CONCRETE TENSION LIMIT FOR PRESTRESSED CONCRETE BRIDGES IN OREGON

Craig Shike, PE, Oregon Department of Transportation

ABSTRACT

Single-trip permit trucks weighing 258,000 Lbs are allowed on Oregon highways. Because of these heavy loads ODOT was concerned whether using 0.19*sqrt(f'c) concrete tension, as allowed by the LRFD Bridge Design Specifications, provides adequate performance for Oregon bridges.

ODOT investigated single-span slab, box beam and bulb-T girder bridges under both service and strength limit states. The analysis revealed the following:

- HL-93 Service Limit State always controlled the design.
- There is a moderate risk of girder cracking under the largest permit truck when members are designed using 0.19*sqrt(f'c) and virtually no risk when using 0.095*sqrt(f'c).

A random group of reinforced concrete and precast prestressed concrete bridges in Oregon were compared. The rating factor for precast prestressed concrete bridges average 70% higher.

Many state DOTs have already made adjustments to the LRFD allowable concrete tension. The information in this paper provides a framework for determining an appropriate tension limit considering a state's actual permit truck loading.

Keywords: LRFD, ODOT, Superloads

INTRODUCTION

In September 2004, the Oregon Department of Transportation (ODOT), Bridge Engineering Section, added a second notional live load to be used for the design of bridges in Oregon. The load, hereafter referred to as the "ODOT notional load" was similar to the HL-93 notional load used in the AASHTO LRFD Bridge Design Specifications. In the ODOT notional load, an ODOT permit truck was combined with the AASHTO lane load. The load was applied to Service I, Service III and Strength I limit states in the same way as HL-93. The ODOT notional load had a greater shear effect for spans longer than 45 ft and a greater moment effect for spans longer than 65 ft. Since it was applied to Service I and III limit states, the ODOT notional load was particularly punitive to prestressed concrete structures.

In January, 2005, ODOT assembled a peer review panel to evaluate whether or not to continue use of the ODOT notional load. The 15-member panel consisted of ODOT bridge engineers (both current and retired), ODOT load rating engineers, consultant engineers, academia and industry. The panel strongly recommended elimination of the ODOT notional load.

The panel also made a recommendation concerning concrete tensile limits. Material reviewed by the peer review panel is the background for this paper. In particular, this paper will investigate the stresses caused by permit truck loading as a basis to determine an appropriate design concrete tensile limit for prestressed superstructure members.

OREGON TRUCK LOADS

Truck loads allowed on Oregon highways are among the largest in the country. Trucks up to a legal load of 80,000 Lbs are allowed provided they do not exceed 34,000 Lbs on a tandem axle. A permit is required for trucks not meeting the legal load configuration. Configurations for annual continuous-trip permits are shown in Figure 1. Continuous-trip permits are offered for trucks up to 105,500 Lbs. These permits allow specific trucks to travel multiple trips along an approved route. These types of permits must be renewed annually and generally do not have restrictions on how many trips can be made. For heavier loads, single-trip permits allow a single pass along an approved route. These trucks can be as high as 258,000 Lbs with up to 60,000 Lbs on a triple axle. Oregon's three largest single-trip permit truck configurations are shown in Figure 2. These three single-trip permit trucks are also referred to as superloads.

Oregon bridges are designed using the HL-93 notional load. The real loads that must be resisted, however, are the permit trucks. Loads less than the permit trucks have substantially less load effect and therefore are not significant for design.

In Oregon, continuous-trip permit trucks are very common. It is quite common to see these types of trucks back to back. Therefore, it is desirable that these loads do not cause tension cracking at the bottom of a prestressed superstructure member.

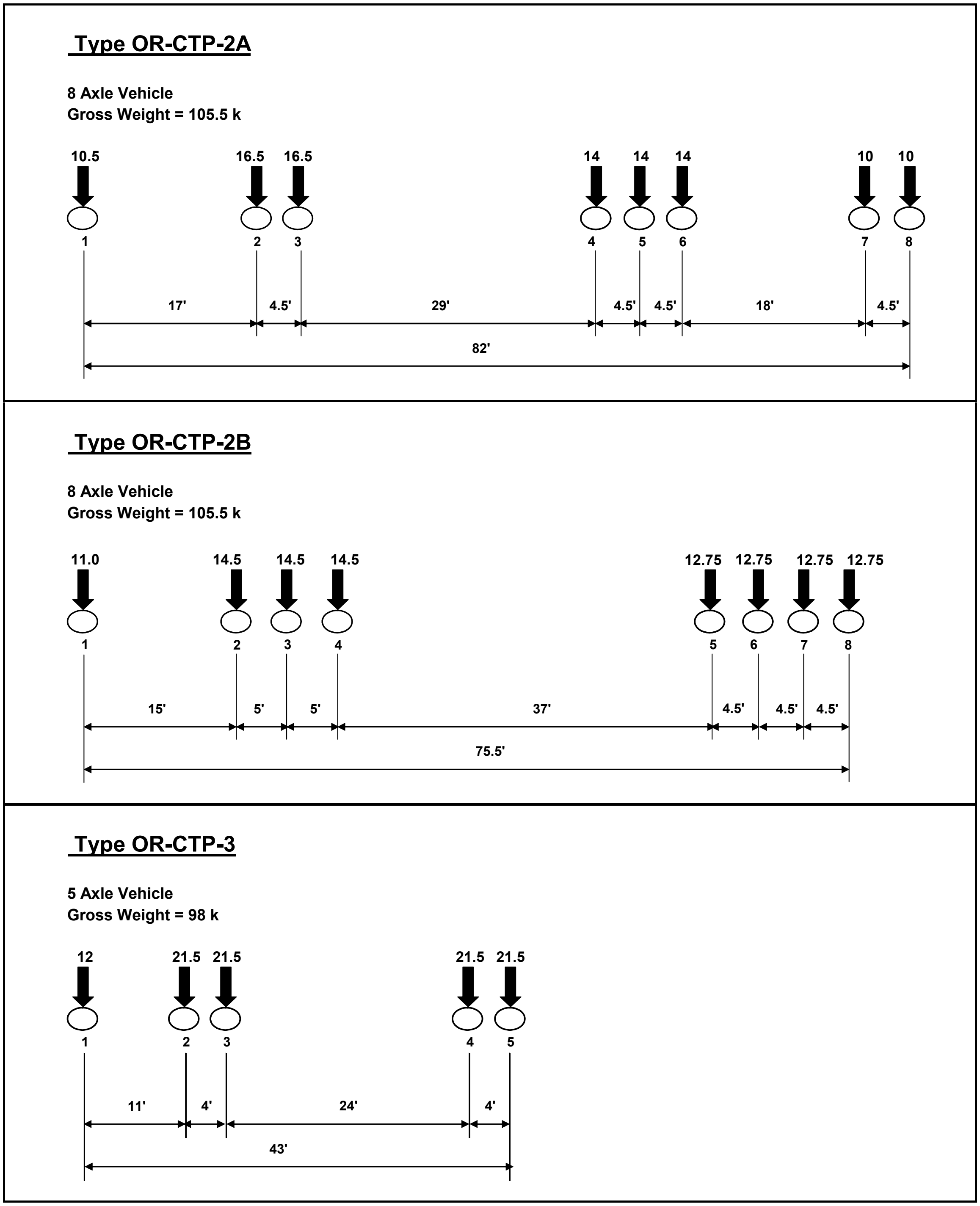

Fig. 1 Oregon Continuous-trip Permit Trucks (2A, 2B & 3). Concentrated axle loads are in kips.

