Implication of increased live loads on the design of precast concrete bridge girders

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- This paper examines the effects of increasing AASHTO LRFD specifications HL93 live loads on the design of precast, prestressed concrete girders.
- Charts that relate girder size and spacing to the span length as a function of the 28-day concrete compressive strength and environmental classifications were developed for HL93 and 1.5 × HL93 live loads.
- It was shown that the increase in live loads can be economically accommodated by increasing the 28-day concrete compressive strength.
- The charts provide a simple, practical method for optimizing the precast concrete girder size and spacing.

Precast, prestressed concrete girders are often used in the construction of medium-span bridges[1] with span lengths varying from 12 m (40 ft) to 54 m (177 ft). The precast concrete girders are cast in a casting yard and then transported to the site, where they are erected using mechanical cranes (**Fig. 1**). Pretensioning offers a cost-effective solution compared with posttensioning by obviating the need for bursting and spalling reinforcing steel in the end zones.

Standard American Association of State Highway and Transportation Officials (AASHTO) girders[2] (Type II to VI) are commonly used for span lengths varying from 12 to 45 m (39 to 148 ft). These girders are designed according to AASHTO *Standard Specifications for Highway Bridges*[3] or *AASHTO LRFD Bridge Design Specifications*,[4] and depending on the span lengths, applied loads, and environmental classifications, the precast concrete girders' sizes and spacing are determined (**Fig. 2**). Bulb tees and modified Type VI girders are sometimes used to accommodate longer spans (up to 54 m [177 ft]). Designs are usually based on trial and error using commercial software to determine the optimal girder spacing and number, size, and profile of prestressing strands.

This paper present graphical plots (charts) that can easily predict the precast, prestressed concrete girder size and spacing as a function of the span length for a practical range of 28-day concrete compressive strengths and envi-

Fabrication of girders

Transportation of girders

Erection of girders

Aerial view of elevated roads, bridges, and ramps

Figure 1. Precast, prestressed concrete girders for Sowa Island, Abu Dhabi, United Arab Emirates. Photo courtesy of Al-Meraikhi Industrial Complex, UAE.

ronmental classifications. These solutions were obtained from a parametric analysis based on the concrete service load stress limits in AASHTO LRFD specifications. The analysis was first conducted for regular HL93 live loads and was then extended to HL93 increased by 50% (HL93 × 1.5),[5] a live load that more realistically represents the actual complex truck weights, axle configurations, and truck weight populations.[6] The implication of increasing the design live loads on the precast concrete girder size and spacing was examined, and alternative girder configurations based on increasing the 28-day concrete strength were presented. This helped optimize the design of precast, prestressed concrete bridge girders subjected to increased live loads.[5] Application of the optimized solution was illustrated with a comprehensive numerical example.

Assumptions

The following assumptions were used for the parametric studies performed for AASHTO Type II to modified Type VI girders (**Fig. 3**) for bridge structures of span lengths from 12 m (39 ft) to 54 m (177 ft):

- The 28-day concrete compressive strength for the precast, prestressed concrete girders varied from 40 to 50 MPa (5800 to 7200 psi) for normalweight concrete.

- The optimal number of 15 mm diameter (0.6 in.) prestressing strands was determined based on the service load stress limits in the initial (release) stage.

- The optimal girder spacing S was determined based on the concrete service load stress limits in the final stage for a girder spacing range of 1.1 m < S ≤ 3 m (3.6 ft < S ≤ 9.9 ft) and a topping slab thickness of 200 mm (8 in.) for HL93 live load and 250 mm (10 in.) for HL93 live load increased by 50%.

- The parametric service load stress analysis was conducted using computer software based on AASHTO LRFD specifications.

Identification of parameters

The main parameters involved in the design of precast, prestressed concrete girders are the span length L, the

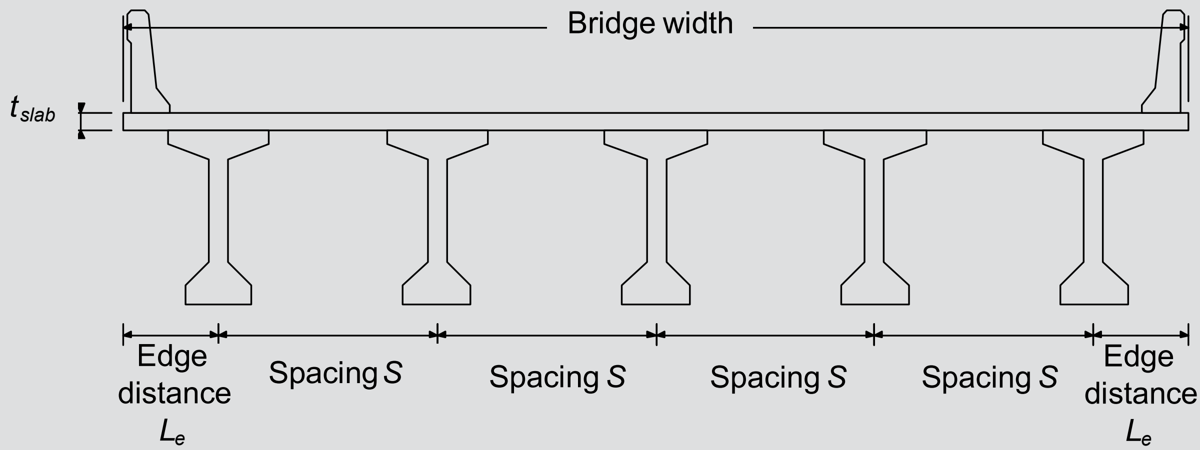


Figure 2. Typical cross section of a bridge structure that contains precast, prestressed concrete girders. Note: The girder spacing and edge distance are usually set based on practical limits: 1.1 m (3.6 ft) < S ≤ 3 m (9.9 ft); L_e ≤ 1.1 m (3.6 ft); 20 cm (8 in.) ≤ t_{slab} ≤ 25 cm (10 in.); 40 MPa (5800 psi) ≤ f_c' ≤ 50 MPa (7200 psi). f_c' = 28-day concrete compressive strength; L_e = distance from centerline of exterior girder to edge of slab; S = girder spacing; t_{slab} = thickness of concrete slab.

girder size, the 28-day concrete compressive strength f_c', the prestressing strand size, the design loads (mainly live load), the concrete service load stress limits, and the girder spacing S. Practical ranges of these parameters are defined as follows.

Span length and girder size

The girder size is a function of the girder design span length L. Commonly used AASHTO Type II to VI girders were adopted in this paper (Fig. 2) for span lengths of 12 m (39 ft) to 45 m (148 ft) in addition to a modified girder that was introduced for span lengths up to 54 m (177 ft). Figure 3 shows the girder dimensions and section properties of Types II to modified VI girders.

Based on a simple span, the following span length ranges were adopted for each girder type:

- Type II: 12 m ≤ L < 20 m (39 ft ≤ L < 66 ft)
- Type III: 18 m ≤ L < 26 m (59 ft ≤ L < 85 ft)
- Type IV: 24 m ≤ L < 34 m (79 ft ≤ L < 111 ft)
- Type V: 30 m ≤ L < 38 m (98 ft ≤ L < 125 ft)
- Type VI: 36 m ≤ L < 45 m (118 ft ≤ L < 148 ft)
- Modified Type VI: 42 m ≤ L < 54 m (138 ft ≤ L < 177 ft)

The span length ranges were set for the maximum live load HL93 × 1.5 and 28-day concrete strength f_c' of 50 MPa (7200 psi), and their upper bound could be extended for the smaller HL93 loading.[2]

