LRFD IMPACT ON PRECAST BRIDGE DESIGN AND CONSTRUCTION IN CALIFORNIA

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ABSTRACT

Effective April 1, 2006, all current Caltrans bridge design projects shall conform to the AASHTO LRFD Bridge Design Specifications as amended by Caltrans. What will this implementation of LRFD impact the current precast bridge design and construction in California? This paper will not only illustrate the code and design difference between LFD and LRFD, but also show the latest California Amendments to AASHTO LRFD Specifications for typical types of Caltrans precast/prestressed bridges. In addition, the paper will emphasize the design/construction limitation and the design/construction improvement for LRFD precast bridge design. Case study will be illustrated for California Standard I-girders and Bulb-Tee girders. A comparison of major specifications differences between the LFD and LRFD, and California Amendments to LRFD is discussed. Although the precast girder bridge design process does not change much, the design methods, which include live load, load distribution factors, load factors for ultimate strength combinations, prestress losses, and application of modified compression field theory for shear capacity calculations have changed. Two sets of numerical design examples regarding to California Standard I-girders and Bulb-Tee girders are conducted. Design results based on both LFD and LRFD Specifications are presented. It is concluded that use of LRFD Specifications leads large P-Jack forces, higher concrete strength, and bigger flexure moments. LRFD Shear design needs to be evaluated case by case since both shear loads and shear capacity calculation method have changed extensively. The purpose of the study is to illustrate the level of difference between LFD and LRFD with amendment in precast bridge design and construction in California. The goal of this paper is to give bridge engineers and precasters a general sense what changes LRFD will bring to us.

Keywords: Precast Girder; Precast/Prestressed Bridges; LRFD; LFD; California Amendments to AASHTO LRFD

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 the early half of the 20th century, Allowable Stress Design (ASD) was the primary design methodology. It utilized principles of allowable stress to formulate stress limits based upon an elastic assumption. The allowable stress was determined by the limiting stress, either yielding or ultimate, divided by a factor of safety. Using larger factors of safety, determined from experience, compensated for variability of loads and material strengths. The disadvantage of this design philosophy is that load variability fluctuates with each type of loading. In addition, each failure mode has a different level of risk. More variability is introduced with different building materials, construction quality, and maintenance consistency creating a need for numerous factors depending upon any given situation. Since the factors were established from engineering judgment and experience, reliability could only progress from mistakes. Unfortunately, it took several notable structural failures to reveal the deficiencies of the structural code resulting in a shift of design philosophy in the 1960's to a more rationale-based approach.

Load Factor Design (LFD) is a method of applying load factors to different types of loads along with resistance factors or strength reduction factors to different material strengths. A successful design is achieved when the factored loads do not exceed the factored strengths. LFD was developed from the deficiencies of ASD and advancements in first-order reliability analysis. It was determined that probability models could be used to predict risks, variability, and uncertainties more accurately. In 1979, Ellingwood, Galambos, MacGregor, and Cornell, under the American National Standards Committee A58, took on the arduous task of developing load criteria for all types of building construction. This would ensure load compatibility when more than one type of construction material was used. In addition to load factors, they developed a methodology of determining material based resistance criteria. Their worked spawned the LFD design philosophy, which has been adopted since by most major structural design codes and specifications.

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.

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. 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 was slow throughout the United States due to the complexities of implementing the new design and analysis methods. Eventually, AASHTO relented to pressure and published another edition of its Standard Specification in 1996 and again in 2002. AASHTO does not plan to publish any further editions of the Standard Specification, and the Federal Highway Administration (FHWA) has set the target year of 2007 for the complete adoption of the LRFD Specification amongst all state departments of transportation. With only one year remaining, many states are at their final stage to facilitate the adoption of the LRFD Bridge Design Specification. However, much still needs to be considered to smoothly integrate from previous LFD design procedures and methods to new LRFD design procedures and methods and safeguard against future difficulties and even conflicting design practice. Currently, quite a lot bridge engineers with strong LFD design experience are still adjusting the changes the new LRFD brings. Some past design practice and rule of thumbs may not be suitable for new LRFD design code.

The California Department of Transportation (Caltrans) is currently experiencing the transition to the LRFD specification. Although the process to investigate the acceptance the new specification and adapting it to current design practices happened for quite a long time, it was until recent years that Caltrans put LRFD implementation on the highest priority, formed a special LRFD Task Group and started to make numerous own modifications to the AASHTO specifications. The modification to the AASHTO specifications is called California Amendments to AASHTO LRFD. The LRFD Task Group had to go through entire AASHTO specifications and must make 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 before the transition begins. Once these preparations are completed, the engineering staff needs to be educated and trained in the new design philosophy and methods. Effective April 1, 2006, all Caltrans current bridge design projects shall conform to the AASHTO LRFD Bridge Design Specifications as amended by Caltrans. This requirement applies to structure design of all components. Full implementation of the AASHTO LRFD Bridge Design Specifications for substructure design is scheduled for April 1, 2007.

CALIFORNIA AMENDMENTS TO LRFD ON PRECAST 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 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.

The following list highlights the parts of California Amendments to LRFD that are related to precast prestressed girder 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 DC only	γ_p 1.5	**0.0**	**0.0**	1.0	**0.0**	**0.0**	1.0	0.50/ 1.20	**0.0**	**0.0**	**0.0**
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**
FATIGUE II?		1.50?									
FATIGUE III?			**(P9 truck?)**								

Table 3.4.1-1 – Load Combinations and Load Factors

Modify Table 3.4.1-1 as follows:

Table 3.4.1-2 (excerpts) Type of Load	Load Factor	
	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

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.

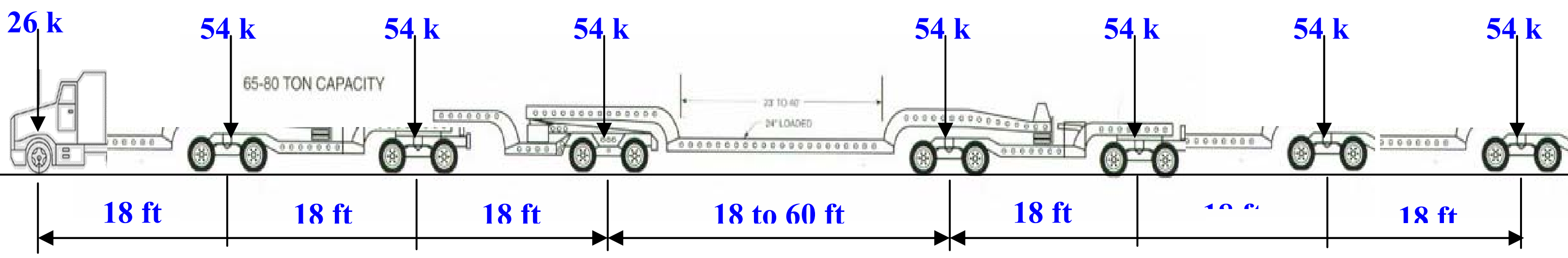


Figure 3.6.1.8.1-1 California P15 truck

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:

- Add a simplified procedure to calculate β and θ for shear design
- Set maximum jacking stress as $0.75f_{pu}$, instead of $0.90f_{py}$
- Set Zero Tension stress limit for components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.
- Change maximum total debonded strands to 33% from 25%
- Change maximum debonded strands to 50% from 40% in any horizontal row

Add a new Article as follows:

5.8.3.4.3 Simplified Procedure

In lieu of Table 5.8.3.4.2-1, β and θ may be evaluated as follows for vertical stirrups:

$$\beta = \frac{4.8}{1+1500\varepsilon_x} \qquad (5.8.3.4.3\text{-}1)$$

$$\theta = 29 + 7000\varepsilon_x \qquad (5.8.3.4.3\text{-}2)$$

C5.8.3.4.3

Equations 1 and 2 were developed by Micheal Collins and adopted into the Canadian Specs (2004). The effect on resulting values for concrete shear resistance was found to be somewhat greater than the sectional method, but less than those per the 2002 *AASHTO Standard Specifications* on which *Caltrans BDS* is based.

5.9.3 Stress Limitations for Prestressing Tendons

Revise Table 5.9.3-1 as follows:

~~Prior to Seating~~	$0.90f_{py}$	~~0.90fpy~~	$0.90f_{py}$

Maximum Jacking Stress	$0.90f_{py}$	$0.75f_{pu}$ (see note)	$0.90f_{py}$

Add a note below Table 5.9.3-1 as follows:

Note: For longer frame structures, tensioning to $0.90f_{py}$ for short periods of time prior to seating may be permitted to offset seating and friction losses provided the stress at the anchorage does not exceed the above value (low relaxation strand, only).

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

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	
	• For components with bonded prestressing tendons or reinforcement, subjected to permanent loads, only.	**No tension**
	• 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.	$0.19\sqrt{f'c}$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions, and are located in Caltrans Environmental Area III.	$0.0948\sqrt{f'c}$ (ksi)
	• For components with unbonded prestressing tendons.	No tension

5.11.4.3 PARTIALLY DEBONDED STRANDS

Revise paragraphs two and three as follows:

The number of partially debonded strands should 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.

LRFD vs. LFD CASE STUDY FOR CALIFORNIA STANDARD PRECAST I-GIRDER AND BULB-TEE GIRDER BRIDGES

The purpose of this case study is to illustrate the level of difference between LFD and LRFD with California amendment in California precast bridge design. Case study uses all California I-Girder and Bulb-Tee Girder sizes to cover the designable span range. Results of simply support span analysis are shown. Results of continuous spans are similar. Adequate software has been used for this case study. Conclusions are included. The goal of this study is to give bridge engineers and precasters a general sense what changes LRFD with California amendment will bring to us.

CALIFORNIA "I" GIRDER

Although the cast-in-place post-tensioned box girder is commonly used in California, the first prestressed concrete bridge constructed in California was a precast, prestressed "I" girder bridge built in the late 1950's. The California "I" girder has been in use in California for nearly 60 years. With bridge span lengths ranging from 50 feet to 120 feet, the California "I" girder could be used. Normally, the "I" girder has a depth-to-span ratio of approximately 0.055 for simple spans, and reduces to 0.050 for multi-span structures made continuous for live loads. This structure type has proven to be an excellent choice for rapid construction and widening existing structures. Since bridge widening are becoming more common in California in recent years, "I" girder has been widely used comparing with other type of structures. Once the deck is poured and the structural section becomes composite, there are no significant up or downward deflections that may transfer unwanted forces to the existing structure. Also, with no need for ground-supported falsework, precast girder construction usually takes far less time than cast-in-place, and the impact to the traveling public is minimal.

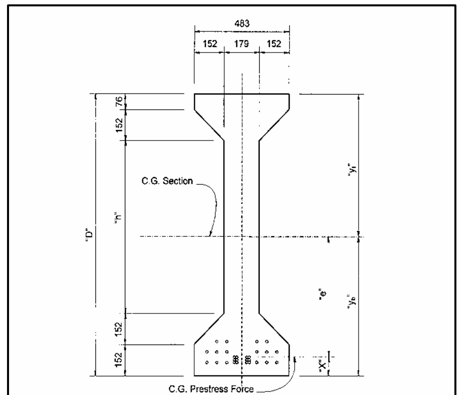
The precast industry in California has worked very closely with the California Department of Transportation for many years. Therefore, by working together, the precast manufactures are able to develop very high quality, cost effective products. Precast girders are economically competitive with cast-in-place structures, especially when there are many girders required, and those girders have nearly the same length and prestressing force required. Several California precast plants are capable of casting up to four girders in a single casting bed, and can achieve initial concrete strengths in excess of 6000 psi in a 14-hour period. With a production rate of up to four girders per day, using California "I" girder becomes more attractive as prolonged traffic congestion due to construction becomes more of an issue.

CALIFORNIA "I" GIRDER: LRFD vs. LFD

Along the current code change from LFD to LRFD, plus California Amendments to LRFD, the design of California "I" girder will change too. It is concluded that the precast girder bridge design process does not change much, the design methods, which include live load, load distribution factors, load factors for ultimate strength combinations, prestress losses, and application of modified compression field theory for shear capacity calculations have changed. A set of California "I" girder analysis and design are conducted here based on both LRFD and LFD codes. The case study illustrates the different results according to different specifications. Analysis results of P-Jack forces, numbers of strands, concrete strength, and moment ratios are summarized and compared by following charts and tables.