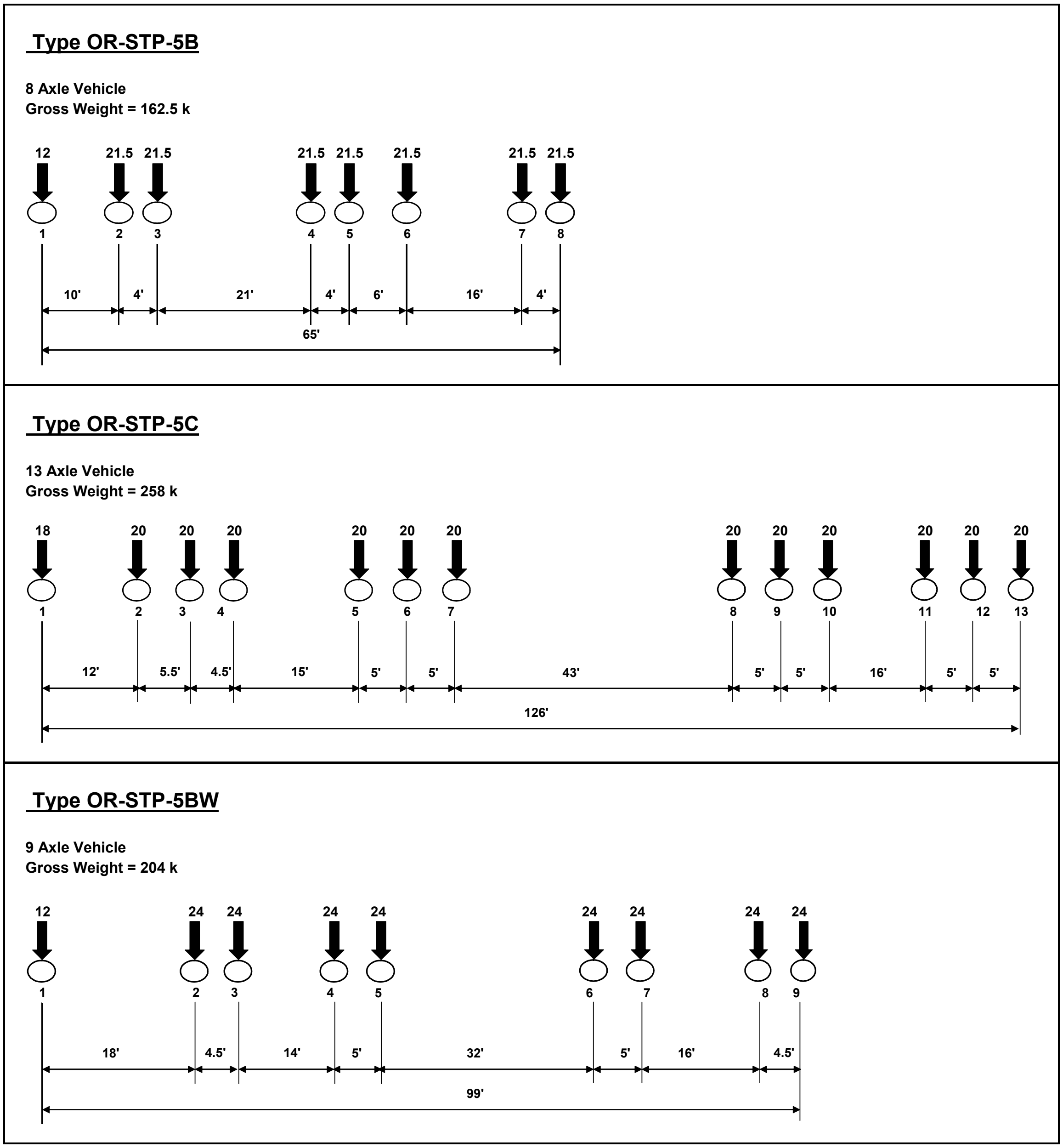

Fig. 2 Oregon Single-trip Permit Trucks (STP 5B, 5C & 5BW). Concentrated axle loads are in kips.

Figure 3 compares service moments for the Oregon CTP-3 permit truck to HL-93 service moments and service moments under the standard specifications. The approximate load at initiation of moment cracking (modulus of rupture) is shown as 0.88*HL-93. Since the LRFD specifications allow the live load tension effect to be reduced by 20% [1], the "design" bottom stress is 0.8 times the bottom stress from HL-93 service loading. If a beam element is designed up to 0.19*sqrt(f'c) concrete tension stress, approximately 8% additional live load would raise the concrete tensile stress up to the modulus of rupture (0.24*sqrt(f'c)). Therefore, 0.88*HL-93 represents the approximate modulus of rupture for a beam element that is designed up to the maximum allowable concrete tension.

Among Oregon's continuous-trip permit trucks, the CTP-3 truck has the greatest load effect. Figure 3 shows the CTP-3 permit truck has less moment effect than 0.88*HL-93. Therefore, if a superstructure member is designed for HL-93 loading, the tension stress caused by the CTP-3 truck will be less than the modulus of rupture and the member will not crack.

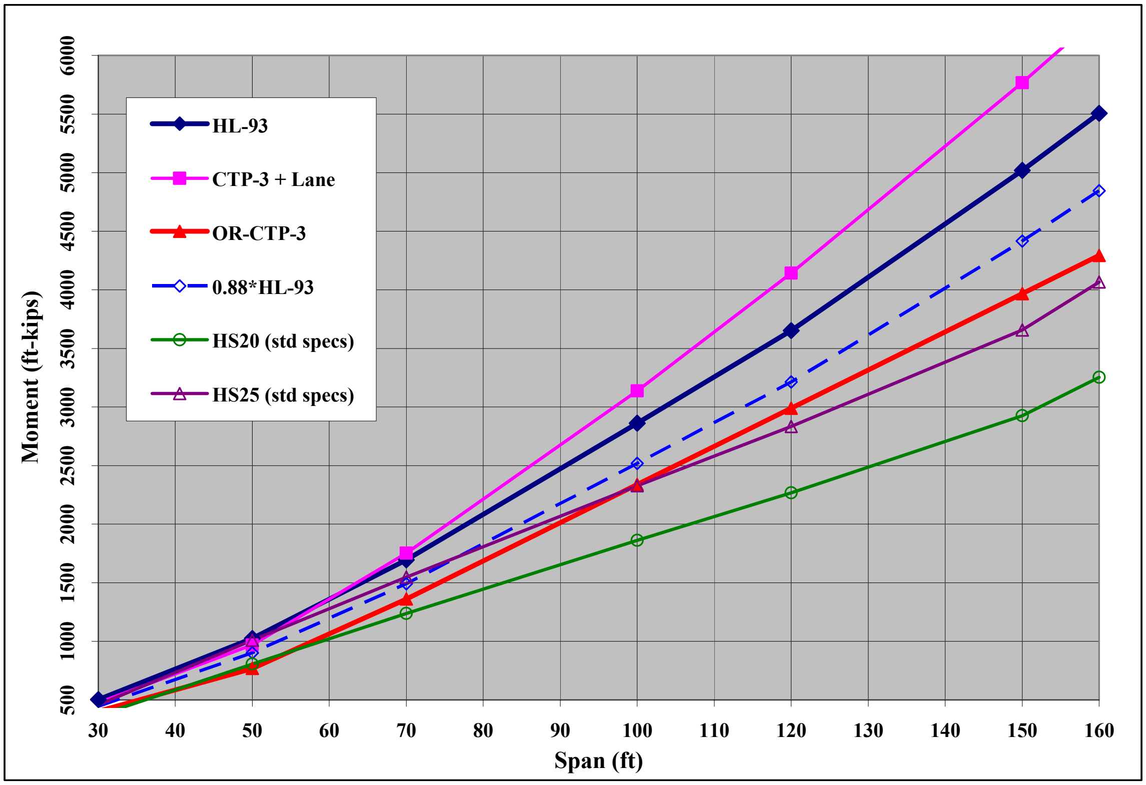

Fig. 3 Mid-span Moment (Live Load + Impact) for Single-span Prismatic Members under Service I Limit State.