28-day concrete compressive strength

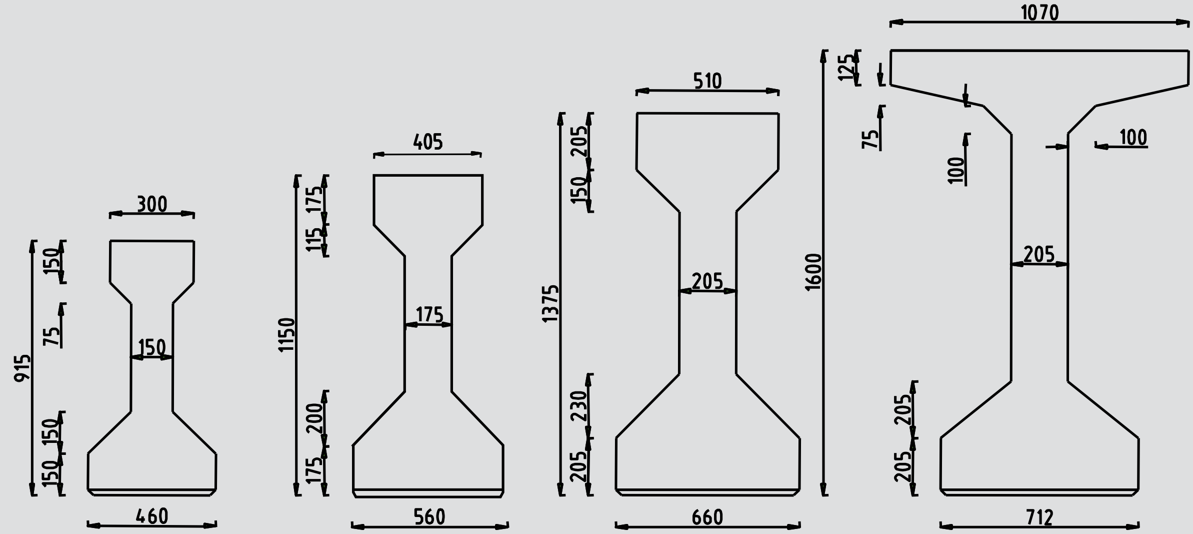
The 28-day concrete strength f_c' for the precast, prestressed concrete girders varied from 40 to 50 MPa (5800 to 7200 psi) in increments of 5 MPa (700 psi) for normalweight concrete. A 28-day concrete strength f_c' of 50 MPa is highly recommended[2] (as shown in the parametric analysis later). However, lower strengths were also considered because they provide a basis of comparison and, contrary to U.S. practice, 50 MPa may be difficult to achieve on a consistent basis in some locations.[2]

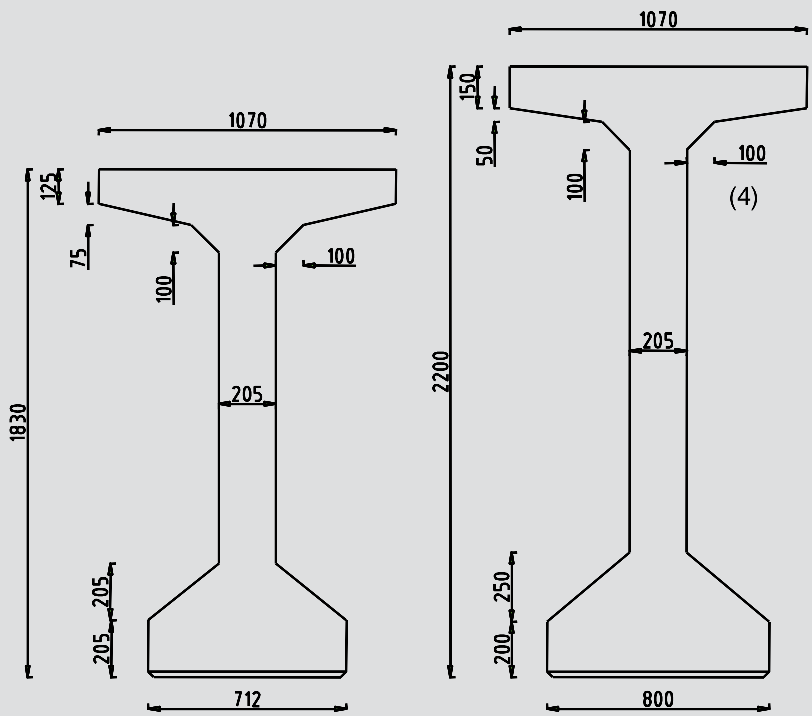
The concrete strength at release f_{ci}' was taken as 0.8 f_c' (commonly used for the concrete strength range considered):[2]

- f_{ci}' = 32 MPa (4600 psi) for f_c' = 40 MPa (5800 psi)
- f_{ci}' = 36 MPa (5200 psi) for f_c' = 45 MPa (6500 psi)
- f_{ci}' = 40 MPa (5800 psi) for f_c' = 50 MPa (7200 psi)

Prestressing steel

The prestressing steel area was based on 15 mm (0.6 in.) diameter, low-relaxation strands where the area per strand A_{strand} was 140 mm^2 (0.217 in.^{2}). The minimum tensile strength f_{pu} was 1860 MPa (270 ksi). The jacking strength f_{pj} was taken as 75% of f_{pu}, which is equal to 1395 MPa (202.5 ksi).[2,4] The strand distribution was based on a 75 mm (3 in.) cover to exposed surfaces (measured from the centerline of the strand to the edge of the exposed surface) and 50 mm (2 in.) spacing measured between the centerline of strands elsewhere.[2,4]





Section properties

Girder type	Y_{bot}, m	A_{nc}, m^2	I_{cg}, m^4
II	0.4	0.24	0.021
III	0.52	0.36	0.053
IV	0.63	0.51	0.11
V	0.81	0.65	0.22
VI	0.92	0.7	0.31
VIM	1.08	0.81	0.52

Figure 3. AASHTO Type II to Type VI girders commonly used for medium-span bridges in addition to a modified Type VI girder that was specifically developed for the new Khalifa Port project in Abu Dhabi, United Arab Emirates. Note: Designed according to British Standards.[7,8] All dimensions are in millimeters. A_{nc} = area of the prestressed concrete girder; I_{cg} = moment of inertia of precast concrete girder; L = span length; Y_{bot} = distance from neutral axis of precast concrete girder to the bottom fiber. 1 mm = 0.0394 in.; 1 m = 3.28 ft.

Concrete service stress limits

The concrete service load stress limits were used in the initial stage (at release) for identifying the optimal number of prestressing strands and in the final stage for identifying the optimal girder spacing. These are presented as follows.

At release (initial stresses) Based on AASHTO LRFD specifications, the allowable concrete service load compressive stress at release $(\sigma_i)_C$ was $0.6 f'_{ci}$ and the allowable concrete service load tensile stress $(\sigma_i)_T$ in MPa was $0.25\sqrt{f'_{ci}} < 1.38$ MPa ($3\sqrt{f'_{ci}} < 0.2$ ksi) for noncompressed zones without bonded reinforcement and $0.63\sqrt{f'_{ci}}$

Table 1. Allowable concrete stresses at release

28-day concrete strength f'_c, MPa	Concrete release strength f'_{ci}, MPa	Allowable compressive stress $(\sigma_i)_C = 0.6 f'_{ci}$, MPa	Allowable tensile stress without bonded steel $(\sigma_i)_T = 0.25\sqrt{f'_{ci}}$, MPa	Allowable tensile stress with bonded steel $(\sigma_i)_T = 0.63\sqrt{f'_{ci}}$, MPa
40	32	19.2	1.4 use 1.38	3.6
45	36	21.6	1.5 use 1.38	3.8
50	40	24.0	1.6 use 1.38	4.0

Note: 1 MPa = 145 psi.

Table 2. Allowable concrete stresses in the final stage

28-day concrete strength f'_c, MPa	Normal environments $(\sigma_e)_T = 0.5\sqrt{f'_{ci}}$, MPa	Aggressive environments $(\sigma_e)_T = 0.25\sqrt{f'_{ci}}$, MPa	Extremely aggressive environments $(\sigma_e)_T = 0$	Allowable compressive stress at 28 days $(\sigma_e)_C = 0.6 f'_c$, MPa
40	3.16	1.58	0	24
45	3.35	1.68	0	27
50	3.54	1.77	0	30

Note: f'_{ci} = concrete compressive strength at release; $(\sigma_e)_C$ = allowable concrete service load compressive stress in the final stage; $(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage. 1 MPa = 145 psi.

($7.5\sqrt{f'_{ci}}$) (modulus of rupture) for noncompressed zones with bonded reinforcement.

