design has undertaken many different forms, primarily ASD (Allowable Stress Design), LFD (Load Factor Design), and LRFD (Load and Resistance Factor Design) in the last century.

In 1994 the American Association of State Highway and Transportation Officials (AASHTO) released the first edition of the Load and Resistance Factor Design (LRFD) Bridge Design Specification. The new specification was to replace the existing Load Factor Design (LFD) Standard Specification, which was to be phased out towards the end of the decade. Load and Resistance Factor Design (LRFD) is primarily a modification of the LFD design philosophy. Instead of having fixed load and resistance factors as in the LFD design philosophy, factors are allowed to vary so that the designer may choose the appropriate one based on the specifics of each load case. This new probabilistic approach recognizes that certain loads are more variable than are others. Not only does this provide greater reliability, but flexibility as well. The load and resistance factors were decided in such ways that the probabilities of failure for each limit state are maintained at a uniform value. This was a disadvantage of the LFD philosophy, which would result in different levels of reliability for each limit state. The factors were also calibrated to previous design codes so that comparable results could be achieved. This means that structures designed using LRFD will not necessarily be weaker or stronger, just more consistent in their level of safety. Along with the new LRFD Bridge Design Specification came several fundamental changes to the pre-existing concrete girder design methods, adoption of the new specification had a slow start due to the complexities of implementing the new design and analysis methods, but gradually all state departments of transportation have adopted the LRFD Bridge Design Specification. However, much still needs to be considered to smoothly integrate from previous LFD design procedures and methods to LRFD design procedures and methods and safeguard against future difficulties and even conflicting design practice.

The California Department of Transportation (Caltrans) had experienced the transition to the LRFD specification with making numerous own modifications to the AASHTO specifications. The modification to the AASHTO specifications is called California Amendments to AASHTO LRFD. The main purpose for California Amendments to AASHTO LRFD Bridge Design Specifications is not only to add adequate California standardized design, of many years, to current AASHTO LRFD Specifications, but also to modify AASHTO LRFD Specifications based on previous successful California bridge design practices to efficiently deliver transportation projects, and to promote quality services while seeking creative solutions and taking intelligent risks. Caltrans experts on subject matters have gone through entire AASHTO specifications and made enormous decisions on what design practices should be retained and what should be changed according to California bridge design practice. Much of this is done to safeguard the state against any conflicts that may arise between past and future designs. Additionally, engineering resources such as software and design aids must be developed and placed into service through the past years. Based on many research studies and past experience, Caltrans amended AASHTO LRFD Specifications every four years based on current California bridge design practices and recent research results, which include areas of live loads, load distribution factors, analysis methods, load factors for ultimate strength combinations, prestress losses, and application of modified compression field theory for shear design. Some amendments reflect the modifications or changes resulting from California precast bridge design practices.

CALIFORNIA MAJOR AMENDMENTS TO LRFD ON CONCRETE GIRDER BRIDGES

The main purpose for California Amendments to LRFD is not only to add adequate California standardized design to current AASHTO LRFD Specifications, but also to modify AASHTO LRFD Specifications based on previous successful California bridge design practices. California is well-known for its using concrete bridge structures, especially Cast-In-Place Prestress Post-Tensioned Box Girder bridges. But some parts of current AASHTO LRFD Specifications are based on research results from precast prestressed girder structures. Therefore, amendment to AASHTO LRFD Specifications has to be made to reflect the modifications or changes according to California bridge design practices. At the same time, the code modifications affect the design of precast girders in California.

The section here concentrates showing specifications comparison between LRFD and California Amendments to LRFD. The main purpose is to share the information of California Amendments to LRFD with other State DOTs and PCI members.

The following list highlights the parts of California Amendments to LRFD that are related to concrete bridge design.