Single-trip permit trucks (superloads) are less common, but are significant for design. A typical bridge may pass only a handful of superloads per month. High-volume bridges, however, may pass several superloads per day. In either case, superloads are less frequent than continuous permit trucks. Some concrete tension, and perhaps some cracking, may be acceptable under this type of loading.

In addition to HL-93 loading, Oregon bridges must also be designed to accommodate superloads under the LRFD Strength II limit state. For single-span bridges, Figure 4 compares the shear loading for HL-93 and ODOT superloads. HL-93 loading is calculated using the LRFD Strength I Limit State (load factor = 1.75) and the superload effect is calculated using the Strength II Limit State (load factor = 1.35). Superloads clearly control the shear design for spans greater than 100 ft. Therefore, tighter stirrup spacing may be needed to accommodate superloads.

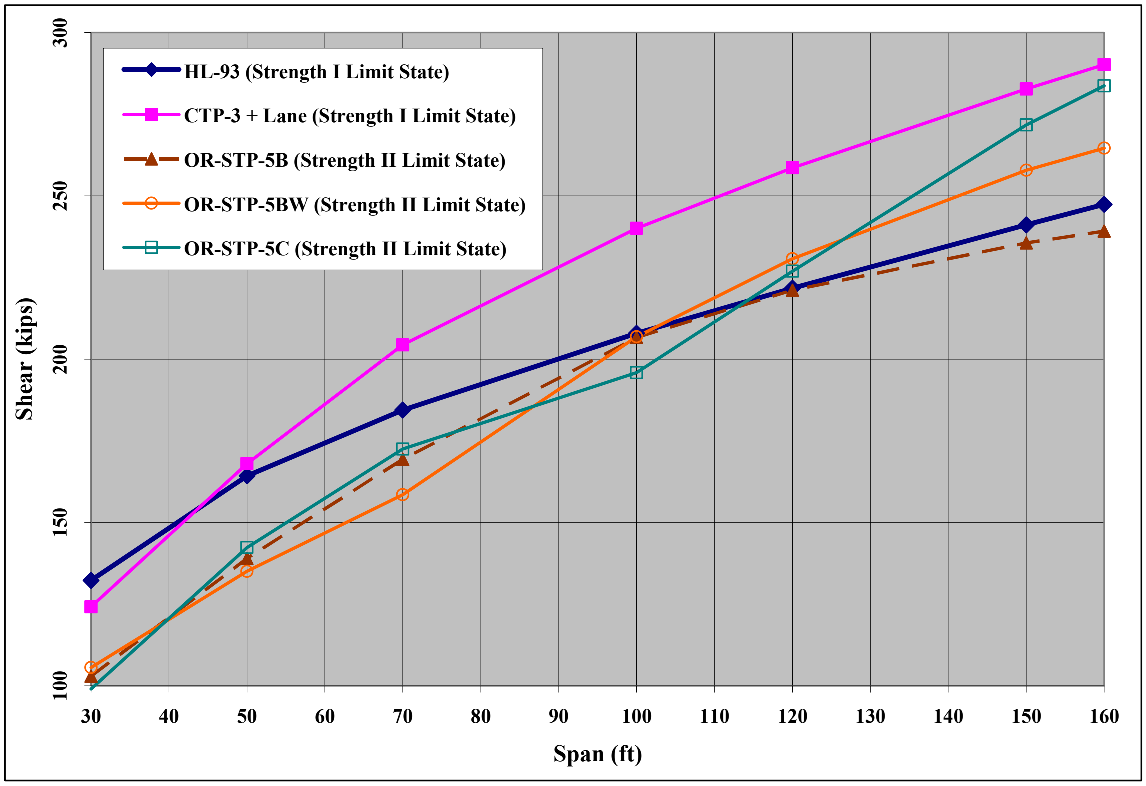

Fig. 4 Maximum Shear (Live Load + Impact) for Single-span Prismatic Members under Strength Limit States.

Figure 5 compares the moment loading from HL-93 with the superloads. Any increase in the moment has potential to affect the member size and/or number of girder lines. Figure 5 clearly shows the superloads do not control for the normal range of girder sizes. The superloads begin to affect the design when the span length exceeds 170 ft.

As stated previously, back to back continuous-trip permit trucks are common at many Oregon bridge sites. Back to back trucks were considered for both continuous-trip and single-trip permit trucks. Figure 6 shows the live load effect from two CTP-3 trucks spaced 50 ft between trailing and leading axles (approximately 40 ft bumper to bumper). Up to a span of 130 ft, the second truck has no affect. After 130 ft, the line representing two CTP trucks is parallel to the HL-93 line. Therefore, HL-93 loading adequately envelopes multiple CTP trucks.

Loading with back to back superloads is possible, but rare. And because these trucks are quite long, only a very long bridge would be able to physically accommodate two such trucks placed longitudinally. Therefore, the AASHTO recommendation to only investigate one permit truck on a bridge is rational.

ODOT's superloads appear to be substantially larger than any of the permit loads considered when the LRFD specifications were developed. However, with only minor modification, bridges designed using the HL-93 design criteria can safely and adequately resist these loads.