The allowable concrete service load tensile stress at release $(\sigma_i)_T$ in MPa of $0.25\sqrt{f'_{ci}} < 1.38$ MPa ($3\sqrt{f'_{ci}} < 0.2$ ksi) was adopted for midspan zones and $0.63\sqrt{f'_{ci}}$ ($7.5\sqrt{f'_{ci}}$) for end zones. **Table 1** summarizes numerical values for these allowable concrete service load stresses as a function of the concrete strength f'_{ci} at release. In this paper a plus sign (+) designates tension and a minus sign (–) designates compression.

Final stresses The allowable concrete service load stresses in the final stage were a function of the environmental classification of the bridge structure.[2,4] The allowable tensile stress $(\sigma_e)_T$ was $0.5\sqrt{f'_{ci}}$ in MPa ($6\sqrt{f'_{ci}}$ in ksi) for normal environments, $0.25\sqrt{f'_{ci}}$ in MPa ($3\sqrt{f'_{ci}}$ in ksi) for aggressive environments, and 0 for extremely aggressive environments. **Table 2** summarizes numerical values for these allowable concrete tensile stresses $(\sigma_e)_T$ as a function of the 28-day concrete strength f'_c adopted in this paper.

Design loads

In the parametric study, the design loads consisted of dead load and live load typically used in the design of precast, prestressed concrete bridge girders. These loads are defined as follows.

Dead load Dead load consists of the self-weight of the precast concrete girder and reinforced concrete topping slab and superimposed dead loads. The self-weight of the girder and slab is based on a unit weight of 25 kN/m^3 (150 lb/ft^3) for normalweight concrete.[2,4] The slab thickness was taken as 200 mm (8 in.) for HL93 live load and 250 mm (10 in.) for 1.5 × HL93 (Fig. 2) (considered a noncomposite dead load based on shored systems).

The composite dead loads comprised two traffic barriers of 10 kN/m (680 lb/ft) uniform load each (Fig. 2), a future wearing surface load of 2.5 kN/m^2 (50 lb/ft^2) (based on 100 mm thick [4 in.] asphalt), and a utility load of 1 kN/m^2 (20 lb/ft^2). These loads were actually larger than normal bridge loads[2,4] because they were based on stringent design criteria.[5]

Live load The design live load consisted of AASHTO LRFD specifications HL93 truck, tandem, and lane loading, that is a combination of a 325 kN (72 kip) truck or 220 kN (50 kip) tandem load (whichever governed) and a 9.3 kN/m (0.64 kip/ft) lane load. The 325 kN truck load comprised three axles of 35 kN (8 kip) (front axle) and 145 kN (32 kip) (middle and rear axles). The spacing between the front and middle axles was 4.3 m (14 ft), while that between the middle and rear axles varied between 4.3 and 9 m (29 ft). The 220 kN tandem was equally distributed between two axles spaced at 1.2 m (4 ft). For span lengths 12 m ≤ L ≤ 54 m (39 ft ≤ L ≤ 177 ft), the combination of the HL93 truck (325 kN) multiplied by 1.33 for impact and lane load[4] (9.3 kN/m [0.64 kip/ft]) governed.

The parametric analysis was first conducted for the HL93 truck plus lane loadings.[4] The analysis was then extended by increasing the AASHTO HL93 truck and lane loadings by 50% to 1.5 × HL93, a live load that is currently being adopted by relevant authorities[5] for the design of bridge structures. This 50% increase was determined based on the British Standards[7,8] design live load (designated as HA and HB) that is larger than AASHTO LRFD specifications HL93 live load.

The British Standards[7,8] HA load consists of a uniformly distributed load w equal to $(336)(1/L)^{0.67}$ kN/m ($[50][1/L]^{0.67}$ kip/ft) for $L < 50$ m (164 ft) and w equal to $(36)(1/L)^{0.1}$ kN/m ($[2.8][1/L]^{0.1}$ kip/ft) for $L > 50$ m (164 ft) with a moving load of 120 kN (27 kip). For the span lengths considered in this paper (12 m $\le L \le$ 54 m [39 ft $\le L \le$ 177 ft]), the uniformly distributed load w varied from 63.5 kN/m (4.3 kip/ft) to 24.2 kN/m (1.7 kip/ft), much higher than AASHTO LRFD specifications HL93 lane load of 9.3 kN/m (0.64 kip/ft). Moreover, the HB[7,8] load normally consists of four 300 kN (67.5 kip) axles spaced at 1.8 m (6 ft) between the first and second axles and the third and fourth axle,s with a spacing of 6 to 26 m (20 to 86 ft) between the second and third axles. This resulted in a gross truck weight of 1200 kN (270 kip), which is much higher than the AASHTO LRFD specifications HL93 truck load of 325 kN (72 kip) multiplied by 1.33 (equal to 432 kN [96 kip]) for impact. Based on a previous study,[9] the HL93 live loads increased by 50% (on average) that were adopted in this study were found to compare with the British Standards HA and HB live loads.

Girder spacing

The upper and lower bounds of the girder spacing S (centerline to centerline of girder, Fig. 2) were set at 3 m (9.9 ft) and 1.1 m (3.6 ft), respectively. This girder spacing range of 1.1 m < $S \le$ 3 m and the constant slab thickness of 200 mm (8 in.) for HL93 live load and 250 mm (10 in.) for 1.5 × HL93 (Fig. 2) allowed the AASHTO LRFD specifications live load distribution factor formulas to be used. The maximum spacing of 3 m was provided so that the 200 mm thick (for HL93 live load) and the 250 mm thick (for 1.5 × HL93 live load) concrete slabs were not overreinforced. The edge distance L_e (that is, the distance from the centerline of the exterior girder to the edge of the slab) was limited to 1.1 m for similar reasons in the overhangs (Fig. 2).

Optimization of the design

In addition to the range of key parameters previously defined, the maximum number of prestressing strands and girder spacing were determined based on the concrete service load stress limits in the initial stage and in the final stage, respectively.

Maximum number of strands

The maximum number of 15 mm diameter (0.6 in.) strands per girder was determined based on the initial (release) stresses in the extreme fibers of the concrete section given by Eq. (1) (bottom fiber in compression) and Eq. (2) (top fiber in tension) as follows:[2,4]

$$-F_i\left(\frac{1}{A_{nc}}+\frac{e}{(S_b)_{nc}}\right)+\frac{M_{SW}}{(S_b)_{nc}}\le(\sigma_i)_C \tag{1}$$

where

F_i = initial prestress force after short-term losses

A_{nc} = cross-sectional area of the prestressed concrete girder

e = prestressing tendon eccentricity

$(S_b)_{nc}$ = bottom-fiber noncomposite section modulus

M_{SW} = self-weight dead load moment

$(\sigma_i)_C$ = allowable concrete service load compressive stress at release (Table 1)

$$-F_i\left(\frac{1}{A_{nc}}-\frac{e}{(S_t)_{nc}}\right)-\frac{M_{SW}}{(S_b)_{nc}}\le(\sigma_i)_T \tag{2}$$

where

$(S_t)_{nc}$ = top-fiber noncomposite section modulus

$(\sigma_i)_T$ = allowable concrete service load tensile stress at release (Table 1)

Based on Eq. (1) and (2), the maximum number of prestressing strands was determined for the range of parameters defined in this paper: 12 m $\le L \le$ 54 m (39 ft $\le L \le$ 177 ft), Type II to modified VI girders (Fig. 3), and f_c' of 40 MPa (5800 psi), 45 MPa (6500 psi), and 50 MPa (7200 psi). The governing stresses in the initial stage were compressive stresses in the bottom fiber of the concrete section in the midspan region as the precompressed tensile zone was subjected to self-weight dead load only at transfer (Eq. [1]). Tensile stresses at transfer were then checked using Eq. (2). Stresses in the girder end zones[10] were controlled by debonding (shielding) of strands according to AASHTO LRFD specifications (the maximum number of strands that could be debonded per girder was 25% of the total number of strands and 40% of the number of strands in a row) and/or using harped strands (a maximum of six strands per girder were harped based on common practice to avoid providing special anchoring systems and bulky formwork to resist the vertical component of the prestress force).

