# IMPLICATION OF USING THE BRITISH STANDARDS ON THE DESIGN OF PRECAST, PRESTRESSED CONCRETE AASHTO GIRDERS

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## ABSTACT

AASHTO and British Standards are used internationally for designing bridge structures. Compared to AASHTO, the British Standards (BS 5400-2:2006, BD37/01 and BS 5400-4:1990) live loads are larger and the service load stress criteria can be more stringent. It is common practice in regions where either code can be adopted to increase the AASHTO design live loads by 50% to represent the British Standards' truck weights and axle configurations.

This paper discusses the implications of using the British Standards on the design of precast bridge girders compared to AASHTO LRFD specifications. A numerical application based on a project case study that comprises 3.64 km (11,942 ft) of bridge structures subdivided into 40 m (132 ft) long spans (a total of 90 spans) and supported by 905 girders is considered for this purpose. The original design based on the British Standards and 2.3 m (7 ft - $6^{9}/_{16}$ in.) deep girders is first presented. A cost-effective solution using a reduced girder depth and weight is then developed. For comparisons, results are finally presented based on AASHTO LRFD design specifications.

The information presented in this paper is useful as it highlights new trends in defining bridge design live loads that more realistically represent current populations of complex truck weights and axles distribution.

**Keywords:** AASHTO LRFD, British Standards, service, stress, live load

## INTRODUCTION

Precast, prestressed girders are commonly used in bridge structures.[1] Standard AASHTO girders[2] can accommodate span lengths varying from 12 m (40 ft) up to 48 m (160 ft). The precast girder size and spacing are determined depending on the span length, applied loads and service load stress criteria using an internationally recognized design code such as AASHTO LFD[3], AASHTO LRFD[4] or the British Standards.[5,6,7]

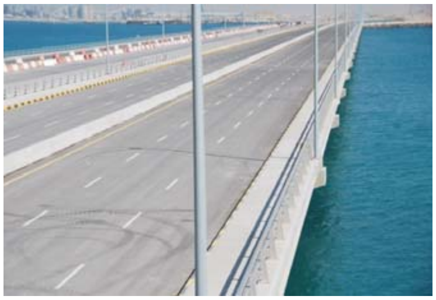
In the United Arab Emirates, both US and British standards are used depending on the project specifications. Compared to AASHTO[3,4], the British Standards' (BS 5400-4:1990, BS 5400-2:2006 and BD37/01 and) live loads are larger and the service load criteria can be more stringent based on the class of concrete and load combination used.

This paper highlights the differences in live load and service load criteria between the British Standards[5,6,7] and AASHTO LRFD[4] specifications and their impact on the precast girder layout by considering a project case study. The project[8] comprises 3.64 km (11,942 ft) of bridge structures subdivided into 90 spans of equal length and supported by 905 precast, prestressed girders that were designed according to the British Standards[5,6,7] as per the project specifications (Fig. 1). Fig. 1a shows the 1 km (3,280 ft) long main bridge (28.9 m wide (94 ft – $9^{13}/_{16}$ in)), Fig. 1b shows the 1 km (3,280 ft) long utility bridge (27.5 m wide (90 ft – $2^{11}/_{16}$ in)) and Fig. 1c shows the 1.64 km (5,380 ft) trestle bridge (12 m wide (39 ft – $4^{7}/_{16}$ in)). All spans are 40 m (131.2 ft) long (measured between centerline of piers), simply supported on laminated reinforced bearing pads (Fig. 1d). Expansion joints are provided in the reinforced concrete deck slab every 3 to 4 spans (e.g. the expansion length varies from 120 m (394 ft) to 160 m (525 ft)). Over the piers, continuity is provided only in the deck slab.

## OBJECTIVE AND OUTLINE

The main objective of this paper is to show the implications of increased live loads on the design of precast bridge girders. For this purpose, reference is made to the British Standards design live loads[6,7] (BS 5400-2:2006 and BD 37/01). The outline of the paper is as follows:

- Identify the differences between the British Standards[6,7] and AASHTO LRFD[4] design live loads.
- Identify the differences between the British Standards[5] and AASHTO[4] service load criteria and load combinations.
- Present the original girder design for a project case study[8] according to the British Standards based on simple span units.
- Present the alternative cost-effective solution that was actually adopted (Fig. 1).
- Develop the girder design according to AASHTO LRFD specifications[4] and show the impact of using the British Standards on the design based on code-to-code comparison of results.
- Show the impact of increased live loads on precast girders made continuous for composite dead loads and live loads.



(a) Main Highway Bridge

(b) Utility Bridge

(c) Trestle Bridge

(d) Precast Girder Placement

Fig. 1 Khalifa Port Project – Bridge Structures
*(Courtesy of Archirodon Construction, UAE)*

## AASHTO[4] vs. BRITISH STANDARDS[6, 7] LIVE LOADS

The British Standards[6,7] design live loads are designated as HA and HB loads. Reference is made to AASHTO[4] LRFD HL93 live loads for comparisons.

The HA load consists of a uniformly distributed load $w$ (kN/m) = $336\times(1/L)^{0.67}$ ($w$ (kips/ft) = $50\times(1/L)^{0.67}$) for L < 50 m (164 ft)) and a Knife Edge Load (KEL) of 120 kN (27 kips) (that is a moving load). For the medium span length range (12 m (40 ft) ≤ L ≤ 48 m (160 ft)), $w$ varies from 63.5 kN/m (4.3 kips/ft) to 25.1 kN/m (1.7 kips/ft), larger than AASHTO[4] HL93 lane load of 9.3 kN/m (0.64 kips/ft).

The HB load consists of four 300 kN (67.5 kips) axles spaced at 1.8 m (6 ft) between the $1^{st}$ and $2^{nd}$ axle and the $3^{rd}$ and $4^{th}$ axle with a varying spacing of 6 m (20 ft) to 26 m (86 ft) between the middle $2^{nd}$ and $3^{rd}$ axles (Fig. 2a). The resulting gross truck weight (including impact) is 1200 kN (270 kips), larger than the AASHTO[4] HL93 design truck (Fig. 2b) of 325 kN (72 kips) multiplied by 1.33 for impact (equal to 435 kN (96 kips)). Note that the British Standards (BD 37/01) require increasing the axle weight of 300 kN (67.5 kips) for HB load to 450 kN (102 kips) (e.g. by 50%, resulting in a gross truck weight of 1800 kN (405 kips)) for motorways, trunk roads and principal road extensions of trunk routes.[7]

According to the British Standards[6,7], the HB load should be applied in combination with HA loadings (unless directed otherwise by the relevant authorities), that is, for multiple lanes, an HB truck is placed in one lane while the remaining lanes are loaded with the lighter HA load. If the number of design lanes is greater than or equal to four, the HA load in the third and subsequent lanes should be multiplied by a reduction factor of 0.6 (BS 5400 -2: 2006, Section 6.4, Table 14).