Design Parameters and Criteria:

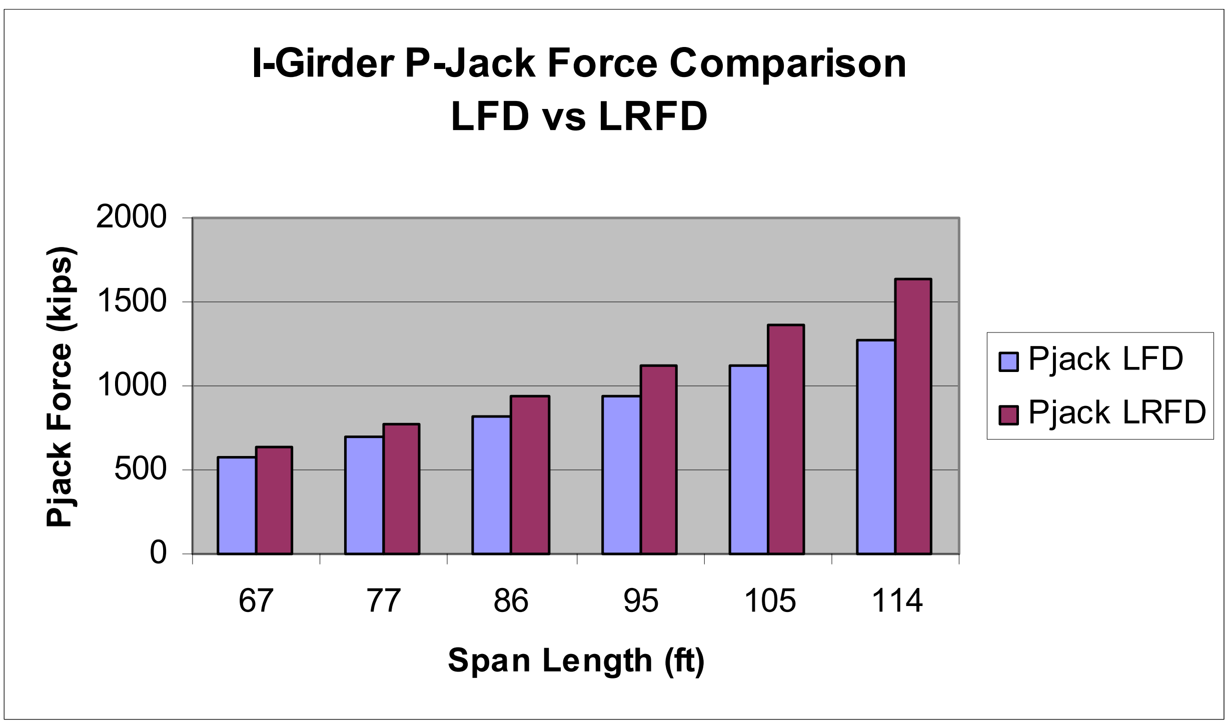
- Bridge Geometry: Simple Span with Span Length 67', 77', 86', 95', 105' and 114'
- Structure Type: Standard I-Girders, total 6 girders per span
- Girder Geometry:
  - D/L Ratio = 0.055
  - Girder Spacing = 1.5xD
- General Assumptions:
  - Deck Thickness: 7.5 to 8.0 inches
  - Barrier Weight, AC Weight
  - Haunch: 1 inch
  - Slab f'c=4 ksi
  - 270 ksi low-lax ps strands
  - PS Strand Harped at 0.4L
  - Unit: kips, feet



SECTION PROPERTIES

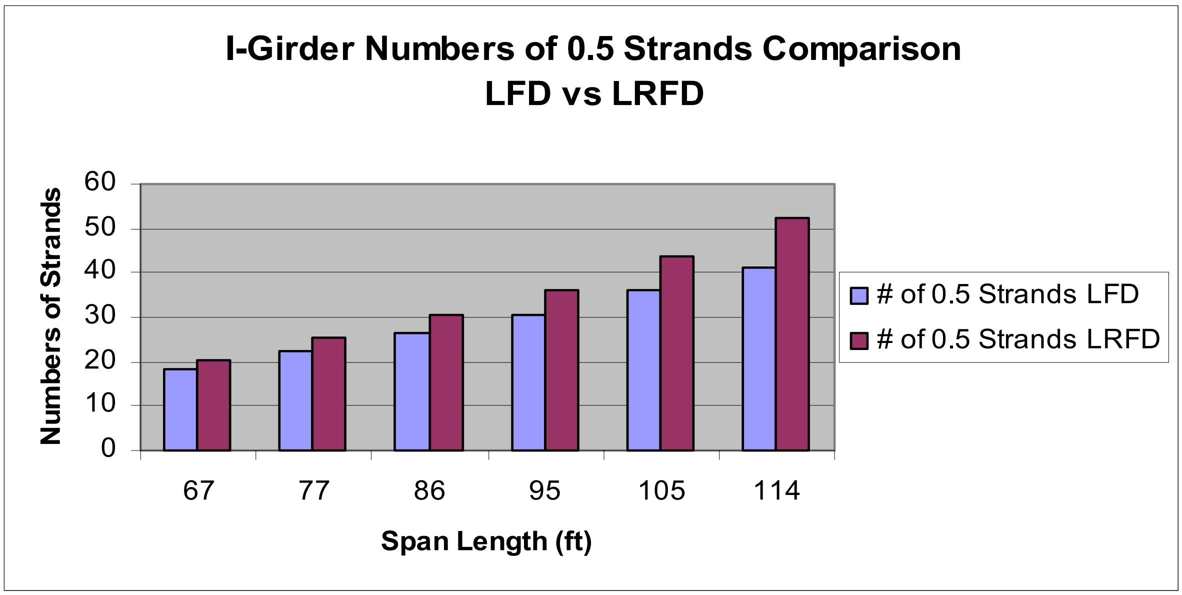
"D" (mm)	"h" (mm)	Area (mm^2)	I (mm^4)	y_b (mm)	y_t (mm)	S_b (mm^3)	S_t (mm^3)	r (mm)	Mass (N/m)	Wt. (Kg/m)
914	382	279 200	26 200 E6	435	479	60.3 E6	54.7 E6	306	6580	671
1067	535	306 500	39 800 E6	508	559	78.3 E6	71.1 E6	360	7223	736
1219	687	333 700	56 800 E6	580	639	97.9 E6	89.0 E6	413	7866	802
1372	840	361 000	77 800 E6	654	718	118.9 E6	108.4 E6	464	8509	867
1524	992	388 300	102 900 E6	728	796	141.4 E6	129.3 E6	515	9152	933
1676	1144	415 600	132 600 E6	802	874	165.3 E6	151.6 E6	565	9795	998