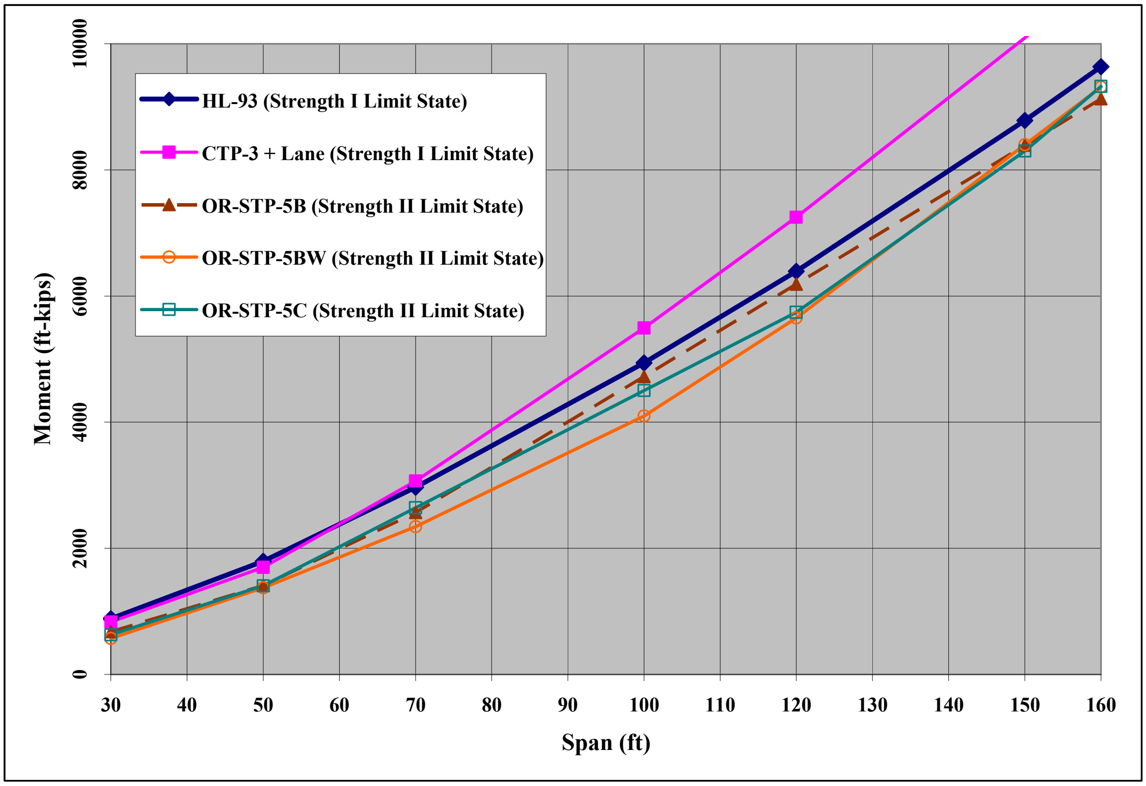

Fig. 5 Mid-span Moment (Live Load + Impact) for Single-span Prismatic Members under Strength Limit States.

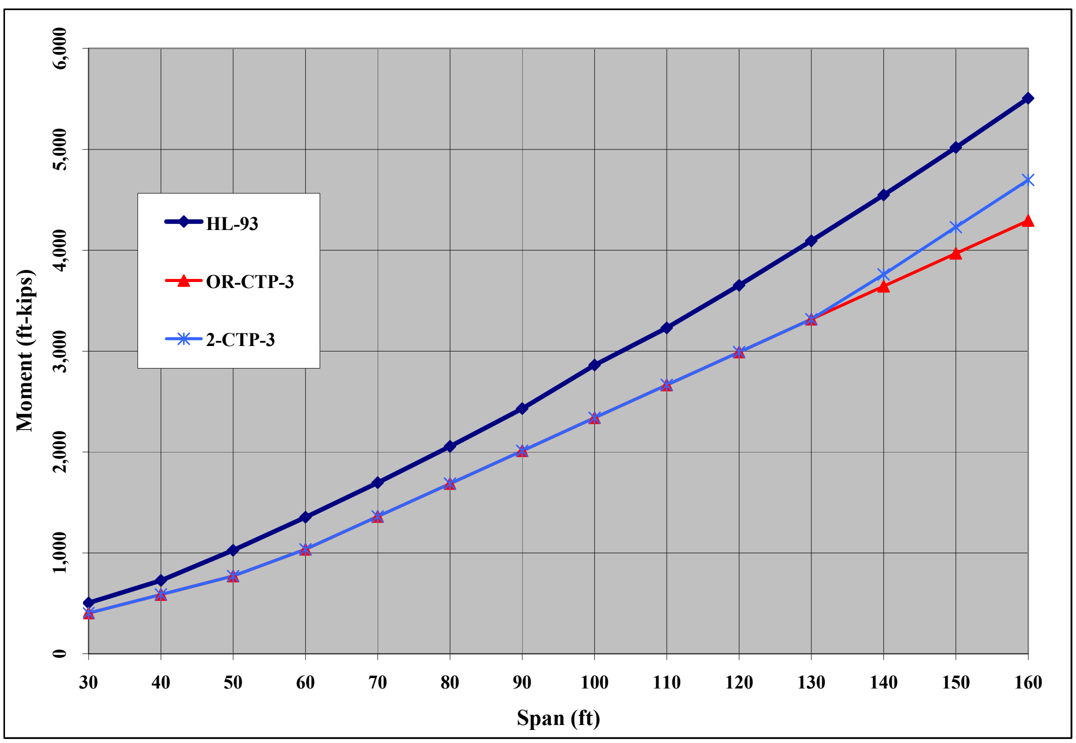

Fig. 6 Mid-span Moment (Live Load + Impact) for Single-span Prismatic Members under Service I Limit State showing one vs. two CTP-3 Permit Trucks.

COMPARISON OF HL-93 AND HS20-NO-TENSION DESIGNS

Before implementation of LRFD, ODOT designed bridges using HS25 loading with an allowable concrete tensile stress of 0.095*sqrt(f'c). Prestressed concrete bridges were also investigated for HS20 loading with no concrete tension allowed. The HS20 "no tension" criteria generally controlled.

As part of the ODOT live load study, several sample designs were prepared to evaluate HL-93 loading with 0.19*sqrt(f'c) tension and HS20-no-tension. The results for 5 sample designs are shown in Tables A1 to A5 in the appendix. The study clearly showed the HL-93 designs as less conservative than older designs meeting the HS20-no-tension criteria. And so the question for ODOT engineers was whether a less conservative design should be allowed considering a large number of bridges meeting the HS20-no-tension criteria were already in place. Since allowable permit loads have increased in recent years, there was reluctance to design new bridges for less capacity than bridges already on the system.

For the 5 sample designs, an attempt was made to determine the extent of cracking under ODOT's superloads. Girder bottom stresses were calculated including the prestress gain due to application of live load. These stresses were compared to a modulus of rupture using 1.2 times the design concrete strength (i.e., 0.24*sqrt(1.2*f'c)). The 20% additional strength is thought to represent a conservative estimate of actual concrete strength.

A summary of the bottom girder stresses is shown in Table 1. For HS20-no-tension designs, there is a minor risk of girder cracking for Bulb-T girders in the 100 to 120 ft range. Future wearing surface for Bulb-T bridges causes approximately 0.230 ksi tension in the bottom. And so the risk of cracking would be completely eliminated if the future wearing surface has not yet been applied.

Table 1 Bottom Stresses for Sample Designs (ksi – tension is negative). Bold indicates stress is exceeded.

	CTP-3	STP-5B	STP-5BW	STP-5C	Crack
70 ft Span with 26" Prestressed Slabs - HL-93	-0.502	-0.606	-0.491	-0.645	-0.662
70 ft Span with 26" Prestressed Slabs - HS20	-0.336	-0.451	-0.323	-0.494	-0.662
100 ft Span with 42" Prestressed Box Beams - HL-93	-0.426	**-0.808**	-0.656	**-0.755**	-0.662
100 ft Span with 42" Prestressed Box Beams - HS20	-0.099	-0.535	-0.361	-0.473	-0.662
100 ft Span with 48" Prestressed Bulb-T Girders - HL-93	-0.267	**-0.910**	-0.654	**-0.819**	-0.662
100 ft Span with 48" Prestressed Bulb-T Girders - HS20	-0.115	**-0.796**	-0.525	**-0.700**	-0.662
120 ft Span with 60" Prestressed Bulb-T Girders - HL-93	-0.182	**-0.826**	**-0.664**	**-0.691**	-0.662
120 ft Span with 60" Prestressed Bulb-T Girders - HS20	-0.030	**-0.713**	-0.542	-0.570	-0.662
160 ft Span with 84" Prestressed Bulb-T Girders - HL-93	-0.210	**-0.762**	**-0.793**	**-0.795**	-0.712
160 ft Span with 84" Prestressed Bulb-T Girders - HS20	-0.075	-0.627	-0.658	-0.660	-0.712

For HL-93 designs, there is potential for cracking in both box beams and bulb-T girders. However, if the future wearing surface has not yet been applied, the risk for cracking is nearly eliminated.
The stresses in Table 1 are based on an impact of 1.33 as required by the LRFD Bridge Specifications. The actual impact for superloads is likely to be significantly less. An NHI course on LRFD design documented the following trends [2]:

- Impact goes down as vehicle weight increases
- Multiple vehicles produce less impact than a single vehicle
- More axles result in less impact

The same document also graphically shows impact as less than 20% for 5-axle truck weights over 150,000 Lbs [3]. Therefore, there is reason to believe the true impact for Oregon's superloads is closer to 20%. Table 2 is a recalculation of bottom stresses for the HL-93 designs using an impact of 20%. After this adjustment, the potential for girder cracking is substantially reduced.