The increase in induced forces due to the British Standards[6,7] live loads was determined in a previous study[9] for the span length range of medium span bridges (12 m (40 ft) $\leq L <$ 48 m (160 ft)). It is designated by a factor $\beta$ as the ratio of the bending moments that develop due to HA and HB loads to the moments that develop due to AASHTO HL93 load. For HA loading alone, $\beta$ varies from 1.55 for $L$ = 12 m (40 ft) to 1.18 for $L$ = 48 m (160 ft). For HA + HB loadings, $\beta$ varies from 1.4 for $L$ = 12 m (40 ft) to 1.25 for $L$ = 48 m (160 ft) if the HB axle load is set at 300 kN (67.5 kips)[7] and $\beta$ varies from 2.0 for $L$ = 12 m (40 ft) to 1.8 for $L$ = 48 m (160 ft) if the heavy HB axle load of 450 kN (102 kips)[7] is specified.

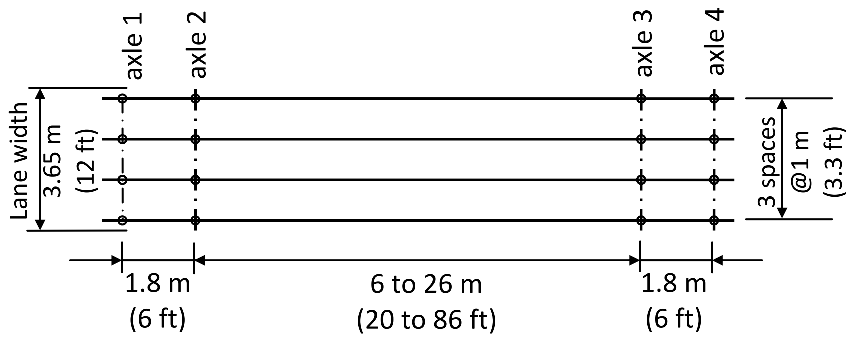
The differences in live loads between the British Standards[6,7] (HA and HB) and AASHTO LRFD[4] (HL93) prompted certain authorities[10] to increase the HL93 design live load by 50% when AASHTO LRFD[4] is adopted for design. It is also considered to more realistically represent nowadays complex truck weight and axle configurations.[11]

## AASHTO[4] vs. BRITISH STANDARDS[5, 6] SERVICE LOAD CRITERIA

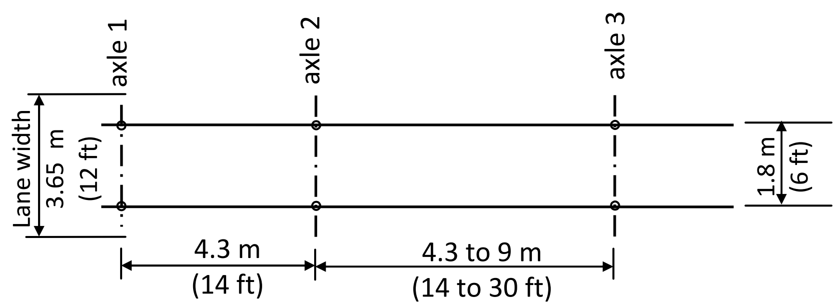
In prestressed concrete bridges, the precast girder size, spacing and prestressing strands distribution are determined based on the service load stress limits. The number of strands is usually controlled by the concrete compressive service stress in the initial stage (at release) $(\sigma_i)_C$ and the girder size and spacing are optimized based on the concrete tensile service stress $(\sigma_e)_T$ in the final stage. Comparisons between the British Standards[5,6] and AASHTO[4] service load stresses in the initial and final stage are discussed in the following sections and are also presented in Table 1 for clarity.

### CONCRETE STRENGTH

The service load stress limits are a function of the concrete 28-day strength. While AASHTO specifications are based on cylindrical strengths ($f'_c$), the British Standards utilize cube strengths $(f'_c)_{cu}$ that are approximately 20% larger than the cylindrical strengths.[12] Typical 28-day cylindrical strengths ($f'_c$) for normal weight concrete vary from 40 MPa (5800 psi) to 50 MPa (7200 psi) (that is 48 MPa (7000 psi) to 60 MPa (8400 psi) based on cube strength $(f'_c)_{cu}$). The conventional 41.7 MPa (6000 psi) cylindrical concrete strength (that corresponds to 50 MPa (7200 psi) cube strength) was specified in the project case study.




(a) HB Axle Load: 300 kN (67.5 kips) (Normal); 450 kN (102 kips) (Heavy)




(b) HL93 Truck Load: Axle 1 = 35 kN (8 kips); Axles 2 & 3 = 145 kN (32 kips) each

Fig. 2 Truck Loads Distribution for AASHTO[4] and BS 5400[6,7] loads

## SERVICE LOAD STRESSES AT RELEASE (INITIAL STAGE)

Based on the British Standards[5,6], the permissible compression stress at release $(\sigma_i)_C$ is $0.5(f'_{ci})_{cu}$ (BS 5400 – 4: 1990, section 6.3.2.4), where $(f'_{ci})_{cu}$ is the cube strength at release. According to AASHTO[4], it is $0.6f'_{ci}$ where $f'_{ci}$ is the concrete cylindrical strength at release. Based on the 1.2 conversion factor between cylindrical and cube strengths[12], the permissible compression stress at release is the same ($0.6f'_{ci}$). Note that the concrete strengths at release ($(f'_{ci})_{cu}$, $f'_{ci}$) are usually taken as 0.8 times the 28-day strength ($(f'_c)_{cu}$, $f'_c$).[2]

Based on the British Standards, the allowable tensile stress at release $(\sigma_i)_T$ is 1 MPa (145 psi) (BS 5400 – 4: 1990, section 6.3.2.4). Based on AASHTO[4], $(\sigma_i)_T$ (in MPa) is $0.25\sqrt{f'_{ci}} < 1.38$ MPa ($(\sigma_i)_T$ (in psi) = $3\sqrt{f'_{ci}} < 200$ psi) for non-compressed zones without bonded reinforcement (e.g. mid-span zones) and $0.63\sqrt{f'_{ci}}$ ($7.5\sqrt{f'_{ci}}$) for non-compressed zones with bonded reinforcement (e.g. end zones). For $f'_c$ = 41.7 MPa (6000 psi), $(\sigma_i)_T$ = $0.25\sqrt{0.8\times 41.7}$ = 1.44 MPa (209 psi) > 1.38 MPa (200 psi), use 1.38 MPa (200 psi) at mid-span and $(\sigma_i)_T = 0.63\sqrt{0.8\times 41.7}$ = 3.64 MPa (528 psi) in the end zones.