(1) P-Jacking Force Comparison



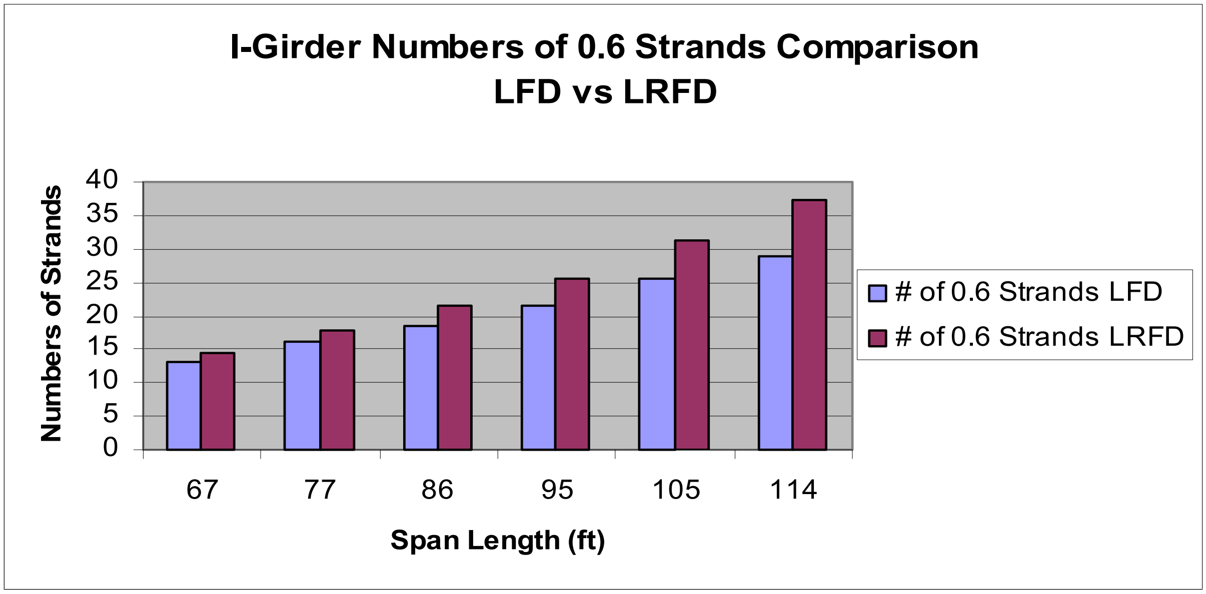
Girder Name	I-Girder 36	I-Girder 42	I-Girder 48	I-Girder 54	I-Girder 60	I-Girder 66
Span L (ft)	67	77	86	95	105	114
LFD	570	700	815	940	1120	1270
LRFD	630	780	940	1120	1360	1630
Increase	11%	11%	15%	19%	21%	28%

(2) Numbers of 0.5" dia. Strands Comparison



Girder Name	I-Girder 36	I-Girder 42	I-Girder 48	I-Girder 54	I-Girder 60	I-Girder 66
Span L (ft)	67	77	86	95	105	114
LFD	18	23	26	30	36	*41*
LRFD	20	25	30	36	*44*	*53*
Increase	2	2	4	6	8	12

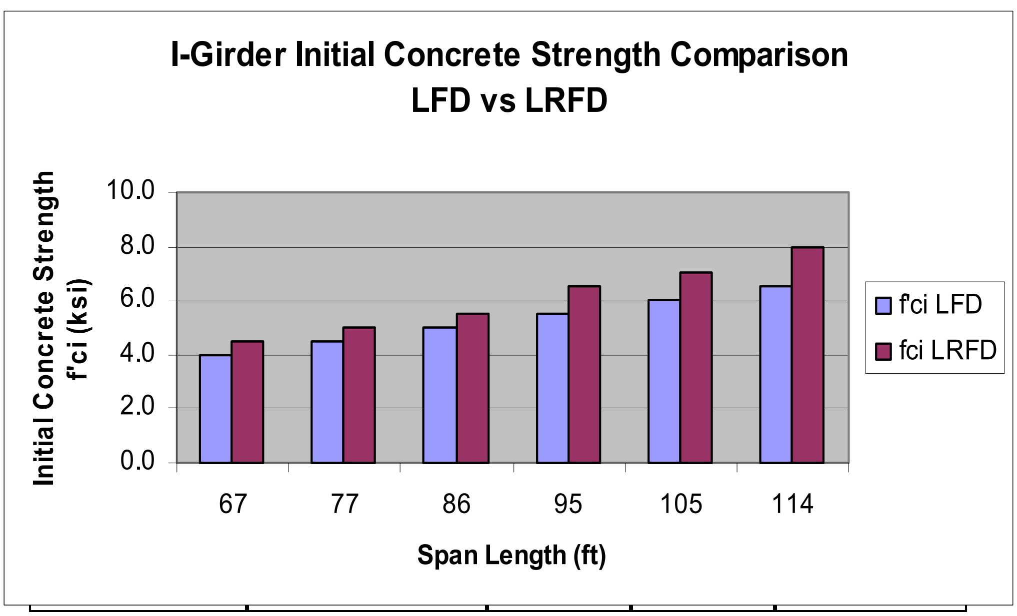
(3) Numbers of 0.6" dia. Strands Comparison



Girder Name	I-Girder 36	I-Girder 42	I-Girder 48	I-Girder 54	I-Girder 60	I-Girder 66
Span L (ft)	67	77	86	95	105	114
LFD	13	16	19	21	26	29
LRFD	14	18	21	26	31	37
Increase	1	2	2	5	5	8

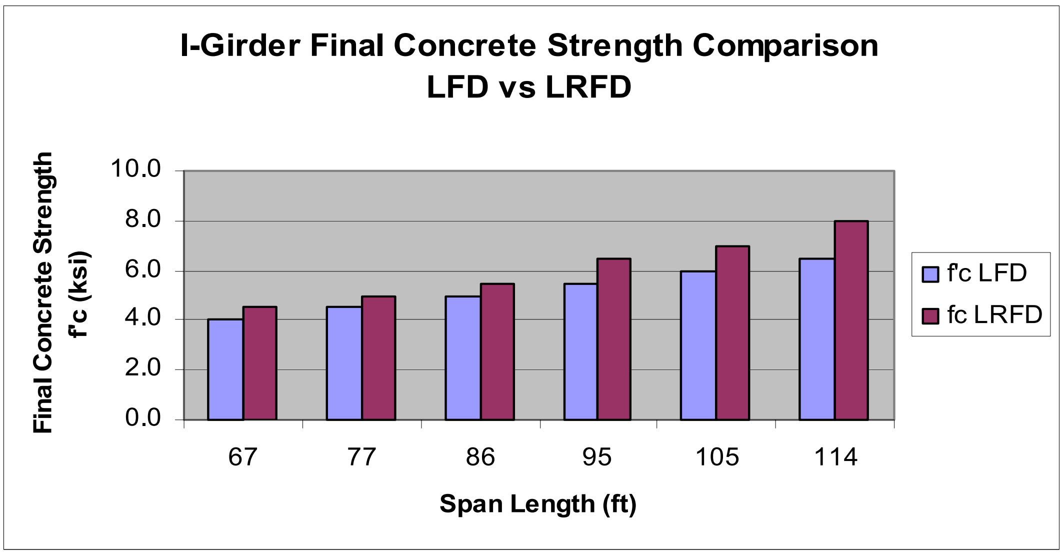
(4) California "I" girder maximum numbers of strands is 40 as shown. Therefore, using 0.5" dia. strands of "I" girder for span length over 100' is not adequate as shown in the table. Using 0.6" dia. strands are required for span length over 100'.