AASHTO LRFD SPECIFICATION SECTION 3 --- LOADS AND LOAD FACTORS

Loads and load factors changes will affect the precast bridge design. The most significant amendments of this section are shown as follows:

- Revise Table 3.4.1-1
- "low boy" truck configuration is a mandatory load, which may control negative bending serviceability in two-span continuous structures with 20- to 60-ft span lengths.
- Add California P15 truck as the permit vehicle
- Multiple presence factor of permit vehicle for one loaded lane is 1.0, instead of 1.2

- Dynamic Load Allowance (IM) for California P15 truck under strength II limit state is 25%, instead of normal 33%.

Revise Table 3.4.1-1 as follows:

Load Combination Limit State	DC DD DW ~~EL~~ EH EV ES PS CR SH	HL93 IM CE BR PL LS	Permit **IM** CE	WA	WS	WL	FR	TU ~~CR~~ ~~SH~~	TG	SE	EQ IC CT CV (use only one)
STRENGTH I	γ_p	1.75	**0.0**	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH II- **DF, LVR,SUB**	γ_p	**0.0**	1.35	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH III	γ_p	**0.0**	**0.0**	1.0	1.4	**0.0**	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
STRENGTH IV EH, EV,EL ES, DW, DD	γ_p	**0.0**	**0.0**	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	**0.0**	**0.0**	**0.0**
DC only	1.5										
STRENGTH V	γ_p	1.35	**0.0**	1.0	0.4	1.0	1.0	0.50/ 1.20	γ_{TG}	γ_{SE}	**0.0**
EXTREME EVENT I	~~γ_p~~ **1.0**	~~γ_{EQ}~~ **0.0**	**0.0**	1.0	**0.0**	**0.0**	1.0	**0.0**	**0.0**	**0.0**	1.00 (EQ)
EXTREME EVENT II	~~γ_p~~ **1.0**	0.5	**0.0**	1.0	**0.0**	**0.0**	1.0	**0.0**	**0.0**	**0.0**	1.00 (IC or CT or CV)
SERVICE I	1.00	1.00	**0.00**	1.00	0.30	1.0	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	**0.0**
SERVICE II	1.00	1.30	**0.00**	1.00	**0.0**	**0.0**	1.0	1.00/ 1.20	**0.0**	**0.0**	**0.0**
SERVICE III	1.00	0.80	**0.00**	1.00	**0.0**	**0.0**	1.0	1.00/ 1.20	γ_{TG}	γ_{SE}	**0.0**
SERVICE IV	1.00	**0.00**	**0.00**	1.00	0.70	**0.0**	1.0	1.00/ 1.20	**0.0**	1.0	**0.0**
FATIGUE I—	**0.00**	0.75	**0.00**	**0.00**	**0.00**	**0.0**	**0.0**	**0.00**	**0.0**	**0.0**	**0.00**

Table 3.4.1-1 – Load Combinations and Load Factors

Modify Table 3.4.1-1 as follows:

Table 3.4.1-2 (excerpts)	Load Factor	
Type of Load	Maximum	Minimum
DC: Component and Attachments; **CR, SH**	1.25	0.90
~~***EL*: Locked-in Erection Stresses**~~ *PS: Secondary Force from Post-Tensioning*	1.00	1.00