Although the British Standards tensile stress limit of 1 MPa (145 psi) is smaller than AASHTO limits of 3.64 MPa (528 psi) in the end zones and 1.38 MPa (200 psi) elsewhere, its impact on design in the initial stage is not significant as: (1) compression stresses usually control the mid-span zone in the initial stage and the limit of $0.6f'_{ci}$ is identical between AASHTO and the British Standards and 2) the induced end zone tensile stresses can be reduced by debonding prestressing strands and according to the British Standards, there are no limits on the number of strands that can be debonded (unlike AASHTO LRFD[4] that limits the number of debonded strands to 25% of the total number of strands).

SERVICE LOAD STRESSES IN THE FINAL STAGE

According to the British Standards[5], the allowable compression stress at the service limit state in the final stage $(\sigma_e)_C$ is $0.4(f'_c)_u$ (equal to $0.48f'_c$) (BS 5400 – 4: 1990 Section 6.3.2.2), smaller than the AASHTO[4] limit of $0.6f'_c$ for the governing load combination that comprises live loads (Table 1). This is not of particular concern as in simple span units the design is usually governed by the tensile stresses that develop at mid-span in the final stage.

According to the British Standards (BS 5400 – 4: 1990 Section 6.3.2.4), the allowable tensile stress $(\sigma_e)_T$ should not exceed 4.8 MPa (700 psi) for class 3 concrete (tensile stress permitted with limited design crack width), $0.45\sqrt{(f'_c)_u}$ in MPa ($5.4\sqrt{(f'_c)_u}$ in psi) for class 2 concrete (tensile stress permitted without visible cracking) and zero for class 1 concrete (no tension). Based on cylindrical strength, the class 2 concrete stress limit translates to $0.5\sqrt{f'_c}$ in MPa ($6\sqrt{(f'_c)}$ in psi) that is 3.2 MPa (464 psi) for $f'_c$ = 41.7 MPa (6000 psi). According to AASHTO[4], the tensile stress limit $(\sigma_e)_T$ in the final stage is a function of the environmental classification of the bridge[2,4,13], that is $(\sigma_e)_T = 0.5\sqrt{f'_c}$ (in MPa) ($6\sqrt{f'_c}$ (in psi)) for normal environments, $0.25\sqrt{f'_c}$ (in MPa) ($3\sqrt{f'_c}$ (in psi)) for aggressive environments and 0 for extremely aggressive environments. These allowable stresses are also presented in Table 1.

In practice, the concrete class 3 stress limit of 4.8 MPa (700 psi) in the British Standards is specified when all lanes are loaded with HB trucks (rare case) while the class 2 concrete stress limit of $0.5\sqrt{f'_c}$ in MPa ($6\sqrt{(f'_c)}$ in psi) applies when one lane is loaded with HB while the remaining lanes are loaded with HA loads (common case). The HB axle load is usually set at 300 kN (67.5 kips)[7] (the heavy axle load of 450 kN (102 kips) is used when directed by the relevant authority). These tensile stress limits set up the basis of comparisons between AASHTO[4] and the British Standards[5,6,7] design depending on the load combinations used.

Load Combinations

According to the British Standards[6] (BS 5400-2: 2006, Part 2, Specifications for Loads), two load combinations are usually considered for the superstructure girder design: load combination 1 that comprises dead load (DL), live load (LL) and prestress (PRE) and load combination 3 that includes differential shrinkage (SH) and temperature gradient (TG) in addition to the loads in load combination 1. According to AASHTO LRFD[4], a similar load combination (labeled as service 3) to the BS load combination 3 is usually specified to check the service load *tensile* stresses. These load combinations are listed in Eqs.1 - 3 as follows:

BS 5400-2: 2006 Load Combination 1: PRE + DL + 1.1LL (1)

BS 5400-2: 2006 Load Combination 3: PRE + DL + LL + SH + 0.8TG (2)

AASHTO[4] LRFD Service 3: PRE + DL + 0.8LL + SH + 0.1TG (3)

Although the British Standards load combination 1 (Eq.1) incorporates a factor of 1.1 for live load, the shrinkage and temperature gradient forces in load combination 3 (Eq.2) usually make it the governing case. Compared to AASHTO (Eq.3), the load factors are different (the live load factor in the British Standards is 1.0 compared to 0.8 in AASHTO and the temperature gradient factor in the British Standards is 0.8 compared to 0.1 in AASHTO). Note that the load factor of 0.1 for the temperature gradient (TG) in Eq.3 was determined by multiplying the factor of 0.5 that is specified in AASHTO LRFD[4] Table 3.4.1-1 by 0.2, a factor that is usually adopted for decks with asphalt subjected to a temperature gradient that causes *tension* in the bottom fiber according to AASHTO[4] Clause 3.12.3.

Based on comparisons of Eqs.2 and 3, the BS load combination (Eq.2) is obviously more critical as its live loads (HA + HB vs. AASHTO HL93) are heavier and its factors (1.0 for live load in Eq.2 compared to 0.8 in Eq.3 and 0.8 for the temperature gradient in Eq.2 compared to 0.1 in Eq.3) are larger. The impact of these differences is illustrated by considering a project case study[8] in the following section.