(5) Initial concrete strength comparison



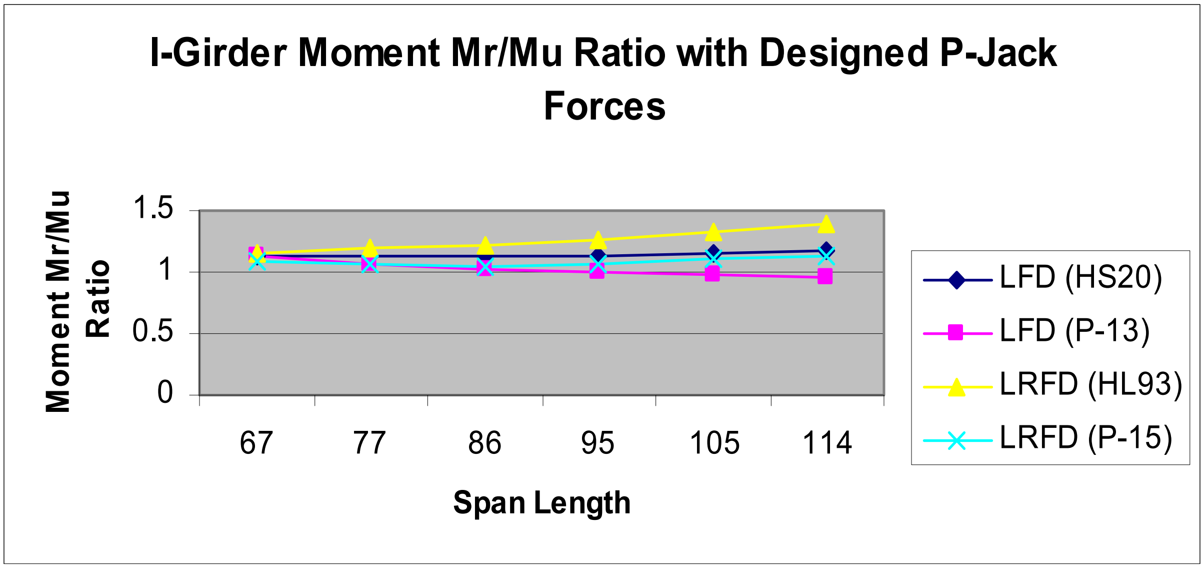
I-Girder 36	**67**	**4.0**	**4.5**	**13%**
I-Girder 42	**77**	**4.5**	**5.0**	**11%**
I-Girder 48	**86**	**5.0**	**5.5**	**10%**
I-Girder 54	**95**	**5.5**	**6.5**	**18%**
I-Girder 60	**105**	**6.0**	**7.0**	**17%**
I-Girder 66	**114**	**6.5**	**8.0**	**23%**

(6) Final concrete strength comparison



Girder Name	Span Length (ft)	LFD	LRFD	Increase
I-Girder 36	**67**	**4.0**	**4.5**	**13%**
I-Girder 42	**77**	**4.5**	**5.0**	**11%**
I-Girder 48	**86**	**5.0**	**5.5**	**10%**
I-Girder 54	**95**	**5.5**	**6.5**	**18%**
I-Girder 60	**105**	**6.0**	**7.0**	**17%**
I-Girder 66	**114**	**6.5**	**8.0**	**23%**

(7) Moment Mr/Mu ratio with designed P-Jack forces



Girder Name	Span L (ft)	LFD (HS20)	LFD (P-13)	LRFD (HL93)	LRFD (P-15)
I-Girder 36	67	1.12	1.12	1.16	1.08
I-Girder 42	77	1.14	1.07	1.19	1.07
I-Girder 48	86	1.13	1.02	1.22	1.05
I-Girder 54	95	1.14	0.99	1.27	1.06
I-Girder 60	105	1.16	0.98	1.33	1.10
I-Girder 66	114	1.17	0.96	1.40	1.13

CALIFORNIA “BULB-TEE” GIRDER

The California “Bulb-Tee” girder was relatively new to design engineers. It was first introduced in the mid 1990’s. The “Bulb-Tee” shape was introduced to compete with the cast-in-place box girder in bridges with span lengths in excess of 100 feet. When used as a fully pretensioned unit, girders up to 140 feet in length have been transported by truck to various locations in California. When post-tensioning is used to splice several girder segments together, span lengths in excess of 180 feet are possible. The depth-to-span ratio for fully pretensioned simple spans is approximately 0.050, and can be reduced to 0.045 when multiple spans are made continuous for live load. When spliced together with post-tensioning, depth-to-span ratios as low as the CIP post-tensioned box girder (0.040) can be achieved.

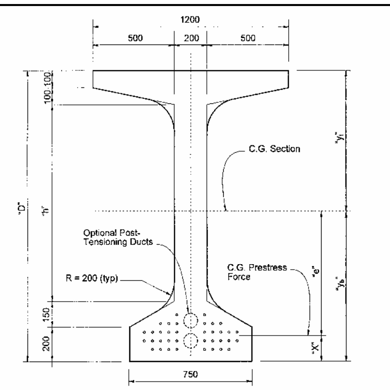
When compared to the California "I" girder, California "Bulb-Tee" girder has several benefits. The characteristics of the "Bulb-Tee" shape provide a larger section modulus, which often eliminates the need for harped prestressing strand. Harping strand is expensive. Also, it is dangerous because the hardware used to harp strand results an abrupt angle change in the strand pattern. Using straight strands is a much more desirable alternative than harping, by allowing up to 33 percent of the strands as debonded to control tensile stresses at the top fiber of the girder ends. The "Bulb-Tee" cross-section, due to significantly wider top and bottom flanges, has a larger lateral moment of inertia than the "I" shape. The increased stiffness in the weak direction requires minimal, if any, lateral bracing to prevent buckling failure during transportation.

CALIFORNIA "BULB-TEE" GIRDER: LRFD vs. LFD

As same as California "I" girder, a set of California "Bulb-Tee" girder analysis and design are conducted based on both LRFD and LFD codes. The case study shows the different results according to different specifications. Analysis results of P-Jack forces, numbers of strands, concrete strength, and moment ratios are summarized and compared by charts and tables.