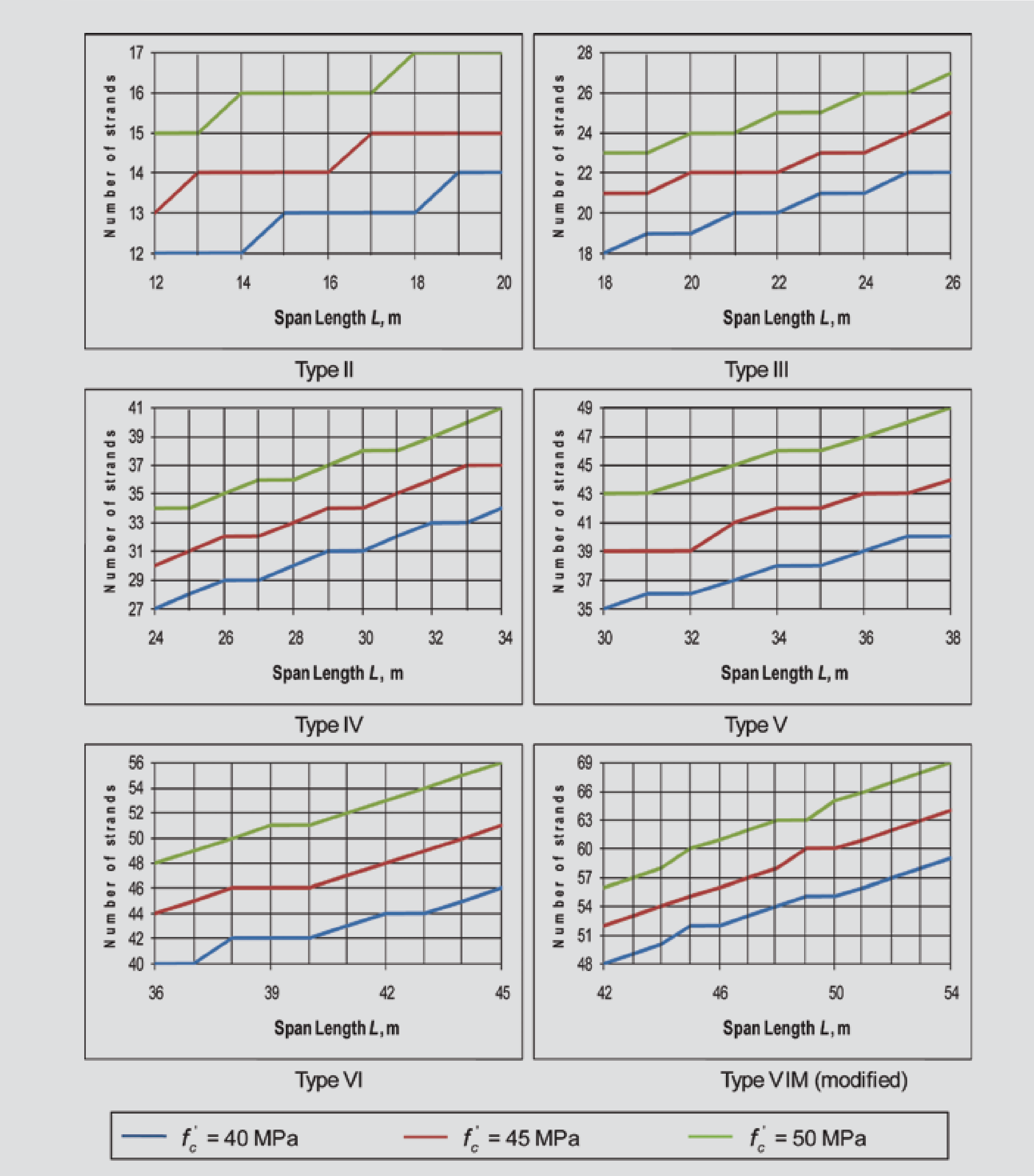


Figure 4. Maximum number of 15 mm diameter (0.6 in.) strands that can be accommodated per girder based on the allowable concrete service load stresses at release. Note: f'_c = 28-day concrete strength. 1 m = 3.28 ft; 1 MPa = 145 psi.

The parametric analysis was conducted using computer software based on AASHTO LRFD specifications for the span lengths (Fig. 3) considered in multiples of 1 m (3.3 ft). Initial prestress losses (elastic shortening) were directly calculated. Consequently, the maximum number of strands that could be accommodated per girder was plotted in **Fig. 4** as a function of the span length L for the span length range (12 m ≤ L ≤ 54 m [39 ft ≤ L ≤ 177 ft]), for AASHTO Type II to modified Type VI girders as a function of the 28-day concrete strength range (f'_c equal to 40 MPa [5800 psi], 45 MPa [6500 psi], and 50 MPa [7200 psi]).

The number of 15 mm diameter (0.6 in.) strands that were used in this paper varied from a minimum of 12 for Type II girder (f_c' = 40 MPa [5800 psi]) up to a maximum of 69 for modified Type VI girder (f_c' = 50 MPa [6500 psi]) (Fig. 4). Precast concrete fabricators in certain regions prefer limiting the number of 15 mm diameter strands to a maximum of 50; otherwise the bulkhead capacity should be increased to withstand the magnitude of the prestress force at transfer.

Maximum girder spacing

The spacing S between girders depends on the design span length L, the 28-day concrete strength f_c', the applied dead and live loads, and the allowable concrete stresses. In this paper, the parametric analysis for the girder spacing was performed based on the maximum number of strands that was determined in the initial stage (Fig. 4). The girder spacing was maximized based on the serviceability check for the maximum tensile stress in the positive moment region in the final stage. This bottom-fiber tensile stress check was performed based on AASHTO LRFD specifications service III load combination using Eq. (3) as follows:

$$-F\left(\frac{1}{A_{nc}}+\frac{e}{(S_b)_{nc}}\right)+\frac{M_{ncdl}}{(S_b)_{nc}}+\frac{M_{cdl}}{(S_b)_c}+(0.8)\left[\frac{M_{LL+I}}{(S_b)_c}\right]\le(\sigma_e)_T \quad (3)$$

where

F = effective prestress force after total losses

M_{ncdl} = noncomposite dead load moment

M_{cdl} = composite dead load moments

M_{LL+I} = live load moment plus impact

$(S_b)_c$ = bottom-fiber composite section modulus

$(\sigma)_T$ = allowable concrete service load tensile stress in the final stage

Compressive stresses were checked not to exceed the allowable concrete service load stress limits. AASHTO LRFD specifications require providing this stress check for different load combinations. In this study, the service I load combination[4] that comprises live load governed and is given by Eq. (4) as follows:

$$-F\left(\frac{1}{A_{nc}}+\frac{e}{(S_t)_{nc}}\right)+\frac{M_{ncdl}}{(S_t)_{nc}}+\frac{M_{cdl}}{(S_t)_c}+\left[\frac{M_{LL+I}}{(S_t)_c}\right]\le(\sigma_e)_C \quad (4)$$

where

$(\sigma_e)_C$ = allowable concrete service load compressive stress in final stage (for service I load combination = 0.6 f_c')

The parametric analysis for selecting the maximum girder spacing was conducted for span lengths (Fig. 3) ($12 \text{ m} \le L \le 54 \text{ m}$ [$39 \text{ ft} \le L \le 177 \text{ ft}$]) in increments of 1 m (3.3 ft) using computer software based on AASHTO LRFD specifications. Final prestress losses (creep, shrinkage, and steel relaxation) were directly calculated by the software based on AASHTO LRFD specifications' approximate method without accounting for elastic gain (elastic gains usually help reducing losses by 3% to 4%). Live load distribution factors were conservatively calculated based on AASHTO equations, though they could be more accurately determined based on grillage models[11] considering the true bridge geometry. Composite dead loads were assumed to be equally distributed among the number of girders.[12]

The optimal girder spacing as a function of the span length was plotted for Type II to modified VI girders for f_c' equal to 40 MPa (5800 psi) (**Fig. 5**), f_c' equal to 45 MPa (6500 psi) (**Fig. 6**), and f_c' equal to 50 MPa (7200 psi) (**Fig. 7**). Each figure contains two charts, one for HL93 live loads[4] and the other for 1.5 × HL93 live loads.[5] Each chart includes plots for the environmental classifications noted in this paper; for example, $(\sigma_e)_T$ of $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) for a normal environment, $(\sigma_e)_T$ of $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$) for an aggressive environment, and $(\sigma_e)_T$ of zero for an extremely aggressive environment.