Table 1: Comparison of service load stresses based on AASHTO[4] and the British Standards[5]

| CRITERIA | British Standards[5] | AASHTO LRFD[4] |
|---|---|---|
| Release: Compression | $(\sigma_i)_C \leq 0.6f'_{ci}$<br>e.g. $(\sigma_i)_C \leq 0.5(f'_{ci})_{cu}$ for $f'_{ci} \approx 0.8(f'_{ci})_{cu}$ | $(\sigma_i)_C \leq 0.6f'_{ci}$ |
| Release: Tension | $(\sigma_i)_T \leq 1$ MPa (145 psi) | Non-compressed zones without bonded steel (mid-span):<br>$(\sigma_i)_T$ (MPa) $\leq 0.25\sqrt{f'_{ci}} < 1.38$ MPa<br>($(\sigma_i)_T$ (psi) $\leq 3\sqrt{f'_{ci}} < 200$ psi)<br><br>With bonded steel (end zones):<br>$(\sigma_i)_T$ (MPa) $\leq 0.63\sqrt{f'_{ci}}$<br>($(\sigma_i)_T$ (psi) $\leq 7.5\sqrt{f'_{ci}}$) |
| Final: Compression | $(\sigma_e)_C \leq 0.48f'_c$<br>e.g. $(\sigma_e)_C \leq 0.4(f'_c)_u$ for $f'_c \approx 0.8(f'_c)_{cu}$ | $(\sigma_e)_C \leq 0.6f'_c$ (with live load)<br><br>$(\sigma_e)_C \leq 0.45f'_c$ (without live load) |
| Final: Tension | Class 3 (limited design crack width):<br>$(\sigma_e)_T \leq 4.8$ MPa (700 psi).<br><br>Class 2 (without visible cracking):<br>$(\sigma_e)_T$ (MPa) $\leq 0.5\sqrt{f'_c}$ ($0.45\sqrt{(f'_c)_u}$)<br>($(\sigma_e)_T$ (psi) $\leq 6\sqrt{f'_c}$ ($5.4\sqrt{(f'_c)_u}$)<br><br>Class 1 (no tension): $(\sigma_e)_T \leq 0$ | Normal Environment:<br>$(\sigma_e)_T$ (MPa) $\leq 0.5\sqrt{f'_c}$<br>($(\sigma_e)_T$ (psi) $\leq 6\sqrt{f'_c}$)<br><br>Aggressive Environment:<br>$(\sigma_e)_T$ (MPa) $\leq 0.25\sqrt{f'_c}$<br>($(\sigma_e)_T$ (psi) $\leq 3\sqrt{f'_c}$)<br><br>Extremely Aggressive: $(\sigma_e)_T \leq 0$ |

## PROJECT CASE STUDY[8]

The project case study[8] comprises three bridge structures, a 28.9 m (94 ft – 9 $^{13}/_{16}$ in) wide main highway bridge (Fig. 1a), a 27.5 m (90 ft – 2 $^{11}/_{16}$ in) wide utility bridge (Fig. 1b) and a 12 m (39 ft – 4 $^{7}/_{16}$ in) wide trestle bridge (Fig. 1c). The bridges were designed according to the British Standards.[5,6,7] The trestle bridge was designed for HA live load while the main highway and utility bridges were designed for HA and HB loads considering the heavy axle load of 450 kN (102 kips) for the HB truck (it was specifically requested in this project since the bridge structures provide access to a harbor zone[8]). The permissible tensile stress was set at 3.2 MPa (464 psi) based on class 2 concrete and a 28-day cube strength $(f'_c)_{cu}$ of 50 MPa (7200 psi) (e.g. a cylindrical strength $f'_c$ = 41.7 MPa (6000 psi)).

The original precast, prestressed girder layout is first presented. The alternative cost-effective solution is then provided. The impact of using the British Standards is finally highlighted based on comparisons with the girder design according to AASHTO LRFD criteria[4] for live loads, service load permissible stresses and load combinations.

### ORIGINAL DESIGN

The original layout consisted of 14 - 2300 mm (7 ft - 6 $^{9}/_{16}$ in.) deep girders that were spaced at 2.05 m (6 ft - 8 $^{11}/_{16}$ in.) for the main highway bridge (Fig. 3a) and at 1.98 m (6 ft - 6 in.) for the utility bridge (Fig. 3b). The 2300 mm (7 ft - 6 $^{9}/_{16}$ in.) deep girders were also used for the trestle bridge (spaced at 2.56 m (8 ft – 4 $^{3}/_{8}$ in.), a total of five girders per span, Fig. 3c). The girder dimensions are shown in Fig. 4a.

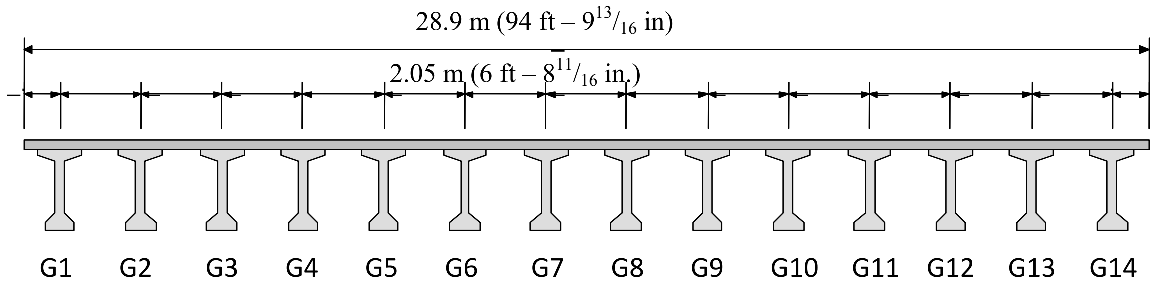
### ALTERNATIVE DESIGN

Based on value engineering, a smaller girder section was used (Fig. 4b). It consisted of a 2200 mm (7 ft - 2 $^{5}/_{8}$ in.) deep girder with a bottom flange width of 800 mm (2 ft – 7 $^{1}/_{2}$ in.), a top flange width of 1070 mm (3 ft - 6 $^{1}/_{8}$ in.) and a web thickness of 200 mm (8 in.). The girder layout (number of girders and spacing) was set as per the original design (Fig. 3).