Design Parameters and Criteria:

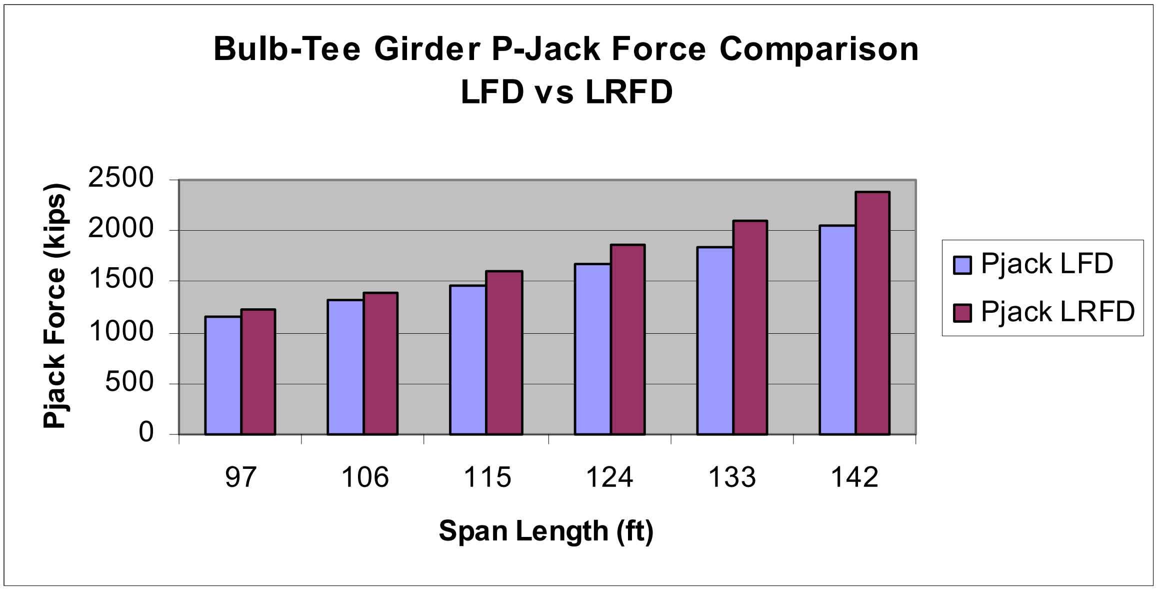
- Bridge Geometry: Simple Span with Span Length 97',106',115',124',133' and 142'
- Structure Type: Standard Bulb-Tee Girders, total 6 girders per span
- Girder Geometry:
  - D/L Ratio = 0.050
  - Girder Spacing = 1.5xD
- General Assumptions:
  - Deck Thickness: 8.0 to 8.5 inches
  - Barrier Weight, AC Weight
  - Haunch: 1 inch
  - Slab f'c=4 ksi
  - 270 ksi low-lax ps strands
  - PS Strand Harped at 0.4L
  - Unit: kips, feet



SECTION PROPERTIES

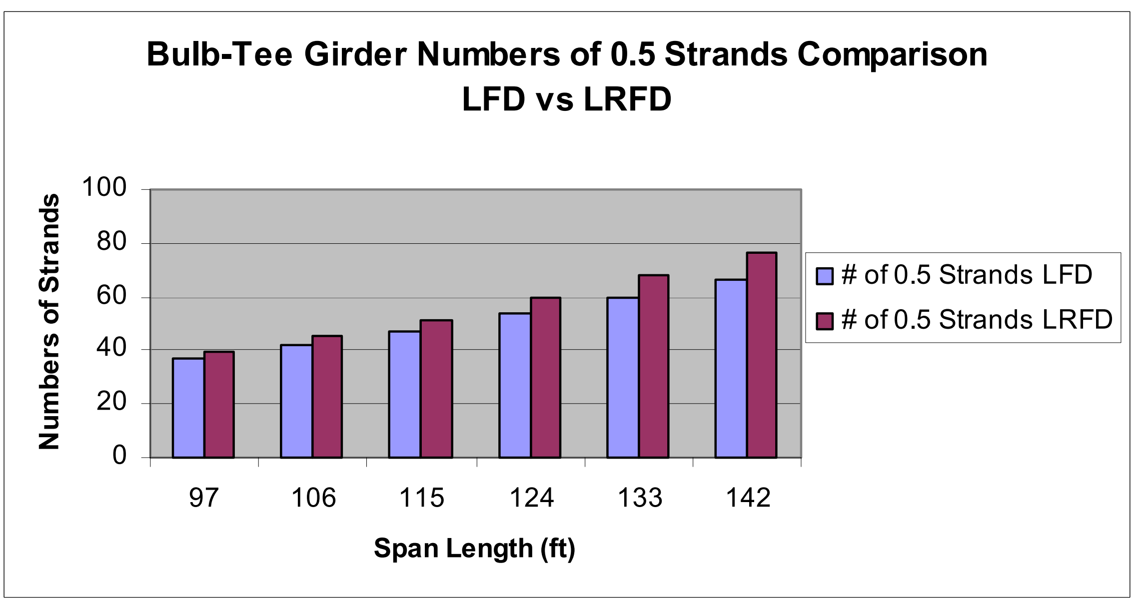
"D" (mm)	"h" (mm)	Area (mm^2)	I (mm^4)	y_b (mm)	y_t (mm)	S_b (mm^3)	S_t (mm^3)	r (mm)	Mass (N/m)	Wt. (Kg/m)
1400	850	596 500	155 400 E6	721	679	215.5 E6	228.8 E6	510	14 060	1433
1550	1000	626 500	201 200 E6	795	755	253.0 E6	266.5 E6	567	14 770	1505
1700	1150	656 700	254 200 E6	870	830	292.1 E6	306.2 E6	622	15 480	1577
1850	1300	686 100	314 500 E6	945	905	333.0 E6	347.5 E6	677	16 180	1650
2000	1450	716 300	382 600 E6	1019	981	375.4 E6	390.0 E6	731	16 890	1722
2150	1600	746 500	458 800 E6	1094	1056	419.4 E6	434.5 E6	784	17 600	1794