The charts developed from the parametric study may also serve as design aids because they allow determining the precast concrete girder configurations and prestressing strand distributions for a wide range of bridge lengths, live loads, concrete strengths, and service load stress limits by interpolation.

Interpretation of results

Based on the charts in Fig. 5, 6, and 7, the effects of three variations were carefully examined: increasing the 28-day concrete compressive strength f_c', reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ as a function of the environmental classification, and increasing the design live load from HL93[4] to 1.5 × HL93.[5]

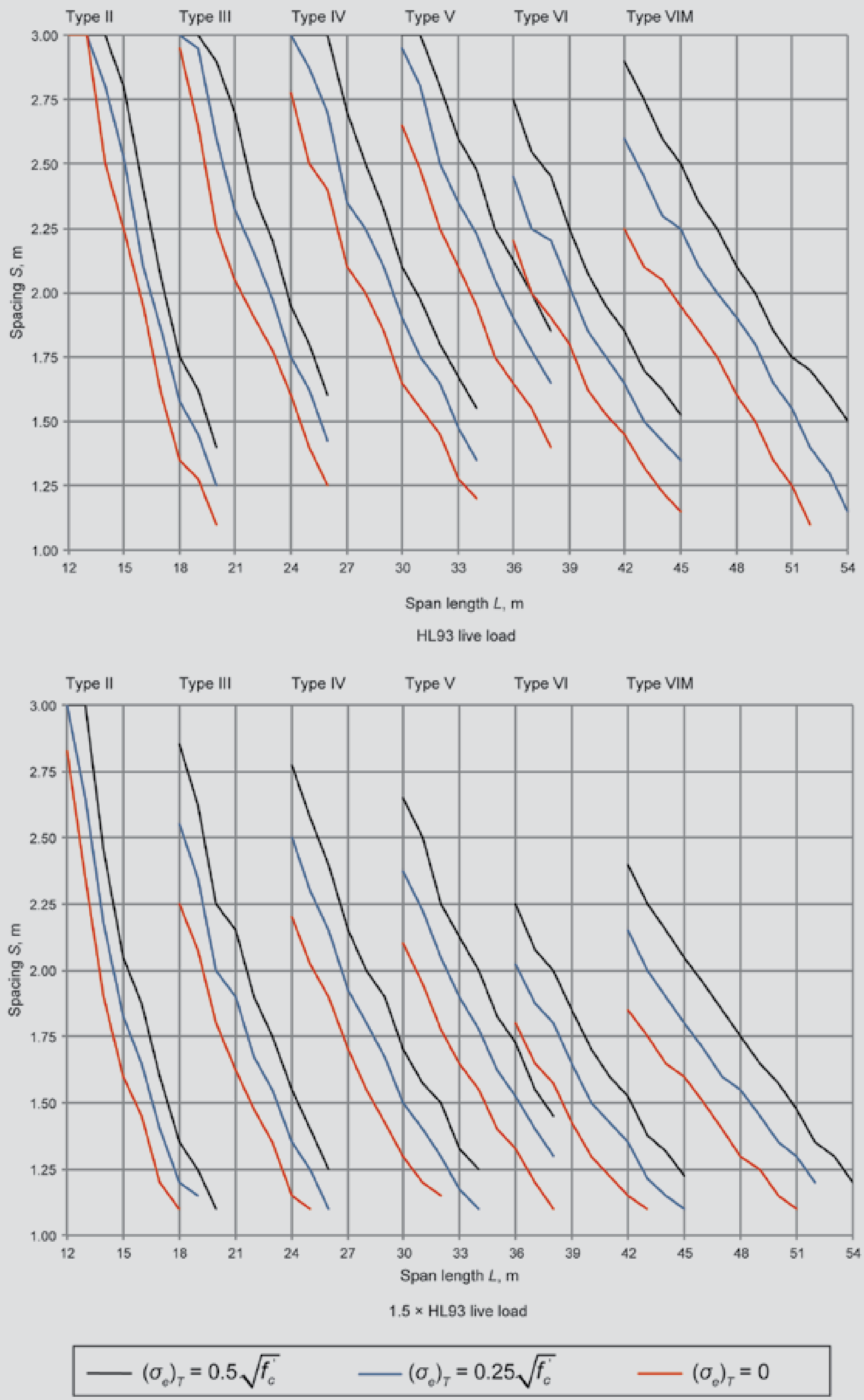


Figure 5. Variation in girder spacing S as a function of the span length L for HL93 and 1.5 × HL93 live loads based on a 28-day concrete strength f'_c of 40 MPa (5800 psi). Note: $(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage. 1 m = 3.28 ft.

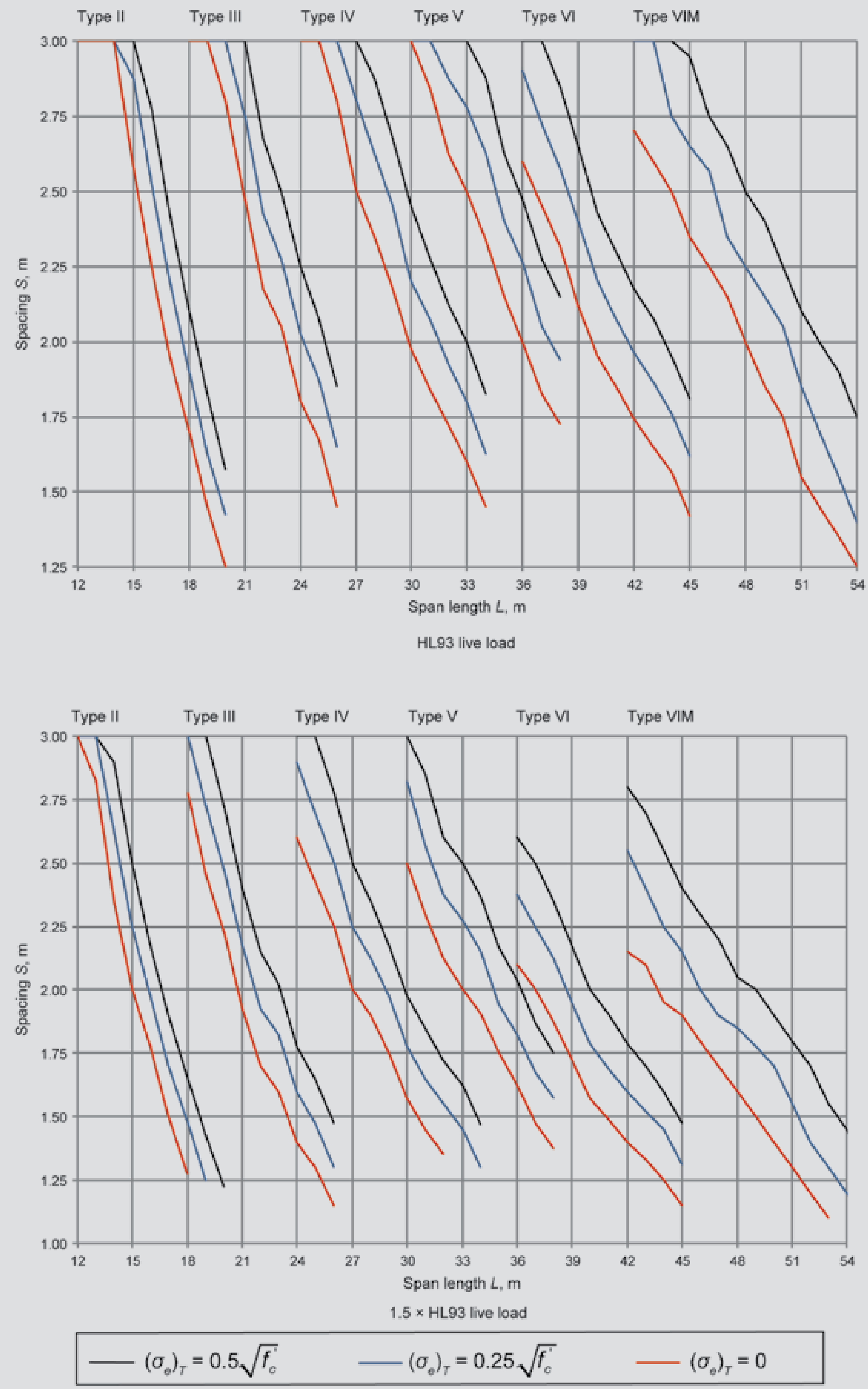


Figure 6. Variation in girder spacing S as a function of the span length L for HL93 and 1.5 × HL93 live loads based on a 28-day concrete strength f_c' of 45 MPa (6500 psi). Note: $(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage. 1 m = 3.28 ft.