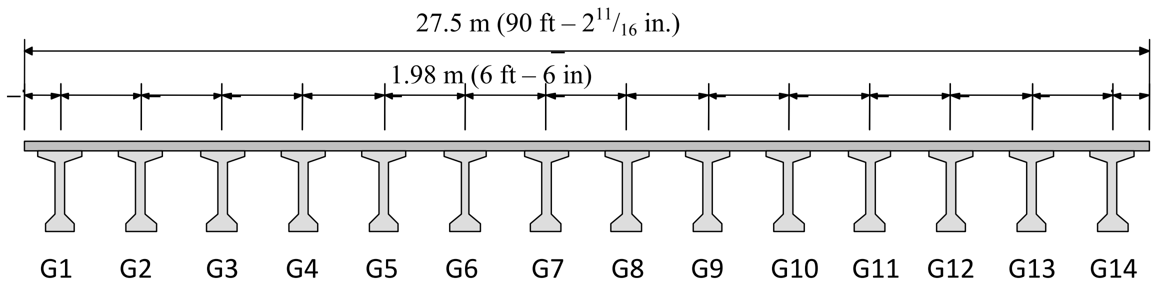
The basis for altering the original girder cross-section was as follows:

- Reduce the web thickness from 250 mm (10 in.) to 200 mm (8 in.) as no strands were provided in the web, and stirrups and ordinary reinforcement were fit-in the web based on the 50 mm (2 in.) specified clear cover. Therefore, the extra web thickness of 50 mm (2 in.) only increased the self-weight of the girder (by 13% compared to the proposed girder) without noticeable contribution to its stiffness.

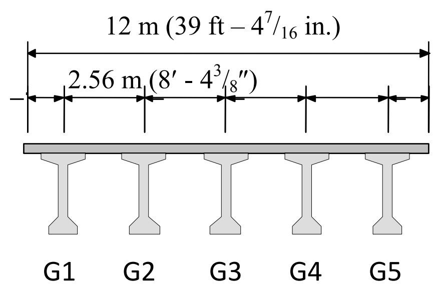
- Increase the top flange width (compression flange for simply supported girders) from 800 mm (2 ft – 7 $^{1}/_{2}$ in.) to 1070 mm (3 ft - 6 $^{1}/_{8}$ in.) which enhanced the stability of the girder (especially that no intermediate concrete diaphragms were specified in the project). The weak axis moment of inertia of 0.0306 $m^4$ (73,517 $in^4$) for the proposed girder was larger compared to 0.0265 $m^4$ (63,667 $in^4$) for the original girder.




(a) Main Highway Bridge




(b) Utility Bridge




(c) Trestle Bridge

Fig. 3 Bridge cross-sections

## COMPARISONS

Comparisons of section properties and service load stresses based on the alternative and original girder sections are presented for a typical interior girder of the main highway bridge

(Fig. 3a) (that is the governing case since its girder spacing of 2.05 m (6 ft - $8^{11}/_{16}$ in.) (Fig. 3a) is larger compared to 1.98 m (6 ft – 6 in.) (Fig. 3b) for the utility bridge and its design live loads (5 lanes of HA + 1 lane of HB) are larger compared to the trestle bridge live loads (3 lanes of HA load)). Note that 48 - 15.2 mm (0.6 in.) diameter strands were used for the alternative girder and 52 strands for the original girder.

The section properties for the original and proposed girders (composite and non-composite) are presented in Table 2. The composite section properties are based on a 225 mm (9 in.) thick cast-in-situ reinforced concrete slab that has the same 28-day concrete cylindrical strength as the precast girder ($f'_c$ = 41.7 MPa (6000 psi)) and a girder spacing of 2.05 m (6 ft - $8^{11}/_{16}$ in.) for the main highway bridge (Fig. 3a).

The benefits of using the section properties of the 2200 mm (7 ft - $2^5/_8$ in.) deep alternative girder compared to the original 2300 mm (7 ft - $6^9/_{16}$ in.) deep girder are illustrated by referring to the service load stress (Eq. 4 below). Eq. 4 was developed for the British Standards[6] load combination 3 (Eq.2).

$$-F\times\left(\frac{1}{\underbrace{A_{nc}}_{\text{reduced by 13\%}}}+\frac{\overbrace{e}^{\text{increased by 4.5\%}}}{(S_b)_{nc}}\right)+\frac{\overbrace{M_{SW}}^{\text{reduced by 13\%}}}{\underbrace{(S_b)_{nc}}_{\text{reduced by 14\%}}}+\frac{M_{ncdl}}{(S_b)_{nc}}+\frac{M_{cdl}}{(S_b)_c}+\frac{M_{LL}}{\underbrace{(S_b)_c}_{\text{reduced by 19\%}}}+\frac{M_{SH}+0.8M_T}{(S_b)_c}\leq(\sigma_e)_T \qquad (3)$$

(-) denotes compression; (+) denotes tension

where $M_{sw}$ is the self-weight moment, $M_{ncdl}$ is the non-composite dead load moments, $M_{cdl}$ is the composite dead load moment, $M_{LL}$ is the live load moment (5HA + HB loads), $M_{SH}$ is the moment due to differential shrinkage and $M_T$ is the moment due to the temperature gradient.

From Eq.4, the following may be concluded (section properties are listed in Table 2):

1. The smaller girder cross-sectional area ($A_{nc}$) by 13% increases the compression axial stress due to prestress ($-F/A_{nc}$) and reduces the self-weight bending moment tensile stress ($M_{SW}/(S_b)_{nc}$). This allows reducing the number of prestressing strands (48 strands were used in the alternative girder compared to 52 strands in the original girder).

2. The increased top flange width shifts the neutral axis of the girder upward (e.g. the distance from the neutral axis to the bottom flange fiber increased). This reduces the non-composite bottom section modulus $(S_{bot})_{nc}$ by 14% and increases the prestressing

strands eccentricity (e) by 4.5%, therefore improving the compression stress contribution in the pre-compressed tensile zones of the girder ($-F \times e/(S_b)_{nc}$).

3. The reduced composite section modulus $(S_{bot})_c$ by 19% increases the bottom fiber tension stresses due to composite dead loads and live loads. These are compensated by the increase in the compression stress due to prestress (1 and 2 above).

The implications of reducing the girder size on induced stresses are illustrated by considering actual stress values as shown in the following section.