Table 2 Bottom stresses with Impact = 1.2 (ksi – tension is negative). Bold indicates cracking stress is exceeded

	STP-5B	STP-5BW	STP-5C	Crack
70 ft Span with 26" Prestressed Slabs - HL-93	-0.477	-0.372	-0.512	-0.662
100 ft Span with 42" Prestressed Box Beams - HL-93	**-0.696**	-0.559	-0.647	-0.662
100 ft Span with 48" Prestressed Bulb-T Girders - HL-93	**-0.718**	-0.487	-0.636	-0.662
120 ft Span with 60" Prestressed Bulb-T Girders - HL-93	-0.645	-0.499	-0.523	-0.662
160 ft Span with 84" Prestressed Bulb-T Girders - HL-93	-0.614	-0.642	-0.644	-0.712

To date, there is no evidence of any mid-span moment cracking in any of Oregon's prestressed concrete bridges. This further supports the belief that impact is substantially less for the superloads.

Table 3 shows the number of prestress strands required for each sample design. Bulb-T designs required only two additional strands to meet the HS20-no-tension criteria. For slab and box beam designs, the additional prestress strand required was significant. Although additional strand increases the ultimate strength of a member, it also has the following negative consequences:

- Mid-span camber is increased resulting in additional build-up.
- Required release strength is increased.
- Long-term shortening is increased resulting in higher future maintenance.
- Overall cost is increased.

The benefits of additional load capacity must be weighed against these consequences when determining what level of concrete tensile stress should be allowed.

Table 3 Prestress Strand Requirements for Sample Designs.

	# STRANDS
70 ft Span with 26" Prestressed Slabs - HL-93	36
70 ft Span with 26" Prestressed Slabs - HS20	42
100 ft Span with 42" Prestressed Box Beams - HL-93	30
100 ft Span with 42" Prestressed Box Beams - HS20	38
100 ft Span with 48" Prestressed Bulb-T Girders - HL-93	36
100 ft Span with 48" Prestressed Bulb-T Girders - HS20	38
120 ft Span with 60" Prestressed Bulb-T Girders - HL-93	42
120 ft Span with 60" Prestressed Bulb-T Girders - HS20	44
160 ft Span with 84" Prestressed Bulb-T Girders - HL-93	52
160 ft Span with 84" Prestressed Bulb-T Girders - HS20	54

The ODOT live load peer review panel recommended designs be allowed up to 0.19*sqrt(f'c) concrete tensile stress. This recommendation was based on the following facts:

- Loading from ODOT continuous-trip permit trucks do not lead to girder cracking.
- The potential for cracking under ODOT single-trip permit trucks (superloads) is low.
- Load ratings for existing prestressed concrete bridges far exceed other bridge types.

LOAD RATING OF PRESTESSED CONCRETE BRIDGES

The purpose of a bridge is to allow various types of vehicles to cross some obstacle. The design of a bridge should be controlled by the vehicle causing the largest load effect. Since there are a variety of vehicles that use highways, it is not always obvious which one causes the greatest load effect. For this reason, the AASHTO LRFD specifications use a notional load. This load does not represent a particular truck. Instead, the notional load envelopes a variety of potential vehicles.

The HS20 truck used in the standard specifications appears to be a common truck. However, the configuration of the truck does not really match any common trucks on our highways. The HS20 load (truck or lane with shear and moment riders) was essentially a notional load.

Load rating is a measure of the capacity of a bridge to safely carry a particular load. The load can be either specific trucks, such as common permit trucks, or other standard loads such as the HL-93 design load.

The LRFD specifications allow permit trucks to be considered under the Strength II Limit State. Under this limit state, a load factor of 1.35 is applied. Using this limit state, the moment effect of a permit truck can be compared against the moment effect of the HL-93 design load using the Strength I Limit State (load factor = 1.75). A rating factor for a permit truck can then be calculated by taking the moment caused by HL-93 loading and dividing by the maximum permit truck moment. For example, the minimum permit truck rating factor for a 30 ft span designed using the LRFD specifications would be as follows:

882.4 ft-kips = HL-93 Strength I Limit State mid-span moment (live load with 1.33 impact)
675.6 ft-kips = Maximum Strength II Limit State mid-span moment (STP-5B truck with 1.33 impact)

Minimum rating factor = 882.4 / 675.6 = 1.31

Table 4 provides a summary of the minimum rating factor for various span lengths. This table shows a clear trend toward higher rating factors with decreasing span lengths. This trend is desirable since any future increases in axle weights will impact short spans to a greater degree than long spans. Since the HL-93 load uses a 32 kip single axle, the load ratings for short spans will be satisfactory until single axle weights of permit trucks approach 32 kips. Therefore, short spans have additional protection against future load increases.

Table 4 Theoretical Minimum Rating Factor (all material types).

Span	HL-93 Loading per LRFD Specs	HS25 Loading per Standard Specs
30 ft	1.31	1.47
70 ft	1.12	1.27
100 ft	1.05	1.07
120 ft	1.03	0.99
160 ft	1.03	0.95

The rating factors shown in Table 4 are theoretical minimums. Load ratings for actual bridges should be higher. Table 5 shows the range of rating factors for 6 conventionally-reinforced concrete bridges [4] and 15 prestressed concrete bridges [5]. All designs met HS20 loading or better. A wide range of span lengths were included. Fewer conventionally-reinforced concrete bridges were included simply because it was more difficult to find non-prestressed bridges designed to at least HS20.

Table 5 Rating Factors for Critical Permit Truck (bridges designed for HS20 or better).

Concrete w/o prestress (6 bridges)	0.98 to 1.65 (1.30 ave.)
Prestressed concrete (15 bridges)	1.66 to 3.11 (2.24 ave.)

The average rating factor for prestressed concrete is 72% higher than conventionally-reinforced concrete. Some of this increase can be accounted to the HS20-no-tension design criteria. The HS20-no-tension criteria is essentially equivalent to HS25. But this can account for only 25% of the difference. A small portion of the difference is due to slightly less conservative load factors and lower impact used in the load rating criteria. The remainder is due to the fact that prestressed concrete design is controlled by service stresses. Prestressed concrete designs meeting existing code requirements (either LRFD or standard specifications) will always have excess ultimate capacity.

Oregon's bridge inventory includes a variety of steel, conventionally-reinforced concrete and prestressed concrete bridges. Table 5 clearly shows that prestressed concrete bridges are not the weak link on Oregon's transportation system. Some of the rating factors are quite high (up to 3.11). These rating factors include a load factor applied to the permit truck to provide a safety margin. Any bridge with a rating factor of 1.0 can safely pass the critical permit truck with adequate reserve capacity to ensure safety and limit potential damage.