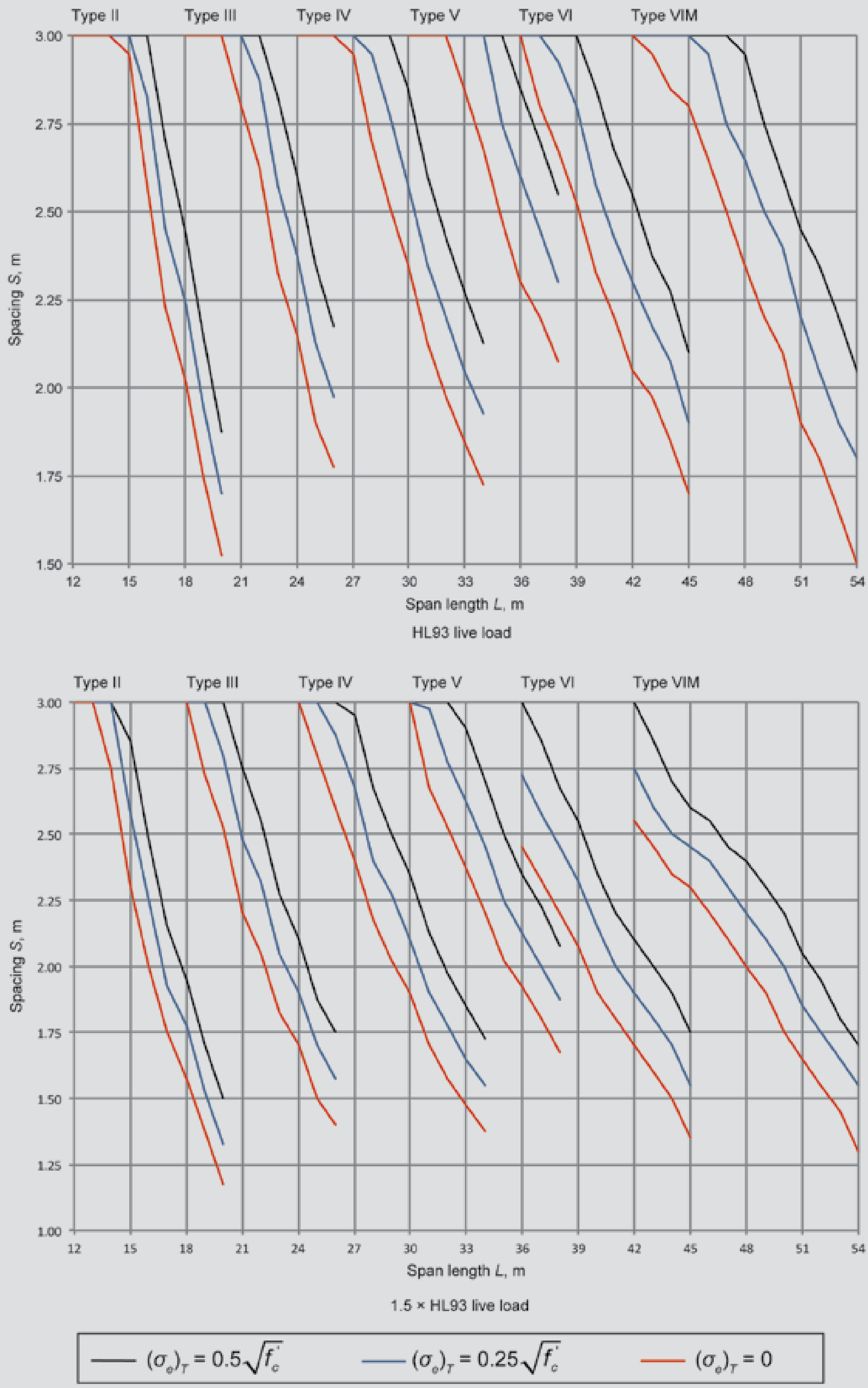


Figure 7. Variation in girder spacing S as a function of the span length L for HL93 and 1.5 × HL93 live loads based on a 28-day concrete strength f_c' of 50 MPa (7200 psi). Note: $(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage. 1 m = 3.28 ft.

Table 3. Effects of varying the concrete strength, live load, and allowable stresses on spacing

Increase in concrete strength f_c', MPa	Increase in girder spacing S, %		Reduction in tensile stress $(\sigma_e)_T$, MPa	Reduction in girder spacing S, %		Increase in f_c' due to increased live load of 1.5 × HL93, MPa	Reduction in girder spacing S, %	
	Average	Maximum		Average	Maximum		Average	Maximum
40 to 45	20	30	$0.5\sqrt{f_c'}$ to $0.25\sqrt{f_c'}$	10	15	40	20	30
45 to 50	20	30	$0.25\sqrt{f_c'}$ to 0	10	15	45	17	25
40 to 50	45	70	$0.5\sqrt{f_c'}$ to 0	20	25	50	15	22

Note: 1 MPa = 145 psi.

Effect of increasing the 28-day concrete compressive strength

The effect of increasing the concrete strength f_c' on the girder spacing S was examined first. Based on the charts, it was shown that increasing f_c' from 40 to 45 MPa (5800 to 6500 psi), and from 45 to 50 MPa (7200 psi) would increase S by approximately 20% for HL93 and 1.5 × HL93 live loads. This implies that the increase is about 40% if f_c' is increased from 40 to 50 MPa. Those increases were determined as average values based on the ratios of the ordinates of the plots from Fig. 5, 6, and 7, for HL93 live load and for 1.5 × HL93 live load. **Table 3** lists the increases in girder spacing (maximum and average values).

Effect of reducing the allowable concrete service load tensile stress

The effect of reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) (normal environment) to $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$) (aggressive environment) and zero (extremely aggressive environment) on the girder spacing S was also examined. The graphs in Fig. 5, 6, and 7 show that reducing the tensile stress from $0.5\sqrt{f_c'}$ to $0.25\sqrt{f_c'}$ and from $0.25\sqrt{f_c'}$ to zero necessitated reducing the girder spacing by an average of about 10%. The greatest reductions in spacing occurred at the greatest span lengths.

Alternatively, the girder spacing can be maintained by increasing the 28-day concrete strength by 5 MPa (700 psi). For example f_c' can be increased from 40 to 45 MPa (5800 and 6500 psi) or 45 to 50 MPa (7200 psi) if $(\sigma_e)_T$ is reduced from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) to $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$), or $0.25\sqrt{f_c'}$ to zero.

Effect of increasing the design live load

Last, the effect of increasing the design live load by 50% on the girder spacing was examined.

From Fig. 5 (f_c' equal to 40 MPa [5800 psi]), the girder spacing S would reduce by 20% if the design live load was increased from HL93 to 1.5 × HL93. From Fig. 6 (f_c' equal to 45 MPa [6500 psi]) the reduction in girder spacing S was 17%, and from Fig. 7 (f_c' of 50 MPa [7200 psi]) it was 15%. These percentages were computed based on average values of the chart ordinates.

As for the case of reducing the allowable concrete service load tensile stress $(\sigma_e)_T$, the effect of increasing the live load could be compensated by increasing the 28-day concrete strength by 5 MPa (700 psi); that is, the girder spacing for f_c' of 40 MPa (5800 psi) and HL93 live load was almost comparable to the case where f_c' was 45 MPa (6500 psi) with a 1.5 × HL93 live load (within 5%), and the girder spacing for f_c' of 45 MPa and HL93 live load was almost comparable to the case where f_c' was 50 MPa (7200 psi) with a 1.5 × HL93 live load (also within 5%).

Effect of reducing the allowable concrete service load tensile stress and increasing the design live load

The worst-case scenario was encountered when the design live load was increased by 50% (from HL93[4] to 1.5 × HL93[5]) and the allowable concrete service load tensile stress $(\sigma_e)_T$ was reduced from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) (normal environment) to $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$) (aggressive environment) and to zero (extremely aggressive environment).