Numerical Values for Induced Stresses

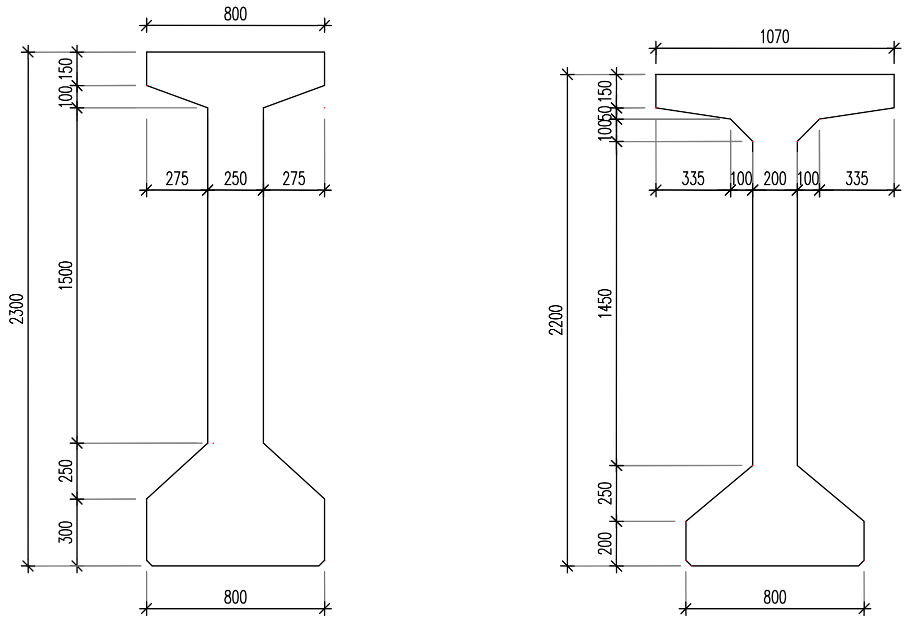
Numerical values for the induced stresses in the original and alternative girders are presented in Table 3. They are based on 40 m (131.2 ft) long simple span units, precast girder lengths of 39.6 m (129 ft - $11^1/_{16}$ in.) and design span lengths of 38.8 m (127 ft – $3^9/_{16}$ in.).

Design loads consist of non-composite and composite dead loads, live loads, differential shrinkage and temperature gradient. Non-composite dead loads include the self-weight of the girder and the 22.5 cm (9 in.) thick cast-in-situ deck (unit weight of concrete equal to 25 $kN/m^3$ (150 pcf)). Composite dead loads include a 10 cm (4 in.) wearing surface (2.25 $kN/m^2$ (50 psf)), a 5 cm (2 in.) future wearing surface (1.2 $kN/m^2$ (25 psf)), two edge barriers at 10.5 kN/m (720 plf) each, a median barrier of 8.2 kN/m (560 plf), two edge sidewalks at 7.5 $kN/m^2$ (155 psf) each and a utility load of 0.75 $kN/m^2$ (16 psf). Differential shrinkage stresses due to casting the slab after casting the girder are based on 60-day elapsed time. The temperature gradient is taken from the British Standards[7] (BD 37/01) based on 8.4°C (47 F) temperature fall in the top fiber of the top slab, 6.5°C (44 F) fall in the bottom fiber of girder and 0°C (32 F) fall near the top fiber of the girder (somehow similar to AASHTO[4]).

Live loads are based on 6 lanes (5HA + HB) where five lanes are loaded with HA while only one lane is loaded with HB. The heavy axle load of 450 kN (102 kips) was specified for HB instead of the normal load of 300 kN (67.5 kips) as per the project requirements.[8]

The total prestress losses were 17.5% for the alternative girder and 18.5% for the original girder (the elastic shortening losses in the original beam were larger since more prestressing strands were used). Based on a jacking force of 75% of the ultimate force, the effective prestress force (after total losses) was 7800 kN (1755 kips) for the alternative girder and 8275 kN (1860 kips) for the original girder.

From Table 3, it can be seen that the reduced girder area and self-weight (by 13%) and number of strands (from 52 to 48) of the alternative girder compared to the original girder resulted in final stresses in the bottom fiber that were almost equal (the difference is only 430 $kN/m^2$ (63 psi)). Therefore, the proposed cost-effective girder was adopted for construction. Note that the final tensile stresses of 2250 $kN/m^2$ (326 psi) in the original girder and 2670 $kN/m^2$ (387 psi) in the proposed girder are both smaller than the 3200 $kN/m^2$ (464 psi) stress limit for class 2 concrete (BS 5400 – 4: 1990 Section 6.3.2.4).




(a) Original Section (not used) (b) Alternative Section (recommended)

Fig. 4 Girder Cross-Section

## DESIGN BASED ON AASHTO LRFD[4]

The impact of using the British Standards on the design of the precast, prestressed girders of the project case study is determined by comparing results based on AASHTO[4] LRFD design. The moment due to HL93 live load plus impact is obtained using the β factor from the Section entitled "AASHTO vs. British Standards Live Loads" (β = 1.9 for L = 38.8 m (127 ft – $3^{9}/_{16}$ in.) and HB truck axle load of 450 kN (102 kips)[7] by interpolation). The resulting moment due to AASHTO HL93 live load is then calculated as (5608 kN-m / 1.9) = 2952 kN-m (2177 ft-kips). Note that these numbers were confirmed by software application.[14]

Based on the reduced live load moment due to AASHTO HL93 live load, the girder spacing can be increased and/or the girder size can be reduced depending on the environmental classification of the bridge. For comparisons, results in this paper are presented based on the specific design requirements of authorities in United Arab Emirates[10] that are to increase the AASHTO HL93 design live load by 50% and to set the permissible service load tension stress at zero (e.g. no tension). The moment due to HL93 live load plus impact × 1.5 is accordingly calculated as 2952 kN-m × 1.5 = 4428 kN-m (3266 ft-kips), that is 21% smaller than the calculated live load moment of 5608 kN-m (4136 ft-kips) due to 5HA + HB loads.

Based on AASHTO[4] service 3 load combination (Eq.3) and zero permissible service load tensile stress, the girder spacing can be increased from 2.05 m (6 ft - $8^{11}/_{16}$ in.) to 2.5 m (8 ft

– $2^7/_{16}$ in.) for the main highway bridge (e.g. 12 girders per span instead of 14), from 1.98 m (6 ft – 6 in.) to 2.4 m (7 ft – $10^1/_2$″ in.) for the utility bridge (e.g. 12 girders per span instead of 14) and from 2.56 m (8 ft - $4^3/_8$ in.) to 2.8 m (9 ft - $2^1/_4$ in.) for the trestle bridge (the number of girders was kept at five due to limitations on the overhang dimensions). This would reduce the number of girders by 100 (from a total of 905 girders based on the British Standards to 805 girders based on AASHTO[4] and the relevant authority design criteria[10]).