(1) P-Jacking Force Comparison



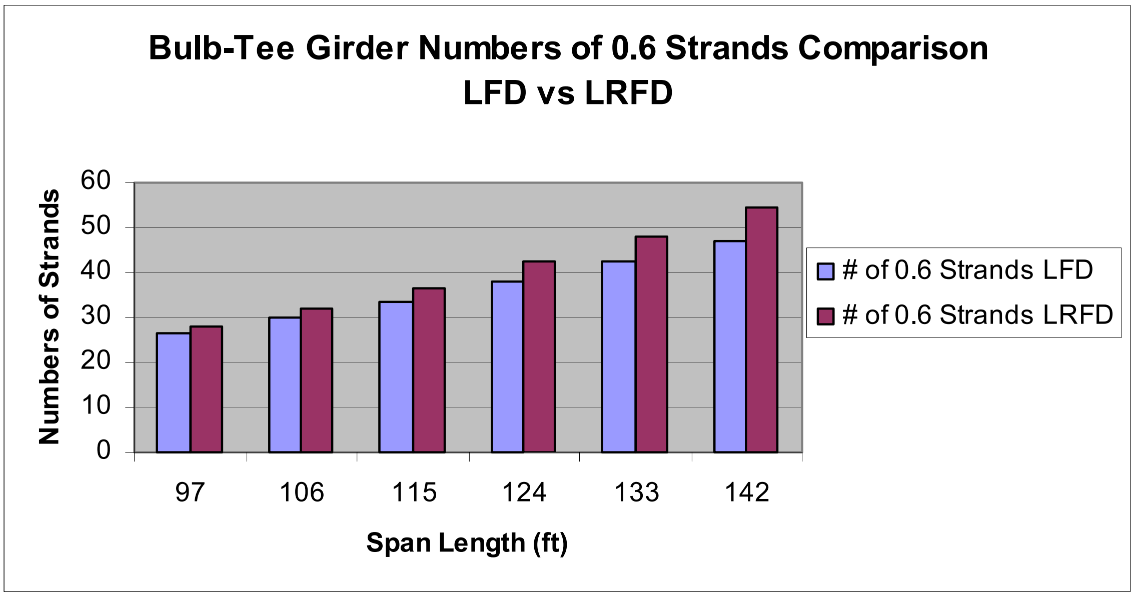
Girder Name	Bulb-Tee 55	Bulb-Tee 61	Bulb-Tee 67	Bulb-Tee 73	Bulb-Tee 79	Bulb-Tee 85
Span L (ft)	97	106	115	124	133	142
LFD	1150	1310	1470	1670	1850	2050
LRFD	1220	1400	1600	1860	2100	2380
Increase	6%	7%	9%	11%	14%	16%

(2) Numbers of 0.5" dia. Strands Comparison



Girder Name	Bulb-Tee 55	Bulb-Tee 61	Bulb-Tee 67	Bulb-Tee 73	Bulb-Tee 79	Bulb-Tee 85
Span L (ft)	**97**	**106**	**115**	**124**	**133**	**142**
LFD	**37**	**42**	**47**	**54**	**60**	***66***
LRFD	**39**	**45**	**52**	**60**	***68***	***77***
Increase	**2**	**3**	**5**	**6**	**8**	**11**

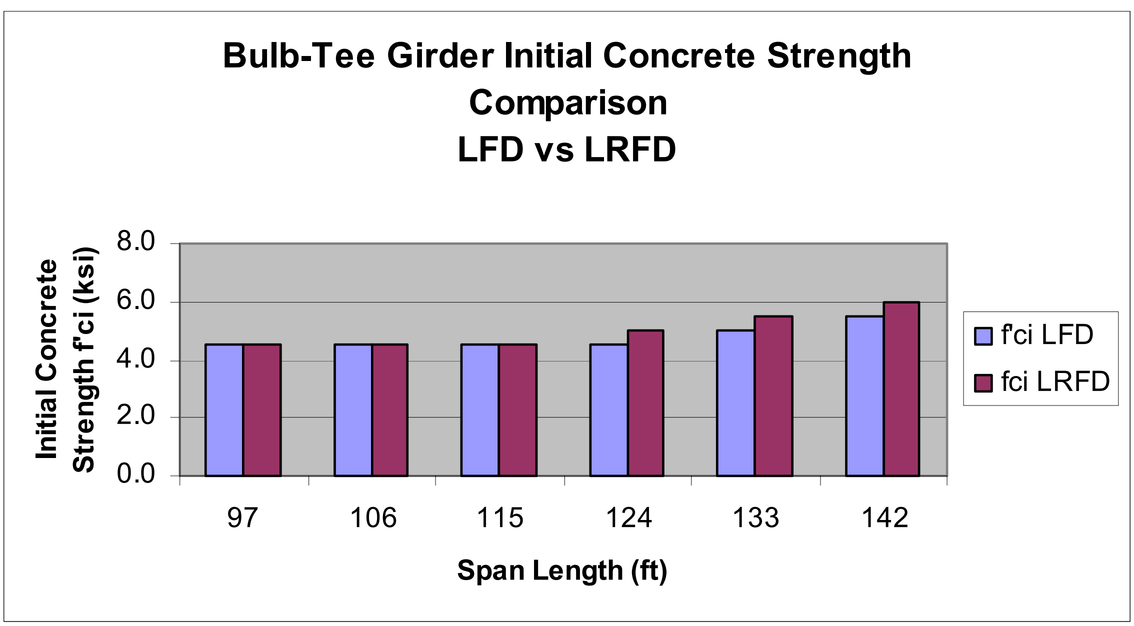
(3) Numbers of 0.6" dia. Strands Comparison



Girder Name	Bulb-Tee 55	Bulb-Tee 61	Bulb-Tee 67	Bulb-Tee 73	Bulb-Tee 79	Bulb-Tee 85
Span L (ft)	**97**	**106**	**115**	**124**	**133**	**142**
LFD	**26**	**30**	**34**	**38**	**42**	**47**
LRFD	**28**	**32**	**37**	**43**	**48**	**54**
Increase	**2**	**2**	**3**	**5**	**6**	**7**