Increasing the live load from HL93 to 1.5 × HL93 and reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ from $0.5\sqrt{f_c'}$ to $0.25\sqrt{f_c'}$ ($6\sqrt{f_c'}$ to $3\sqrt{f_c'}$) resulted in reducing the girder spacing by 30%. This 30% reduction was determined as an average value of the ratios of the ordinates of the graphs.

The reduction in girder spacing was 40% if the allowable concrete service load tensile stress $(\sigma_e)_T$ reduced from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) to zero (determined as an average value of the ratios of the ordinates of the graphs in Fig. 5, 6, and 7 for $(\sigma_e)_T$ equal to zero compared with $(\sigma_e)_T$ equal to $0.5\sqrt{f_c'}$).

The reductions in girder spacing could be compensated by increasing the concrete strength to its upper-bound value f_c' of 50 MPa (7200 psi).[2] For the case where the live load increased from HL93 to 1.5 × HL93 and the allowable concrete service load tensile stress $(\sigma_e)_T$ reduced from $0.5\sqrt{f_c'}$ to $0.25\sqrt{f_c'}$ ($6\sqrt{f_c'}$ to $3\sqrt{f_c'}$), the girder spacing from Fig. 5 was comparable to the girder spacing from Fig. 7. For the case where the allowable concrete service load tensile stress $(\sigma_e)_T$ reduced from $0.5\sqrt{f_c'}$ to zero, the girder spacing from Fig. 7 was 10% smaller than the girder spacing from Fig. 5.

Summary

Figures 5, 6, and 7 provide graphs that related the girder size (Type II to modified VI) and spacing (1.1 m < S ≤ 3 m [3.6 ft < S ≤ 9.9 ft]) to the span length (12 m ≤ L ≤ 54 m [39.4 ft ≤ L ≤ 177.2 ft]) as a function of concrete strength f_c' of 40 MPa, 45 MPa, and 50 MPa (5800 psi, 6500 psi, and 7200 psi) and permissible tensile stress $(\sigma_e)_T$ of $0.5\sqrt{f_c'}$, $0.25\sqrt{f_c'}$, and zero ($6\sqrt{f_c'}$, $3\sqrt{f_c'}$, and zero) for HL93 and 1.5 × HL93 live loads. The following conclusions were made from these figures:

- Increasing the 28-day concrete strength f_c' from 40 to 45 MPa (5800 to 6500 psi) and from 45 to 50 MPa (7200 psi) allowed increasing the girder spacing S by 20%.

- Reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) (normal environment) to $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$) (aggressive environment) and from $0.25\sqrt{f_c'}$ (aggressive environment) to zero (extremely aggressive environment) allowed reducing the girder spacing S by 10%. These reductions could be compensated by increasing the 28-day concrete strength by 5 MPa (700 psi).

- Increasing the design live load from HL93 to 1.5 × HL93 allowed reducing the girder spacing S by up to 20%. These reductions could be compensated by increasing the 28-day concrete strength by 5 MPa (700 psi).

- Reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) (normal environment) to $0.25\sqrt{f_c'}$ ($3\sqrt{f_c'}$) (aggressive environment) and increasing the design live load from HL93 to 1.5 × HL93 allowed reducing the girder spacing S by up to 30%. This 30% reduction could be compensated by increasing the 28-day concrete strength to 50 MPa (7200 psi).

- Reducing the allowable concrete service load tensile stress $(\sigma_e)_T$ from $0.5\sqrt{f_c'}$ ($6\sqrt{f_c'}$) (normal environment) to zero (extremely aggressive environment) and increasing the live load from HL93 to 1.5 × HL93 (worst-case scenario)[5] decreased the girder spacing S by 40%. This can be reduced to 10% by increasing the 28-day concrete strength to 50 MPa (7200 psi).

In conclusion, specifying the 28-day concrete strength f_c' as 50 MPa (7200 psi)[2] could result in major cost savings by reducing the number of girders, especially in extreme loadings and environmental conditions.[5]

Numerical example

Description

The applicability of the parametric study was illustrated by considering a 40 m long (132 ft) bridge structure. The length of the precast concrete girders was equal to 39.6 m (131 ft), and the design span length L (between bearings) was equal to 38.8 m (128 ft). The bridge width was 11.2 m (37 ft), which comprised two 3.65 m wide (12 ft) lanes, two 1 m wide (5 ft) shoulders, and two 0.45 m wide (1.5 ft) barriers. The concrete slab thickness t_{slab} was 200 mm (8 in.) for HL93 live load and 250 mm (10 in.) for 1.5 × HL93 live load. Superimposed loads consisted of two barriers with a weight of 10 kN/m (680 lb/ft) each, a 100 mm thick (4 in.) asphalt surface and a 1 kN/m^2 (20 lb/ft^2) utility load. It was required to determine the optimal girder spacing and number of prestressing strands that should withstand the HL93 and 1.5 × HL93 live loads for the various environmental classifications and concrete strength considered in this paper.

Girder size and maximum number of strands

From Fig. 3, a Type VI girder was required for a design length L of 38.8 m (128 ft). From Fig. 4, the maximum number of 15 mm diameter (0.6 in.) strands was 42 for f_c' of 40 MPa (5800 psi), 46 for f_c' of 45 MPa (6500 psi), and 51 for f_c' of 50 MPa (7200 psi).

Prestressed girder spacing

The girder number, spacing S, and edge distance L_e (Fig. 2) were determined based on the charts in Fig. 5, 6, and 7

Table 4. Numerical example results

	f'_c = 40 MPa		f'_c = 45 MPa		f'_c = 50 MPa	
	HL93	**1.5 × HL93**	**HL93**	**1.5 × HL93**	**HL93**	**1.5 × HL93**
$(\sigma_e)_T = 0.5\sqrt{f'_c}$	S = 2.3 m 5 girders L_e = 1 m	S = 1.9 m 6 girders L_e = 0.85 m	S = 2.65 m 5 girders L_e^* = 0.6 m	S = 2.25 m 5 girders L_e = 1.1 m	S = 3 m 4 girders L_e = 1.1 m	S = 2.55 m 5 girders L_e^* = 0.6 m
$(\sigma_e)_T = 0.25\sqrt{f'_c}$	S = 2.05 m 6 girders $L_e^\dagger$ = 0.6 m	S = 1.65 m 7 girders L_e = 0.65 m	S = 2.45 m 5 girders L_e = 0.7 m	S = 2 m 6 girders L_e = 0.6 m	S = 2.8 m 5 girders L_e^* = 0.6 m	S = 2.35 m 5 girders L_e = 0.9 m
$(\sigma_e)_T = 0$	S = 1.85 m 6 girders L_e =0.975 m	S = 1.4 m 8 girders L_e = 0.7 m	S = 2.25 m 5 girders L_e = 1.1 m	S = 1.75 m 7 girders $L_e^\ddagger$ =0.65 m	S = 2.55 m 5 girders L_e^* = 0.6 m	S = 2.1 m 6 girders $L_e^\dagger$ = 0.6 m

* S is reduced to 2.5 m to comply with edge distance L_e limits (Fig. 2).

† S is reduced to 2 m to comply with edge distance L_e limits (Fig. 2).

‡ S is reduced to 1.65 m to comply with edge distance L_e limits (Fig. 2).

Note: f'_c = 28-day concrete strength; L_e = distance from centerline of exterior girder to edge of slab; S = girder spacing; $(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage. 1 m = 3.28 ft; 1 MPa = 145 psi.

as a function of the 28-day concrete strength, design live load, and environmental classification. **Table 4** summarizes results, which are discussed as follows:

- For f'_c of 40 MPa (5800 psi), HL93 live load, $(\sigma_e)_T$ of $0.5\sqrt{f'_c}$ ($6\sqrt{f'_c}$) (normal conditions), the girder spacing S was 2.3 m (7.6 ft). For an 11.2 m (37 ft) width, five Type VI girders with an edge distance L_e of 1 m (3.3 ft) were required (Fig. 2). For f'_c of 45 MPa (6500 psi), though the girder spacing S increased to 2.65 m (8.75 ft), the number of girders remained at five to satisfy the edge distance limits (Fig. 2). If f'_c was increased to 50 MPa (7200 psi), the number of girders reduced to four with S equal to 3 m (9.93 ft).