It should be noted that if the lighter HB axle load of 300 kN (67.5 kips) was used instead of 450 kN (102 kips)[7], the induced live load moment due to 5HA + HB would be 3982 kN-m (2937 ft-kips), 10% smaller than the moment of 4428 kN-m (3266 ft-kips) due to AASHTO HL 93 × 1.5. The precast girder configuration (number and spacing) based on the British Standards would then be almost identical to the girder configuration based on the modified AASHTO LRFD criteria.[10] This indicates that the concept of increasing the AASHTO HL 93 live load by 1.5 and setting the service load tensile stress limit at zero copes better with the British Standards if the HB truck axle load of 300 kN (67.5 kips) is adopted[7] and the class 2 concrete stress limit of 3.2 MPa (464 psi) is specified.

Table 2. Non-composite section properties (original and alternative precast girders)

| ITEM | Original | Proposed | COMMENTS |
|---|---|---|---|
| **Area:** | | | |
| Non-Composite | $A_{nc} = 0.919\ m^2$ (1432 $in^2$) | $A_{nc} = 0.799\ m^2$ (1245 $in^2$) | The proposed girder is 13% lighter |
| Composite | $A_c = 1.38 m^2$ (2150 $in^2$) | $A_c = 1.26 m^2$ (1953 $in^2$) | |
| **Moment of Inertia:** | | | |
| Non-Composite | $(I_{cg})_{nc} = 0.57\ m^4$ (1,369,671 $in^4$) | $(I_{cg})_{nc} = 0.514\ m^4$ (1,234,890 $in^4$) | The proposed girder moment of inertia is 10% smaller. |
| Composite | $(I_{cg})_c = 1.152\ m^4$ (2,767,211 $in^4$) | $(I_{cg})_c = 0.956\ m^4$ (2,296,799 $in^4$) | The proposed girder moment of inertia is 17% smaller. |
| **Offset of extreme fiber in tension to neutral axis:** | | | |
| Non-Composite | $(Y_{bot})_{nc} = 1.039$ m (40.9 in.) | $(Y_{bot})_{nc} = 1.086$ m (42.8 in.) | |
| Composite | $(Y_{bot})_c = 1.5$ m (59.0 in.) | $(Y_{bot})_c = 1.535$ m (60.4 in.) | |
| **Bottom fiber section modulus:** | | | |
| Non-Composite | $(S_{bot})_{nc} = 0.549\ m^3$ (33,502 $in^3$) | $(S_{bot})_{nc} = 0.473 m^3$ (28,864 $in^4$) | The proposed girder section modulus is 14% smaller. |
| Composite (based on 225 mm (9 in.)) thick slab | $(S_{bot})_c = 0.768\ m^3$ (46,866 $in^3$) | $(S_{bot})_c = 0.623\ m^3$ (38,006 $in^3$) | The proposed girder section modulus is 19% smaller |

Table 3. Comparisons of stresses at midspan (interior girder)

| ITEM | Original Girder | Proposed Girder |
|---|---|---|
| Bending Moments | $M_{sw}$ = 4309 kN-m (3199 ft-kips)<br>$M_{ncdl}$ = 2524 kN-m (1862 ft-kips)<br>$M_{cdl}$ = 2278 kN-m (1680 ft-kips)<br>$M_{LL}$ = 5608 kN-m (4164 ft-kips) | $M_{sw}$ = 3747 kN-m (2764 ft-kips)<br>$M_{ncdl}$ = 2524 kN-m (1862 ft-kips)<br>$M_{cdl}$ = 2278 kN-m (1680 ft-kips)<br>$M_{LL}$ = 5608 kN-m (4164 ft-kips) |
| # 15.2 mm (0.6 in.) dia. strands: $f_{pu}$ = 1860 MPa (270 ksi) | 52 | 48 |
| Stress: Prestress (PRE)<br>$\sigma_{bot} = \frac{-F_e}{A_{NC}} - \frac{F_e \times e}{(S_{bot})_{NC}}$ | $\sigma_{bot} = \frac{-8275}{0.919} - \frac{8275 \times .89}{0.549}$<br>= -22,420 kN/m$^2$ (-3.25 ksi) | $\sigma_{bot} = \frac{-7800}{0.799} - \frac{7800 \times .94}{0.473}$<br>= -25,263 kN/m$^2$ (-3.66 ksi) |
| Stress: selfweight (SW)<br>$\sigma_{bot} = \frac{M_{SW}}{(S_{bot})_{NC}}$ | $\sigma_{bot} = \frac{4309}{0.549} =$<br>7,849 kN/m$^2$ (1.14 ksi) | $\sigma_{bot} = \frac{3747}{0.473} =$<br>7,922 kN/m$^2$ (1.15 ksi) |
| Stress: NC loads (NCD)<br>$\sigma_{bot} = \frac{M_{NCDL}}{(S_{bot})_{NC}}$ | $\sigma_{bot} = \frac{2524}{0.549} =$<br>4,598 kN/m$^2$ (0.67 ksi) | $\sigma_{bot} = \frac{2524}{0.473} =$<br>5,336 kN/m$^2$ (0.77 ksi) |
| Stress: Composite (CDL)<br>$\sigma_{bot} = \frac{M_{CDL}}{(S_{bot})_{C}}$ | $\sigma_{bot} = \frac{2278}{0.768} =$<br>2,966 kN/m$^2$ (0.43 ksi) | $\sigma_{bot} = \frac{2278}{0.623}$<br>= 3,657 kN/m$^2$ (0.53 ksi) |
| Stress: Live load (LL)<br>$\sigma_{bot} = \frac{M_{LL}}{(S_{bot})_{C}}$ | $\sigma_{bot} = \frac{5608}{0.768} =$<br>7,302 kN/m$^2$ (1.06 ksi) | $\sigma_{bot} = \frac{5608}{0.623} =$<br>8,989 kN/m$^2$ (1.3 ksi) |
| Stress:<br>Differential Shrinkage (SH)<br>Temperature Gradient (T) | 275 kN/m$^2$ (0.04 ksi)<br>2100 kN/m$^2$ (305 ksi) | 269 kN/m$^2$ (0.039 ksi)<br>2200 kN/m$^2$ (319 ksi) |
| Total Stress:<br>PRE + DL + LL + SH +0.8T | 2250 kN/m$^2$ (0.326 ksi) | 2680 kN/m$^2$ (0.389 ksi) |

## EFFECT OF THE MAKING THE PRECAST GIRDERS CONTINUOUS

The concept of making the precast, prestressed girders continuous for composite dead loads and live loads[15] was investigated. Its main advantage is that it reduces the positive moments and induced stresses due to superimposed dead loads and live loads near mid-span. Contrarily, it increases induced stresses in the end zones of the precast girder resulting from negative moments especially if the larger British Standards[6,7] live loads are used (stresses that develop in the bottom compression fiber due to superimposed dead loads and live loads add to the compression stress due to the axial prestress force ($-F/A_{nc}$) and stresses in the top tension fiber add to the tensile stress due to the eccentric prestress force ($F \times e/(S_t)_{nc}$)).