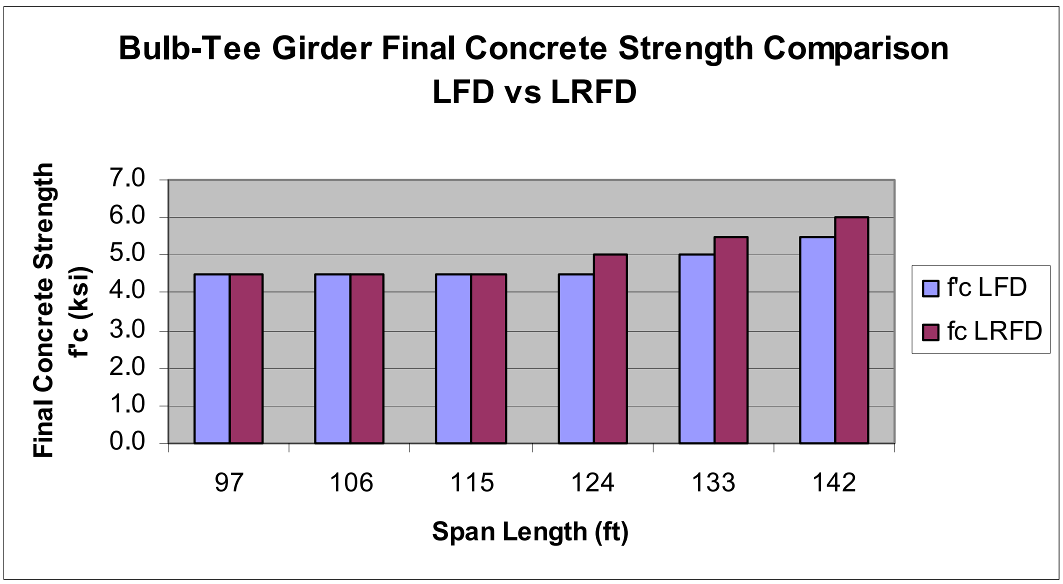
(4) California "Bulb-Tee" girder maximum numbers of strands is 60. Therefore, using 0.5" dia. strands of "Bulb-Tee" girder for span length over 125' is not adequate. Using 0.6" dia. strands are required.

(5) Initial concrete strength comparison



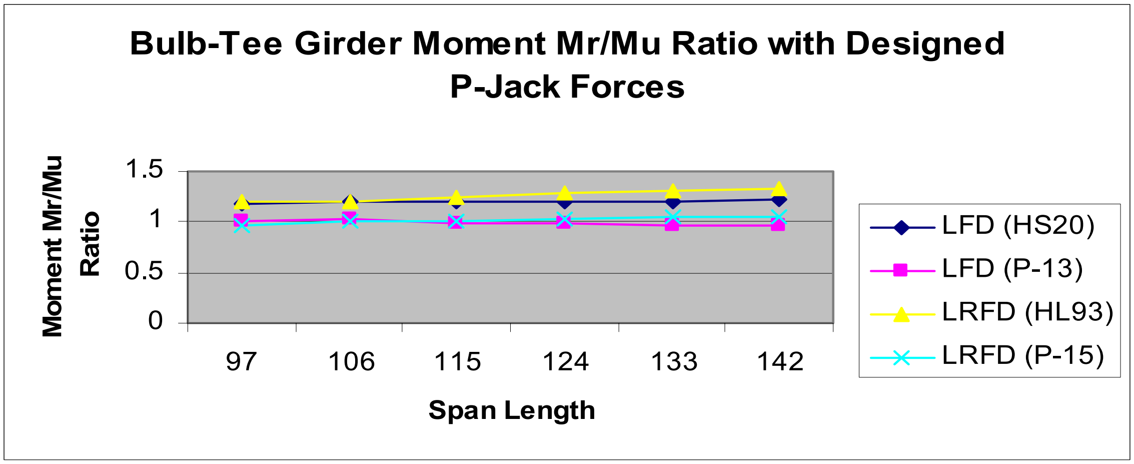
Girder Name	Span Length (ft)	LFD	LRFD	Increase
Bulb-Tee 55	97	4.5	4.5	0%
Bulb-Tee 61	106	4.5	4.5	0%
Bulb-Tee 67	115	4.5	4.5	0%
Bulb-Tee 73	124	4.5	5.0	11%
Bulb-Tee 79	133	5.0	5.5	10%
Bulb-Tee 85	142	5.5	6.0	9%

(6) Final concrete strength comparison



Girder Name	Span Length (ft)	LFD	LRFD	Increase
Bulb-Tee 55	97	4.5	4.5	0%
Bulb-Tee 61	106	4.5	4.5	0%
Bulb-Tee 67	115	4.5	4.5	0%
Bulb-Tee 73	124	4.5	5.0	11%
Bulb-Tee 79	133	5.0	5.5	10%
Bulb-Tee 85	142	5.5	6.0	9%

(7) Moment Mr/Mu ratio with designed P-Jack forces



Girder Name	Span L (ft)	LFD (HS20)	LFD (P-13)	LRFD (HL93)	LRFD (P-15)
Bulb-Tee 55	97	1.18	1.01	1.20	0.96
Bulb-Tee 61	106	1.19	1.02	1.21	1.01
Bulb-Tee 67	115	1.19	0.99	1.24	1.01
Bulb-Tee 73	124	1.20	0.98	1.28	1.03
Bulb-Tee 79	133	1.21	0.96	1.30	1.04
Bulb-Tee 85	142	1.22	0.96	1.33	1.05

SUMMARY AND CONCLUSIONS

1. Bridges being designed by LRFD specifications with California Amendments, instead of LFD specifications, will not necessarily be stronger or weaker, just more consistent in their level of safety.
2. Although most California Amendments to LRFD are based 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.
3. LRFD precast design is the extension of LFD design. Although the design process does not change, the design methods that include loads, prestress losses, and shear strength have changed.
4. For California “I” girder LRFD vs. LFD case study, the results show use of LRFD specifications leads 11% to 28% larger P-Jack forces. Both initial and final girder concrete strength are higher for LRFD design. Girder strength limit Mr/Mu ratios are adequate with designed P-Jack forces.
5. For California “Bulb-Tee” girder LRFD vs. LFD case study, the results show use of LRFD specifications leads 6% to 16% higher P-Jack forces. Both initial and final girder concrete strength increase slightly for LRFD design. Girder ultimate moment Mr/Mu ratios are adequate with designed P-Jack forces.
6. Case study could not conclude certain pattern comparison of shear design between LRFD and LFD. Therefore, shear design needs to be evaluated case by case mainly due to loading and load factor changes and nominal shear strength method change.

7. For constructability purpose, it is recommended to use 0.6” dia. prestress strands instead of 0.5” dia. prestress strands for both California “I” girder and California “Bulb-Tee” girder. Using 0.5” dia. strands of “I” girder for span length over 100’ is not adequate since maximum numbers of strands of “I” girder are 40. Same as using 0.5” dia. strands of “Bulb-Tee” girder for span length over 125’ is not adequate as maximum numbers of strands of “Bulb-Tee” girder are 60.
8. Case study shows that California “Bulb-Tee” girder sections are more efficient sections and use of its section is recommended for span over 95 feet.

REFERENCES

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2. Bridge Design Specifications, LFD Version, April 2000, based on Standard Specifications for Highway Bridges, Sixteenth Edition, 1996 (with interims through 1998)
3. California Amendments to AASHTO LRFD, v0.05, March 29, 2006
4. J. Ma, “LRFD Precast Design and Software in California”, Caltrans-PCMAC Bridge Seminar, Nov., 2005