BACKGROUND

The current *AASHTO LRFD Specifications* (Article 3.4.1 paragraph six) require a load factor of 1.2 on *CR/SH/TU* deformations, and 0.5 on other *CR/SH/TU* force effects.
The lower value had been rationalized as dissipation of these force effects over time, particularly in the columns and piers. [Caltrans' *Memo to Designer* 6-1 Column Analysis Considerations" similarly recommends a 25% reduction in *TU*.]
The *AASHTO Guide Specs for Segmental Bridges* (1989, 1999) used a value of 1.0 when factoring creep and shrinkage. Application of the factor is assumed to be for deformations and force effects, super- and substructure. New software packages can better analyze changes in material properties over time, making the arbitrary redistribution or dissipation of *CR* and *SH* not appropriate or necessary, in the opinion of some engineers.
Assigning load factors for creep and shrinkage is not straight-forward because *CR*, *SH* are "super-imposed deformations" i.e. force effects due to a change in material properties that cause a change in the statical system. For safety and simplicity in design, they are treated as loads--despite not being measurable at t = 0. However, behavior is nonlinear and application of the load factor must also be considered (Item 9). Some software will run service load analysis twice: once with and once without *CR*, *SH* effects. The *CR* and *SH* can then be isolated by subtracting the two runs, and factored. Other software will couple the *CR* and *SH* with the dead load, giving you a shrinkage- or creep adjusted dead load i.e. a load factor different than $\_{DC}$ isn't possible. The proposed compromise is to assign creep and shrinkage the same load factor as the *DC* loads, but permit a factor of 1.0 if the project-specific creep coefficient can be determined and is then used in the linear analysis software. This approach was taken on the San Francisco – Oakland Bay Bridge. A new load *PS* has been defined for "secondary post-tensioning" (previously under *EL*). Secondary prestressing forces i.e. secondary moments are the force effects in continuous members, as a result of continuous post-tensioning. In frame analysis software, the secondary moments are generally obtained by subtracting the primary (*P*e*) from the total PS moments. Alternatively, the support reactions that can be developed when prestressing, are used. A factor of 1.0 is appropriate because *PS* can increase or decrease the factored total load at a section. Secondary prestressing forces tend to diminish when increased loading causes the structure to behave inelastically. For fixed columns, Caltrans' *Memos to Designer* 6-1 "Column Analysis Considerations," suggests a 50% reduction in PS force effects given the elasto-plastic characteristics of the soil surrounding the foundations. The definition of *EL* is revised to include jacking apart of cantilevers in segmental construction, along with steel girders with prestressed components (Article 3.4.1, par.12), and to accommodate analysis of any existing structures when construction affects can be defined.

3.6.1.3.1
Add a 4th bullet as follows:

- For both negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 100 percent of the effect of two design tandems spaced anywhere from 26.0 ft. to 40 ft. from the lead axle of one tandem to the rear axle of the other, combined with the design lane load specified in Article 3.6.1.2.4.

C3.6.1.3.1
Revise paragraph three as follows:

The notional design loads were based on the information described in Article C3.6.1.2.1, which contained data on "low boy" type vehicles weighing up to about 110 kip. ~~Where multiple lanes of heavier versions of this type of vehicle are considered probable, consideration should be given to investigating negative moment and reactions at interior supports for pairs of the design tandem spaced from 26.0 ft. to 40.0 ft. apart, combined with the design lane load specified in Article 3.6.1.2.4. One hundred percent of the combined effect of the design tandems and the design lane load should be used.~~ In California, side-by-side occurrences of the "low boy" truck configuration are routinely found. This amendment is consistent with Article 3.6.1.2.1, will control negative bending serviceability in two-span continuous structures with 20- to 60-ft span lengths, and should not be considered a replacement for the Strength II Load Combination.

Add a new Article as follows:

3.6.1.8 Permit Vehicles

3.6.1.8.1 General

Permit design live loads, or P loads, are special design vehicular loads. The weights and spacings of axles and wheels for the overload truck shall be as specified in Figure 3.6.1.8.1-1.