Rating factors above 1.0 provide reserve capacity beyond current load requirements. A moderate reserve capacity provides a buffer to protect the system against future axle weight increases. But reserve capacity can only be used if all bridges on the system have this capacity. In the case of Oregon bridges, non-prestressed bridges clearly do not have the same reserve capacity. Therefore, if a route is controlled by the capacity of non-prestressed bridges, the excess capacity provided by the prestressed concrete bridges provides little or no benefit to the route.

CONCLUSIONS

The ODOT live load peer review panel strongly recommended allowing up to 0.19*sqrt(f'c) concrete tensile stress. ODOT, however, settled on 0.095*sqrt(f'c). This decision was based on following issues:

- 0.095*sqrt(f'c) tension using the LRFD specifications is the closest match to an HS20-no-tension design using the standard specifications.

- The majority of ODOT's bridge inventory consists of prestressed concrete superstructure members. ODOT was reluctant to move to a lesser design for this type of bridge.

- The vast majority (>90%) of new bridges in Oregon use prestressed concrete superstructure members. Therefore, Oregon is likely to have many routes which only contain prestressed concrete bridges.

- Since no mid-span moment cracking has ever been detected on an Oregon prestressed concrete bridge, the long-term consequences of allowing a girder to crack under extreme loads is not clearly understood.

- Should any cracks develop from extreme loads, ODOT's continuous-trip permit trucks cause sufficient stress to partially re-open those cracks. The long-term consequences of re-opening these cracks under everyday loading is not clearly understood.

- There is concern that allowable axle weights may increase in the future due to Oregon's strong trucking lobby.

- The concept of reduced impact for superloads (single-trip permit trucks) has not been proven to the point where it can be used for load ratings.

In coastal environments ODOT still allows up to 0.095*sqrt(f'c). This value sufficiently reduces the risk of cracking. Oregon coastal routes also have a significantly lower potential for superloads.

States adjacent to Oregon have different policies regarding concrete tensile stress. Although all three neighboring states use the LRFD Bridge Specifications, each has a different tensile limit.

California:	0.19*sqrt(f'c)
Idaho:	0.095*sqrt(f'c)
Washington:	No-tension

Concrete tensile stress up to 0.19*sqrt(f'c) as allowed by the AASHTO LRFD Bridge Specifications should clearly be adequate for most states and local agencies. This paper demonstrates how agencies can evaluate cracking potential using their own suite of permit trucks.

Additional research is needed to confirm assumptions regarding acceptable concrete cracking under extreme loads. In Oregon, we have always assumed that mid-span cracking under superloads would not have any long-term negative consequences. Prestressing would be expected to close any cracks immediately after the load exits the bridge. Now that we realize we have never experienced any mid-span cracks, we are looking for field data to confirm whether or not our original assumptions are valid.

Additional research is also needed to determine realistic impact factors for superloads. Use of a constant 1.33 impact factor in the LRFD Bridge Specifications has led many to the conclusion that the impact factor should never be challenged. There appears to be evidence impact is influenced by a number of factors including truck weight and number of axles. If this can be both confirmed and quantified by further research, the basis for using 0.19*sqrt(f'c) will be greatly strengthened. For such research to be effective, it must be accepted and implemented into established load rating procedures. The bottom line is new bridges must be able to pass all potential permit vehicles. A reduced impact (only for superloads) can be used in design only if it is also used in load rating.

The LRFD Bridge Specifications using the HL-93 notional load results in efficient and safe bridges that can accommodate most permit truck loads. Some agencies, such as those in Pacific Northwest, are making adjustments to the specification in order to ensure superloads can also be accommodated. If the additional research above is performed and accepted, fewer agencies will be inclined to make such adjustments. The result for the precast concrete industry will be more economical prestressed concrete members that are easier to fabricate.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 3rd Edition, Section 3.4.1, page 3-9.

2. FHWA Publication No. HI-95-016, NHI Course No. 13061, "Load and Resistance Factor Design for Highway Bridges", Participant Notebook, Volume 1 (Version 2.03), January 1995, page 3-54.

3. FHWA Publication No. HI-95-016, NHI Course No. 13061, "Load and Resistance Factor Design for Highway Bridges", Participant Notebook, Volume 1 (Version 2.03), January 1995, page 3-56.

4. Conventionally-reinforced concrete bridge load ratings are from the following bridges:
   - BR 00351A – Mud Slough – 36' - 48' – 36' R/C slab: Rating Factor = 1.23 (HS20 design)
   - BR 16161 – NB O'xing Commercial – 47'– 63'– 54' RCDG: RF = 1.44 (HS20)
   - BR 16858 – Dry Creek Bridge – 17' – 60' – 17' RCDG: RF = 1.41 (HS20)
   - BR 16859 – Copeland Creek – 20' – 80' – 20' RCDG: RF = 0.98 (HS20)
   - BR 17015 – O'xing Marietta St – 22' R/C slab: RF = 1.65 (HS25)
   - BR 18348 – Santiam O'flow No. 7 – 5 – 39' R/C slab: RF = 1.08 (HL-93)

5. Prestressed concrete bridge load ratings are from the following bridges:
   - BR 00241C – Dick Creek – 30' P/S slabs: Rating Factor = 2.25 (HS25 design)
   - BR 00365A – Miller Creek – 38' P/S slabs: RF = 2.54 (HS25)
   - BR 00416A – Ash Swale – 6- 50' P/S girders: RF = 2.61 (HS25)
   - BR 00578A – Lebanon Ditch Bridge – 55' P/S girders: RF = 3.11 (HS25)
   - BR 01206A – Mary's River – 70' – 90' – 70' P/S girders: RF = 1.66 (HS20)
   - BR 03113A – Nehalem River – 91' – 115' – 103' P/S girders: RF = 1.91 (HS20)
   - BR 04970B – Tualatin River – 105' – 106' – 105' P/S girders: RF = 1.93 (HS20)
   - BR 08431A – Oro Dell O'xing – 3 – 131' P/S girders: RF = 2.03 (HS20)
   - BR 17158 – Middle Fk Cold Springs – 86' P/S box beams: RF = 1.94 (HS25)
   - BR 17226 – U'xing Center Street – 2- 105' P/S box beams: RF = 2.58 (HS25)
   - BR 17346 – Clear Creek – 68' P/S slabs: RF = 2.25 (HS25)
   - BR 17398 – Mill Creek – 3 - 36' P/S slabs: RF = 2.29 (HS25)
   - BR 17399 – West Fork Mill Creek – 3 - 65' P/S slabs: RF = 2.18 (HS25)
   - BR 17460 – Neil Creek Bridge – 44' P/S slabs: RF = 2.39 (HS25)
   - BR 17939 – Amazon Creek – 86' P/S box beams: RF = 1.91 (HS25)

APPENDIX

Table A1 Tension Stress for HL-93 Design vs. HS20-no-tension Design – 70 ft span with 26” precast prestressed concrete slabs.