According to AASHTO[4], the tensile stress check may be waived in the end zones that are designed as reinforced concrete members at the strength limit state. However, potential cracking of the solid end diaphragms in the non-compressed end zone area sometimes necessitate performing this stress check[10] which makes it the governing case for design instead of mid-span. According to AASHTO, the end zone allowable tensile stress is $0.63\sqrt{f'_c}$ (in MPa) ($7.5\sqrt{f'_c}$ (in psi)) (that is the rupture stress), e.g. 4 MPa (585 psi) for $f'_c$ = 41.7 MPa (6000 psi). According to the British Standards, it is 3.2 MPa (464 psi) based on class 2 concrete (as for mid-span).

The overstress in the end zones can be reduced by debonding strands. AASHTO[4] limits the number of strands that can be debonded to 25% of the total number of strands. The British Standards do not impose limits. In the project case study, the tensile stresses in the end zones would still exceed the limit of 3.2 MPa (464 psi) even if all strands were debonded. Therefore this option was not used. Detailed discussions on this topic may be found elsewhere.[9]

## CONCLUSIONS

This paper shows the implications of using the British Standards[5,6,7] on the design of precast, prestressed girder bridges compared to AASHTO LRFD[4]. Based on comparisons between live loads and service load tensile stress limits, the following conclusions were drawn:

- The British Standards live loads (designated as HA for lane load and HB for truck load) are larger than AASHTO HL93 live loads. The induced bending moments due to HA + HB live loads can be double the moments due to HL93 live loads if the HB truck axle load of 450 kN (102 kips) is used and 40% larger if the HB truck axle load of 300 kN (67.5 kips) is used.

- The load combination for the service load stress check *in tension* is more stringent in the British Standards as the factor for the temperature gradient is 0.8 compared to 0.1 in AASHTO LRFD and the factor for live load is 1.0 compared to 0.8 in AASHTO.

- The permissible service load tensile stress in the British Standards is commonly set at $0.5\sqrt{f'_c}$ in MPa ($6\sqrt{(f'_c)}$ in psi) for class 2 concrete (tensile stress permitted without visible cracking). This compares with the case of normal environment in AASHTO, but is larger than the permissible value of $0.25\sqrt{f'_c}$ in MPa ($3\sqrt{(f'_c)}$ in psi) for aggressive and zero for extremely aggressive environments.

From these findings and based on a project case study, it was concluded that designs based on the British Standards HA and HB live loads with the HB axle load set at 300 kN (67.5 kips)) and a service load tensile stress limit of $0.5\sqrt{f'_c}$ in MPa ($6\sqrt{(f'_c)}$ in psi) compare with AASHTO LRFD[4] designs if the AASHTO HL93 live load is increased by 50% and the service load tension stress limit is set at zero. This justifies the requirements of authorities[10] in countries were both codes are used to specify these limits in the project design criteria when AASHTO LRFD is adopted. It was also shown that if the British Standards heavy HB axle load of 450 kN (102 kips) was used, the required number of precast, prestressed girders would be larger.

The concept of making the girders continuous for composite loads and live loads was briefly presented and the effects of overstress in the end zones of the precast girders due to increased live loads were discussed. It is worth noting that the information provided in this paper pave 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.[16]

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## REFERENCES

1. Khaleghi B. (2005). Use of Precast Members for Accelerated Bridge Construction in Washington DC. *6th International Bridge Conference*. Journal of the Transportation Research Board, CD – 11S, Transportation Research Board of the National Academies, Washington, DC, pp. 187 – 196.
2. PCI Bridge Design Manual (2003). Precast/Prestressed Concrete Institute: Chicago, IL.
3. AASHTO (2007a). "Standard Specifications for Highway Bridges", 17th Edition, Washington, DC.
4. AASHTO (2010). "LRFD Bridge Design Specifications*", published by the American Association of State Highway & Transportation Officials*, 5th Edition, Washington, DC.
5. British Standards BS 5400-4:1990. Steel, Concrete and Composite Bridges – Part 4: Code of Practice for Design of Concrete Bridges.
6. British Standards BS 5400-2:2006. Steel, Concrete and Composite Bridges – Part 2: Specification for loads.
7. BD37/01, "Loads for Highway Bridges," Design Manual for Roads and Bridges, V. 1, Section 3, Part 14, Department of Transport, Highway and Traffic, UK Highways Agency, Aug. 2001, 123 pp.
8. Karapiperis D., Lykidis G., Savvopoulos K., El-Sayed K., Loukakis K. (2010). Combining effort. ASCE Civil Engineering Magazine, July 2010.
9. Gergess A. and Sen R. (2011). "Design of Precast Bridge Girders Made Continuous under Increased Live Loads". Paper under completion.
10. Abu Dhabi Municipality (2008). Structural Design Requirements for Bridges. Abu Dhabi Municipality Regulations. Website: www.adm.gov.ae

11. Sivakumar B., Sheikh Ibrahim F. (2007) "Enhancement of Bridge Live Loads using Weigh-in-Motion Data", Bridge Structures, Vol. 3, Numbers 3-4, September – December, Taylor & Francis.
12. ASTM C 39. 1996. Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. ASTM Standards. American Society for Testing and Materials.
13. FDOT (2010). Florida Department of Transportation Structural Design Guidelines. FDOT, Structural Design Office, Tallahassee, Florida.
14. LEAP CONSPAN (2009), precast/prestressed bridge beam analysis, Bentley.
15. Miller, R.A., Castrodale, R., Mirmiran, A., and Hastak, M. (2004) "NCHRP Report 519 - Connection of Simple-Span Precast Girders for Continuity", Transportation Research Board, Washington, D.C.
16. Gerges N. and Gergess A. (2011). "Implication of Increased Live Loads on the Design of Precast Bridge Girders". PCI journal. Paper in press.