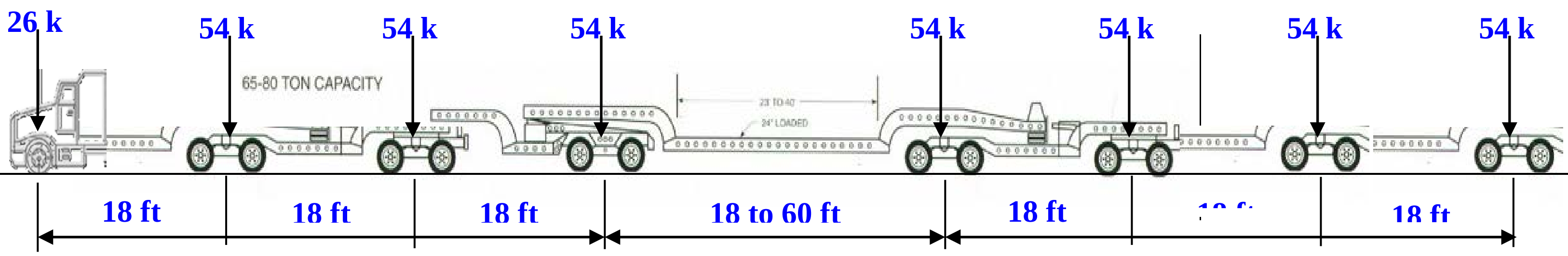

Figure 3.6.1.8.1-1 California P15 truck

3.6.1.8.2. Application

The permit design live loads shall be applied in combination with other loads as specified in Article 3.4.1. Axles that do not contribute to the extreme

Dynamic load allowance shall be applied as specified in 3.6.2.

Multiple presence factors shall be applied as specifed in Article 3.6.1.1.2. However, when only one lane of permit is being considered, the MPF for one loaded lane shall be 1.0.

3.6.2 Dynamic Load Allowance: IM

3.6.2.1 General

Revise paragraph as follows:

Unless otherwise permitted in Articles 3.6.2.2 and 3.6.2.3, the static effects of the design truck, ~~or~~ design tandem, or permit vehicle other than centrifugal and braking forces….

Revise Table 3.6.2.1-1 as follows:

Component	*IM*
Deck Joints—All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit State	15%
• Strength II Limit State	25%
• All Other Limit States	33%

C3.6.2.1

Revise paragraphs four and five as follows:

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 1 is the product of 4/3 and the basic 25 percent. California removed the 4/3 factor for Strength II because a lane load isn't a part of the design permit vehicle used. Furthermore, force effects due to shorter permit vehicles approach those due to the HL93. The HL93 tandem*1.33 + lane generally has a greater force effect than that due to the P15 on short-span bridges.

AASHTO LRFD SPECIFICATION SECTION 5 --- CONCRETE STRUCTURES

Most of changes and modifications of current AASHTO LRFD Specifications are related to Cast-In-Place Prestress Post-Tensioned Box Girder bridge design. But some of amendments still affect the precast bridge design. The most significant amendments of this section related to precast bridge design are shown as follows:

- Change the size of PT ducts to 0.5 (from 0.4) times the girder web thickness for spliced precast girder bridges
- Add the resistance factor as 0.95 for spliced precast girders
- Ensure that the net tensile strain in the extreme tensile steel is not less than 0.004 which is equivalent to the previously established practice of limiting the maximum reinforcement ratio in a cross section to be not greater than 0.75 times the balanced reinforcement ratio
- Set maximum jacking stress as $0.75f_{pu}$, instead of $0.90f_{py}$ for post-tensioning
- Set Zero Tension stress limit for components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.
- Adjust the values for K and u based on span length
- Add the time-dependent lump sum prestress loss as 20 ksi for post-tensioning
- Change maximum total debonded strands to 33% from 25%
- Change maximum debonded strands to 50% from 40% in any horizontal row

5.4.6.2 Size of Ducts

Modify the second paragraph:

The size of ducts shall not exceed ~~0.4~~ 0.5 times the least gross concrete thickness at the duct.

BACKGROUND

For CIP Post-Tensioned Box Girder, girder web width normally is 12” or larger, there is no current issues for the max. size limit of 4.8” (0.4x12”) based on this article. But for post-tensioned spliced precast girders, the girder web width is 8”. If we limit the size of ducts to 40% of web width, which the max. size of ducts is limited to 3.2” OD. It only can accommodate 12.6 PT systems. Designers need to use more tendons and may not be able to fit the tendons (and anchorages) in the girder. Therefore, the current code limits the prestress force for the girders and also limits bridge span length capacity.

PCI Bridge Design Manual Chapter 11 “Extending Spans” states that 40% of the web width requirement has been traditionally used to size webs for internal ducts in segmental bridge construction. Historically, the requirement has not existed and has not been observed for I-beams. Also, a number of other States have used duct size over 40% of web width with no reported problems.