0.24*sqrt(1.2*f'c) = 0.662 ksi

HL-93 Design Impact = 1.33 DF = 0.301 0.24*sqrt(f'c) = 0.605 ksi

0.19*sqrt(f'c) = 0.484 ksi # strand = 36 f'c = 6.5 ksi

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		408	510	527.53	572.37	522.43	589.29
Stress (ksi)	0.717	0.997	1.246	1.288	1.398	1.276	1.439
Net design bottom stress (ksi)		-0.280	-0.529	-0.571	-0.681	-0.559	-0.722
P/S Gain due to LL (ksi)		3.484	4.355	4.502	4.885	4.458	5.029
P/S comp stress due to LL (ksi)		0.054	0.067	0.069	0.075	0.069	0.077
Net btm stress after P/S gain (ksi)		-0.227	-0.463	-0.502	-0.606	-0.491	-0.645

HS20-no-tension Design Impact = 1.222 DF = 0.337 Impact = 1.33 for permit trucks

0.095*sqrt(f'c) = 0.242 ksi # strand = 42

Load	Dead + P/S	HS20	HS25	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		415	519	590.62	640.83	584.91	659.77
Stress (ksi)	1.016	1.014	1.267	1.443	1.565	1.429	1.611
Net design bottom stress (ksi)		0.002	-0.251	-0.427	-0.549	-0.413	-0.595
P/S Gain due to LL (ksi)		3.537	4.421	5.034	5.462	4.985	5.623
P/S comp stress due to LL (ksi)		0.064	0.080	0.091	0.098	0.090	0.101
Net btm stress after P/S gain (ksi)		0.066	-0.171	-0.336	-0.451	-0.323	-0.494

HL-93 Design Impact = 1.33 for design and 1.20 for permit trucks DF = 0.301

0.19*sqrt(f'c) = 0.484 ksi # strand = 36

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		408	510	475.97	516.43	471.37	531.69
Stress (ksi)	0.717	0.997	1.246	1.163	1.261	1.151	1.299
Net design bottom stress (ksi)		-0.280	-0.529	-0.446	-0.544	-0.434	-0.582
P/S Gain due to LL (ksi)		3.484	4.355	4.062	4.407	4.023	4.537
P/S comp stress due to LL (ksi)		0.054	0.067	0.062	0.068	0.062	0.070
Net btm stress after P/S gain (ksi)		-0.227	-0.463	-0.383	-0.477	-0.372	-0.512

Properties	**HL-93 design**	**HS20 design**
Girder composite area	850 in^2	850 in^2
Girder composite bottom section modulus	4913 in^3	4913 in^3
Center of gravity of girder	12.93 in	12.93 in
Area of prestressing strand	5.51 in^2	6.43 in^2
Center of gravity of prestressing strand	5.00 in	4.93 in

Note: Distribution factors used assumes potential for permit trucks in adjacent lanes.

Table A2 Tension Stress for HL-93 Design vs. HS20 Design – 100 ft span with 42" precast prestressed box beams.

0.24*sqr(1.2*f'c) = 0.662 ksi
0.24*sqr(f'c) = 0.605 ksi
f'c = 6.5 ksi

HL-93 Design Impact = 1.33 DF = 0.285

0.19*sqr(f'c) = 0.484 ksi # strand = 30

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		644	805	666.84	997.60	865.98	950.95
Stress (ksi)	0.344	0.792	0.991	0.821	1.228	1.066	1.171
Net design bottom stress (ksi)		-0.448	-0.647	-0.477	-0.884	-0.722	-0.827
P/S Gain due to LL (ksi)		3.4808	4.351	3.605	5.394	4.682	5.142
P/S comp stress due to LL (ksi)		0.049	0.061	0.050	0.076	0.066	0.072
Net btm stress after P/S gain (ksi)		-0.400	-0.586	-0.426	**-0.808**	-0.656	**-0.755**

HS20-no-tension Design Impact = 1.222 DF = 0.334 Impact = 1.33 for permit trucks

0.095*sqr(f'c) = 0.242 ksi # strand = 38

Load	Dead + P/S	HS20	HS25	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		620	775	779.77	1166.55	1012.64	1111.99
Stress (ksi)	0.780	0.763	0.954	0.960	1.436	1.246	1.369
Net design bottom stress (ksi)		0.017	-0.174	-0.180	-0.656	-0.466	-0.589
P/S Gain due to LL (ksi)		3.666	4.583	4.611	6.898	5.988	6.575
P/S comp stress due to LL (ksi)		0.064	0.080	0.081	0.121	0.105	0.115
Net btm stress after P/S gain (ksi)		0.081	-0.093	-0.099	-0.535	-0.361	-0.473

HL-93 Design Impact = 1.33 for design and 1.2 for permit trucks DF = 0.285

0.19*sqr(f'c) = 0.484 ksi # strand = 30

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		644	805	601.66	900.09	781.33	858.00
Stress (ksi)	0.344	0.792	0.991	0.741	1.108	0.962	1.056
Net design bottom stress (ksi)		-0.448	-0.647	-0.397	-0.764	-0.618	-0.712
P/S Gain due to LL (ksi)		3.4808	4.351	3.253	4.867	4.225	4.639
P/S comp stress due to LL (ksi)		0.049	0.061	0.046	0.068	0.059	0.065
Net btm stress after P/S gain (ksi)		-0.400	-0.586	-0.351	**-0.696**	-0.559	-0.647

Properties	**HL-93 design**	**HS20 design**
Girder composite area	843 in^2	843 in^2
Girder composite bottom section modulus	9749 in^3	9749 in^3
Center of gravity of girder	20.80 in	20.80 in
Area of prestressing strand	4.59 in^2	5.81 in^2
Center of gravity of prestressing strand	2.62 in	2.92 in

Note: Distribution factors used assumes potential for permit trucks in adjacent lanes.

Table A3 Tension Stress for HL-93 Design vs. HS20 Design – 100 ft span with 48" precast prestressed Bulb-T girders @ 6.25 ft spacing.

0.24*sqrt(1.2*f'c) = 0.662 ksi
0.24*sqrt(f'c) = 0.605 ksi
f'c = 6.5 ksi

HL-93 Design Impact = 1.33 DF = 0.533

0.19*sqrt(f'c) = 0.484 ksi # strand = 36

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1203	1504	1246.46	1864.73	1618.70	1777.52
Stress (ksi)	1.030	1.416	1.769	1.466	2.194	1.904	2.091
Net design bottom stress (ksi)		-0.386	-0.739	-0.436	-1.164	-0.874	-1.061
P/S Gain due to LL (ksi)		6.982	8.729	7.234	10.823	9.395	10.317
P/S comp stress due to LL (ksi)		0.164	0.204	0.169	0.253	0.220	0.242
Net btm stress after P/S gain (ksi)		-0.222	-0.535	-0.267	**-0.910**	-0.654	**-0.819**

HS20-no-tension Design Impact = 1.222 DF = 0.568 Impact = 1.33 for permit trucks

0.095*sqrt(f'c) = 0.242 ksi # strand = 38

Load	Dead + P/S	HS20	HS25	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1058	1323	1328.53	1987.51	1725.28	1894.55
Stress (ksi)	1.259	1.245	1.556	1.563	2.338	2.030	2.229
Net design bottom stress (ksi)		0.014	-0.297	-0.304	-1.079	-0.771	-0.970
P/S Gain due to LL (ksi)		6.101	7.629	7.661	11.461	9.949	10.925
P/S comp stress due to LL (ksi)		0.151	0.188	0.189	0.283	0.246	0.270
Net btm stress after P/S gain (ksi)		0.165	-0.109	-0.115	**-0.796**	-0.525	**-0.700**