- If the live load was increased to 1.5 × HL93 live load, S reduced to 1.9 m (6.2 ft) for f'_c of 40 MPa (5800 psi), (for example, six Type VI girders with L_e of 0.85 m [2.8 ft]). For f'_c of 45 MPa (6500 psi), S increased to 2.25 m (7.45 ft) (for example, five girders, which is the same as for normal conditions). For f'_c of 50 MPa (7200 psi), though S increased to 2.55 m (8.4 ft), five girders were also required to comply with the edge distance limits (Fig. 2).

- For f'_c of 40 MPa (5800 psi), HL93 live load and $(\sigma_e)_T$ of $0.25\sqrt{f'_c}$ ($3\sqrt{f'_c}$) (aggressive conditions), S was equal to 2 m (6.6 ft) (for example, six girders). For f'_c of 45 MPa (6500 psi), S increased to 2.45 m (8.1 ft), (for example, five girders, which is the same as for normal conditions). For f'_c of 50 MPa (7200 psi), though S increased to 2.8 m (9.2 ft), five girders were also required to comply with the edge distance limits (Fig. 2).

- If the live load was increased to 1.5 × HL93 live load, S reduced to 1.65 m (5.45 ft) for f'_c of 40 MPa (5800 psi), for example, seven girders. If f'_c was increased to 50 MPa (7200 psi), S increased to 2.35 m (7.8 ft), for example, five girders (same as for normal conditions).

- For f'_c of 40 MPa (5800 psi), HL93 live load and $(\sigma_e)_T$ equal to zero (extremely aggressive conditions), S was equal to 1.85 m (6.1 ft) (for example, six girders). For f'_c of 45 MPa (6500 psi), S increased to 2.25 m (7.4 ft), (for example, five girders, which is the same as for normal conditions). For f'_c of 50 MPa (7200 psi), though S increased to 2.55 m (8.4 ft), five girders were also required to comply with the edge distance limits (Fig. 2).

If the live load was increased to 1.5 × HL93 live load, S reduced to 1.4 m (4.63 ft) (for example, eight girders). For f'_c of 50 MPa (7200 psi), S increased to 2 m (6.6 ft) and the number of girders reduced to six.

The numerical application illustrated the benefits of increasing the concrete strength f'_c on the design of precast concrete girders subjected to increased live loads. Based on HL93 live load, it was shown that increasing f'_c to 50 MPa (7200 psi) reduced the number of girders by one. This effect was more noticeable when the live load was increased to 1.5 × HL93[5] and the environmental classifica-

Trestle bridge after completion

Main bridge during construction

Precast concrete girder lifting

Precast concrete girder placement

Figure 8. Photos of the new Khalifa Port trestle bridge structure and main bridge during and after construction in Abu Dhabi, United Arab Emirates. Photo courtesy of Archirodon Construction, UAE.

tion was set as extremely aggressive, where $(\sigma_e)_T$ was equal to zero, as the number of girders reduced from eight for f_c' of 40 MPa (5800 psi) to six for f_c' of 50 MPa (7200 psi).

Practical application

The numerical application was based on data taken from a recently completed project in the Middle East[13] that comprised a 1000 m long (3300 ft), 28.9 m wide (94.8 ft) main bridge; a 1000 m long (3300 ft), 27.5 m wide (90.2 ft) utility bridge; and a 1640 m long (5380 ft), 12 m wide (39 ft) trestle bridge (**Fig. 8**) subdivided into 40 m (130 ft) spans measured between centerlines of piers and designed according to British Standards[7,8] using a concrete cylinder strength f_c' of 41.7 MPa (6000 psi). It was shown in the numerical example that for extremely aggressive environmental conditions and 1.5 × HL93 live load, AASHTO Type VI girders spaced 1.4 m (4.6 ft) were required for f_c' of 40 MPa (5800 psi), and the spacing *S* increased to 2 m (6.6 ft) for f_c' of 50 MPa (7200 psi). However, 90 tonne (200 kip), modified Type VI girders were fabricated instead to match the larger girder spacing of 2 m (6.6 ft) that was presented in the original design.[13] Based on 40 m (130 ft) spans measured between centerlines of piers, the total number of spans in all three bridges was 90 (25 + 25 + 40), and the total number of modified Type VI precast concrete girders was 905. If f_c' was increased to 50 MPa (7200 psi), the modified Type VI girder spacing would increase to 2.55 m (8.4 ft). For example, the number of girders could be reduced by about 20%, a savings of 180 girders. Alternatively, the smaller Type VI girders could have been used based at the same spacing of 2 m. This clearly illustrates the cost and time savings benefits of increasing the concrete strength.

Conclusion

AASHTO HL93 live loads do not always represent the actual traffic conditions for bridge design, especially in regions where the enforcement characteristics on truck weight distributions are more stringent.[6] Increasing live loads (1.5 × HL93), required a reduction of the girder spacing by 20%. This reduction in girder spacing could be compensated by increasing the concrete strength f_c' by approximately 5 MPa (700 psi). A more severe situation was encountered when the live load was increased and the allowable concrete service load tensile stress $(\sigma_e)_T$ was reduced based on the environmental classification (from 0.5 $\sqrt{f'_c}$ [6 $\sqrt{f'_c}$] for normal environment to 0.25 $\sqrt{f'_c}$ [3 $\sqrt{f'_c}$] for aggressive environment and to zero for extremely aggressive environment). The reduction in girder spacing was noted at 30% to 40% and was greatly improved by increasing the concrete strength to 50 MPa (7200 psi).

Furthermore, the design aids provided in this paper not only set up the basis for optimization but also helped reduce trial and error in predicting the precast concrete girder size and spacing for a wide range of bridge live loads and configurations. Such information paves the way for more rigorous investigations on the effects of new trends of bridge design live loads that could be soon adopted in design specifications.

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Notation

A_{nc} = area of prestressed concrete girder

A_{strand} = area of strand

e = prestressing tendon eccentricity

F = effective prestress force after total losses

F_i = initial prestress force after short-term losses

f'_c = 28-day concrete compressive strength

f'_{ci} = concrete strength at release

f_{pj} = jacking strength

f_{pu} = minimum tensile strength.

I_{cg} = moment of inertia of precast concrete girder

L = span length

L_e = distance from centerline of exterior girder to edge of slab

M_{cdl} = composite dead load moment

M_{LL+I} = live load moment plus impact

M_{ncdl} = noncomposite dead load moment

M_{SW} = self-weight dead load moment

S = girder spacing

$(S_b)_c$ = bottom-fiber composite section modulus

$(S_b)_{nc}$ = bottom-fiber noncomposite section modulus

$(S_t)_{nc}$ = top-fiber noncomposite section modulus

t_{slab} = thickness of concrete slab

w = uniformly distributed load

Y_{bot} = distance from neutral axis of precast concrete girder to the bottom fiber

$(\sigma_e)_C$ = allowable concrete service load compressive stress in the final stage

$(\sigma_e)_T$ = allowable concrete service load tensile stress in the final stage

$(\sigma_i)_C$ = allowable concrete service load compressive stress at release

$(\sigma_i)_T$ = allowable concrete service load tensile stress at release

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Abstract

American Association of State Highway and Transportation Officials (AASHTO) design live loads are sometimes increased as directed by relevant authorities to reflect actual traffic conditions. A 50% increase is adopted in some regions of the Middle East based on comparisons of AASHTO LRFD specifications HL93 live loads with the British Standards HA + HB live loads. Even in some U.S. jurisdictions, the AASHTO live loads may not represent modern truck configurations. This paper examines the effects of increasing AASHTO LRFD specifications HL93 live loads on the design of precast, prestressed concrete girders. A parametric study was first conducted for this purpose. Design aids in the form of charts that relate the precast concrete girder size and spacing to the span length as a function of the 28-day concrete compressive strength and environmental classifications were developed for HL93 and 1.5 × HL93 live loads. It was shown that the increase in live loads can be economically accommodated by increasing the 28-day concrete strength. The charts not only set up the basis of comparisons but also provided a practical solution that can be simply used by precast concrete designers for optimizing the precast concrete girder size and spacing.

Keywords

AASHTO, design aids, HL93, implication, live load, optimization, parametric.

Review policy

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

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