5.5.4.2.1 Conventional Construction

Insert the following under the first bullet:

- **For tension-controlled cast-in-place prestressed concrete sections and spliced precast girder sections as defined in Article 5.7.2.1……………………..…..0.95**

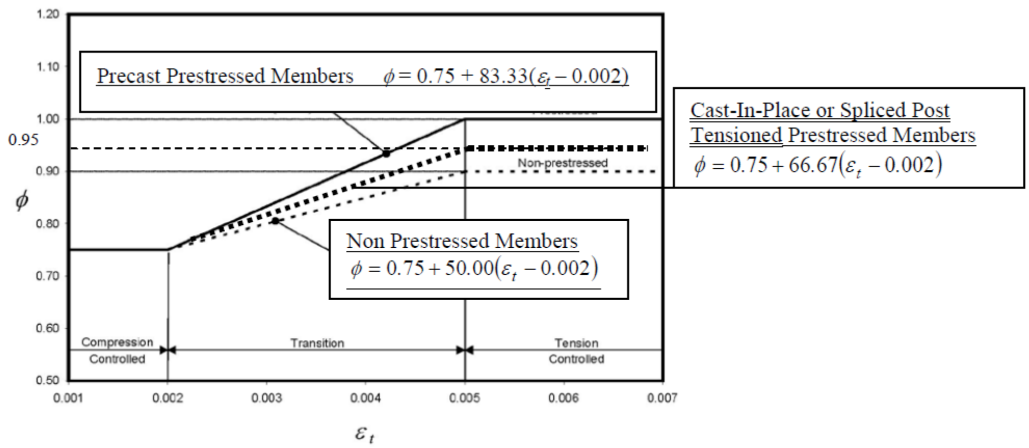

Figure C5.5.4.2.1-1 – Variation of ϕ with net tensile strain ε_t for Grade 60 reinforcement and for prestressed~~ing~~ members ~~steel~~.

BACKGROUND

The code did not provide a resistance factor for CIP PT Box Girders. California uses 0.95 for CIP PT Box Girders if it is tension-controlled members in all cases. This amendment adds it as text and also to the Figure C5.5.4.2.1. The reason 0.95 is used for CIP PT Box Girders is that quality control for it is between that of CIP RC which has a value of 0.9 and PC PS which has a value of 1.0. Also, the code did not provide a resistance factor for spliced precast Girders. Currently California Amendment uses 0.95 for CIP PT Box Girders if it is tension-controlled members in all cases. This amendment adds 0.95 resistance factor for PT spliced precast girder as well. Based on CA practice, we use a resistance factor of 1.0 for precast prestressed concrete sections.

5.7.2.1 General

Revise the 11th "bullet" as follows:

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. For non-prestressed concrete flexural sections including girders, bent caps, and deck slabs, the net-tensile strain in the extreme tension steel shall not be less than 0.004.

C5.7.2.1

Revise the 4th Paragraph as follows:

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than 0.005), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Ensuring that the net tensile strain in the extreme tensile steel is not less than 0.004 is equivalent to the previously established practice of limiting the maximum reinforcement ratio in a cross section to 0.75 times the balanced reinforcement ratio. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

BACKGROUND

Clarify why we are limiting the NTS value.

5.8.2.9 Shear Stress on Concrete

Revise the 2nd paragraph as follows:

In determining the web width at a particular level, one-half the diameters of ungrouted ducts up to a maximum of 2" or one-quarter the diameter of grouted ducts up to a maximum of 1" at that level shall be subtracted from the web width for spliced precast girder. It is not necessary to reduce b_v for the presence of ducts in fully grouted cast-in-place box girder frames.