HL-93 Design w/ Impact = 1.33 for design and 1.2 for permit trucks DF = 0.533

0.19*sqrt(f'c) = 0.484 ksi # strand = 36

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1086	1357	1124.63	1682.46	1460.48	1603.78
Stress (ksi)	1.030	1.277	1.597	1.323	1.979	1.718	1.887
Net design bottom stress (ksi)		-0.247	-0.567	-0.293	-0.949	-0.688	-0.857
P/S Gain due to LL (ksi)		6.384	7.98	6.612	9.891	8.586	9.429
P/S comp stress due to LL (ksi)		0.150	0.187	0.155	0.232	0.201	0.221
Net btm stress after P/S gain (ksi)		-0.098	-0.380	-0.138	**-0.718**	-0.487	-0.636

Properties	**HL-93 design**	**HS20 design**
Girder composite area	1115 in^2	1115 in^2
Girder composite bottom section modulus	13,874 in^3	13,874 in^3
Center of gravity of girder	45.20 in	45.20 in
Area of prestressing strand	6.43 in^2	6.73 in^2
Center of gravity of prestressing strand	4.02 in	3.93 in

Note: Distribution factors used assumes potential for permit trucks in adjacent lanes.

Table A4 Tension Stress for HL-93 Design vs. HS20 Design – 120 ft span with 60" precast prestressed Bulb-T girders @ 6.25 ft spacing.

0.24*sqrt(1.2*f'c) = 0.662 ksi
0.24*sqrt(f'c) = 0.605 ksi
f'c = 6.5 ksi

HL-93 Design Impact = 1.33 DF = 0.5321

0.19*sqrt(f'c) = 0.484 ksi # strand = 42

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1555	1943	1590.89	2439.77	2227.11	2262.49
Stress (ksi)	1.024	1.345	1.681	1.376	2.110	1.926	1.957
Net design bottom stress (ksi)		-0.321	-0.657	-0.352	-1.086	-0.902	-0.933
P/S Gain due to LL (ksi)		6.691	8.361	6.846	10.49	9.583	9.735
P/S comp stress due to LL (ksi)		0.166	0.208	0.170	0.261	0.238	0.242
Net btm stress after P/S gain (ksi)		-0.155	-0.449	-0.182	**-0.826**	**-0.664**	**-0.691**

HS20-no-tension Design Impact = 1.204 DF = 0.5682 Impact = 1.33 for permit trucks

0.095*sqr(f'c) = 0.242 ksi # strand = 44

Load	Dead + P/S	HS20	HS25	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1288	1611	1698.83	2605.30	2378.21	2415.99
Stress (ksi)	1.250	1.114	1.393	1.469	2.253	2.057	2.090
Net design bottom stress (ksi)		0.136	-0.143	-0.219	-1.003	-0.807	-0.840
P/S Gain due to LL (ksi)		5.515	6.898	7.274	11.155	10.182	10.344
P/S comp stress due to LL (ksi)		0.144	0.180	0.190	0.291	0.265	0.270
Net btm stress after P/S gain (ksi)		0.279	0.037	-0.030	**-0.713**	-0.542	-0.570

HL-93 Design w/ Impact = 1.33 for design and 1.2 for permit trucks DF = 0.5321

0.19*sqrt(f'c) = 0.484 ksi # strand = 42

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1555	1943	1435.39	2201.30	2009.42	2041.35
Stress (ksi)	1.024	1.345	1.681	1.242	1.904	1.738	1.766
Net design bottom stress (ksi)		-0.321	-0.657	-0.218	-0.880	-0.714	-0.742
P/S Gain due to LL (ksi)		6.691	8.361	6.177	9.465	8.646	8.783
P/S comp stress due to LL (ksi)		0.166	0.208	0.153	0.235	0.215	0.218
Net btm stress after P/S gain (ksi)		-0.155	-0.449	-0.064	-0.645	-0.499	-0.523

Properties	**HL-93 design**	**HS20 design**
Girder composite area	1115 in^2	1115 in^2
Girder composite bottom section modulus	13,874 in^3	13,874 in^3
Center of gravity of girder	45.20 in	45.20 in
Area of prestressing strand	6.43 in^2	6.73 in^2
Center of gravity of prestressing strand	4.02 in	3.93 in

Note: Distribution factors used assumes potential for permit trucks in adjacent lanes.

Table A5 Tension Stress for HL-93 Design vs. HS20 Design – 160 ft span with 84” precast prestressed Bulb-T girders @ 6.25 ft spacing.

0.24*sqr(1.2*f'c) = 0.712 ksi
0.24*sqr(f'c) = 0.650 ksi
f'c = 7.5 ksi

HL-93 Design Impact = 1.33 DF = 0.475

0.19*sqr(f'c) = 0.520 ksi # strand = 52

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		2094	2618	2041.01	3213.58	3279.65	3282.81
Stress (ksi)	0.751	1.124	1.405	1.096	1.725	1.761	1.762
Net design bottom stress (ksi)		-0.373	-0.654	-0.345	-0.974	-1.010	-1.011
P/S Gain due to LL (ksi)		5.3256	6.657	5.191	8.173	8.341	8.349
P/S comp stress due to LL (ksi)		0.138	0.173	0.135	0.212	0.216	0.217
Net btm stress after P/S gain (ksi)		-0.235	-0.481	-0.210	**-0.762**	**-0.793**	**-0.795**

HS20-no-tension Design Impact = 1.222 DF = 0.477 Impact = 1.33 with permit trucks

0.095*sqr(f'c) = 0.260 ksi # strand = 54

Load	Dead + P/S	HS20	HS25	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		1553	1941	2049.16	3226.42	3292.75	3295.93
Stress (ksi)	0.885	0.834	1.042	1.100	1.732	1.768	1.769
Net design bottom stress (ksi)		0.051	-0.157	-0.215	-0.847	-0.883	-0.884
P/S Gain due to LL (ksi)		3.932	4.915	5.188	8.169	8.337	8.345
P/S comp stress due to LL (ksi)		0.106	0.132	0.140	0.220	0.225	0.225
Net btm stress after P/S gain (ksi)		0.157	-0.025	-0.075	-0.627	-0.658	-0.660

HL-93 Design Impact = 1.33 for design and 1.20 for permit trucks DF = 0.475

0.19*sqr(f'c) = 0.520 ksi # strand = 52

Load	Dead + P/S	0.8*HL-93	HL-93	CTP-3	STP-5B	STP-5BW	STP-5C
Moment (ft-k)		2094	2618	1841.51	2899.47	2959.08	2961.93
Stress (ksi)	0.751	1.124	1.405	0.989	1.556	1.588	1.590
Net design bottom stress (ksi)		-0.373	-0.654	-0.238	-0.805	-0.837	-0.839
P/S Gain due to LL (ksi)		5.3256	6.657	4.683	7.374	7.526	7.533
P/S comp stress due to LL (ksi)		0.138	0.173	0.121	0.191	0.195	0.195
Net btm stress after P/S gain (ksi)		-0.235	-0.481	-0.116	-0.614	-0.642	-0.644

Properties	**HL-93 design**	**HS20 design**
Girder composite area	1207 in^2	1207 in^2
Girder composite bottom section modulus	22,354 in^3	22,354 in^3
Center of gravity of girder	59.27 in	59.27 in
Area of prestressing strand	7.96 in^2	8.26 in^2
Center of gravity of prestressing strand	4.90 in	4.93 in

Note: Distribution factors used assumes potential for permit trucks in adjacent lanes.