BACKGROUND

Caltrans' bridge inventory of CIP PT Box Girders was designed and built without reducing *bv* for the presence of ducts when fully grouted Cast-in-place box girder frames with grouted ducts are integral with surrounding concrete. For the strength limit state stage, it is un-necessary to reduce ¼ of the diameter of the duct size. Current AASHTO design code applies to spliced precast girder because its narrow web width, even though for Caltrans CIP PT Box Girders, it is un-necessary to reduce ¼ of the diameter of the duct size, as stated in the previous 2008 amendments. For PT spliced precast girders, since designer won't know the size of ducts until shop drawing review, it is necessary to provide the max. duct size deduction for determining web width of shear design. Per CA Amendment 5.4.6.2 “Size of Ducts”, the size of ducts shall not exceed 0.5 times the least gross concrete thickness at the duct. Currently, the max. web width of Caltrans Std. precast girder is 8”. Therefore, the max. duct size shall not be more than 4”. In item 1, one-half the diameters of ungrouted ducts is up to a maximum of 2” and one-quarter the diameter of grouted ducts is up to a maximum of 1”

5.8.3.4.3 Simplified Procedure for Prestressed and Nonprestressed Sections

Delete entire Article and revise as follows:

<u>Article 5.8.3.4.3 “Simplified Procedure for Prestressed and Nonprestressed Sections” shall not be used.</u>

BACKGROUND

Most engineers are now used to Article 5.8.3.4.2 “General Procedure”, the “beta and theta” method of estimating shear resistance. Adding another option to calculate shear capacity will hinder LRFD implementation. Furthermore, NCHRP Panel Member and Caltrans T10 member Sue Hida refers to a comparison of the shear estimates using the various methods (Table 8 of reference, below) with those from Response2000, and questions the accuracy of the “*Vci/Vcw*” method. Also, more options mean more solutions. It is impractical from a management standpoint when designer and checker come out with different shear capacities and different stirrup designs based on the same Specifications.

Table 5.9.3-1 Stress Limitations for Prestressing Tendons

Revise Table 5.9.3-1 as follows:

Condition	Tendon Type		
	Stress-Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Prior to Seating --- short-term $fpbt$ may be allowed	$0.90f_{py}$	$0.90\ f_{py}$	$0.90f_{py}$
Immediately prior to transfer ($fpbt$)	$0.70\ f_{pu}$	$0.75\ f_{pu}$	---
At service limit state after all losses(f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$
Post-tensioning			
~~Prior to Seating short-term f_{pbt} may be allowed~~	~~$0.90\ f_{py}$~~	~~$0.90\ f_{py}$~~	~~$0.90\ f_{py}$~~
Maximum Jacking Stress–short-term f_{pbt} may be allowed	$0.75\ f_{pu}$	$0.75\ f_{pu}$	$0.75\ f_{pu}$
At anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.70\ f_{pu}$	$0.70\ f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70\ f_{pu}$	$0.74\ f_{pu}$	$0.70\ f_{pu}$
At service limit state after losses(f_{pe})	$0.80f_{py}$	$0.80f_{py}$	$0.80f_{py}$

BACKGROUND

Caltrans design practice limits 75% of fpu when stressing. 0.90fpy (which fpy is about 0.9fpu for low relax strands) equals about 0.81fpu for low relax strands. Therefore, 0.75fpu max limit is necessary for initial jacking stress for post-tensioning. Current AASHTO LRFD Table 5.9.3-1 does not specify stress limit for prior to seating for pretensioning. Therefore, adding one row for pretensioning to set up the stress limits is necessary.

Table 5.9.4.2.2-1 Tensile Stress Limits in Prestressed Concrete at Service Limit State After Losses, Fully Prestressed Components

Revise Table as follows:

Bridge Type	Location	Stress Limit
Segmental and Non-Segmental Bridges	Precompresssed Tensile Zone Bridges, Assuming Uncracked Sections—components with bonded prestressing tendons or reinforcement, subjected to permanent loads only.	No tension
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections • For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions, and are located in Caltrans' Environmental Areas I or II. • For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans' Environmental Area III. • For components with unbonded prestressing tendons.	$0.19\sqrt{f'_c}$ (ksi) $0.0948\sqrt{f'_c}$ (ksi) No tension
Segmentally Constructed Bridges	*(no changes)*	*(no changes)*

BACKGROUND

Caltrans has a more stringent service limit state requirement than AASHTO when determining the prestressing force. No tension is allowed for the PS concrete members subject to permanent loads. Also, under environmental area I and II condition, Caltrans uses AASHTO $0.19\sqrt{f'c}$ (ksi) tensile stress limit. But under environmental area III condition, Caltrans uses $0.0948\sqrt{f'c}$ (ksi) tensile stress limit.

C5.9.5.2.2b

Add a new last Paragraph:

For tendon lengths greater than 1200 feet, investigation is warranted on current field data of similar length frame bridges for appropriate values of μ.

Revise as follows:

Table 5.9.5.2.2b-1 Friction Coefficients for Post-Tensioning Tendons

Type of Steel	Type of Duct	K /ft	μ
Wire or strand	Rigid and semi-rigid galvanized metal sheathing	~~0.0002~~	~~0.15-0.25~~
	Tendon Length:		
	< 600 ft	0.0002	0.15
	600 ft < 900 ft	0.0002	0.20
	900 ft < 1200 ft	0.0002	0.25
	> 1200 ft	0.0002	>0.25
	Polyethylene	0.0002	0.23
	Rigid steel pipe deviators for external tendons	0.0002	0.25
High-strength bars	Galvanized metal sheathing	0.0002	0.30

BACKGROUND

The AASHTO Specifications do not adjust the values for K and u based on span length. Experience in California has shown these values to be appropriate, with the max span length limitation.

5.9.5.3 Approximate Estimate of Time-Dependent Losses

Add a new last paragraph:

For cast-in-place post-tensioned box girder bridges, the approximate estimate of time-dependent losses may be taken as a lump sum value of 20 ksi.

C5.9.5.3

Add a new last paragraph:

The expressions for estimating time-dependent losses in Table 5.9.5.3-1 were developed for pretensioned members and should not be used for post-tensioned structures. Research performed by the University of CA, San Diego (SSRP-11/02) indicates time-dependent losses for cast-in-place post-tensioned box girder bridges are lower than previously expected. A parametric study using equations presented in the aforementioned research indicates losses may range from 11 ksi to 21 ksi. The variance is due to several parameters, such as relative humidity, area of non-prestressing steel and strength of concrete.

5.11.4.3 Partially Debonded Strands

Revise the 2nd and 3rd Paragraphs as follows:

The number of partially debonded strands ~~should~~ shall not exceed ~~25~~ 33 percent of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed ~~40~~ 50 percent of the strands in that row.

BACKGROUND

Current AASHTO code’s limit of numbers of debonded strands made it very difficult or even impossible for design of mid and long span girder using totally debonding method (no harp strands). Since debonding is a safer construction method. Several other States increase the limit of numbers of debonded strands. By discussing with California precast industry and from previous experience, minor increase the limit will help both design and construction more effective to use debonded strands. “should” needs to be changed to “shall” per specs language.

SUMMARY

1. The paper intends to serve the purpose of giving other State policy makers and bridge engineers a general sense what California Amendments are and to open the discussion among the different State policy makers and PCI bridge members if some of the California Amendments are needed to be adopted by AASHTO and its pros and cons to the precast industry.
2. California Amendments for AASHTO LRFD Specifications Section 3: Loads and Load Factors and Section 5: Concrete Structures are illustrated in this paper because changes and modifications of these two sections have most impacts on the precast girder bridge design.
3. Although most California Amendments to LRFD are based on for Cast-In-Place Prestress Post-Tensioned Box Girder to reflect the California bridge design practices, some code amendments affect the design of precast girder in California, which are listed here.
4. A study has been done to measure how much impact California Amendments has of the modifications of the AASHTO LRFD Specifications. The summaries and conclusions of the study will be presented at the PCI/NBC Bridge Conference.

REFERENCES

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