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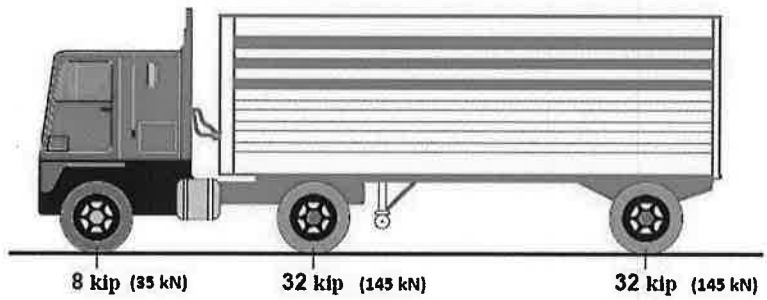
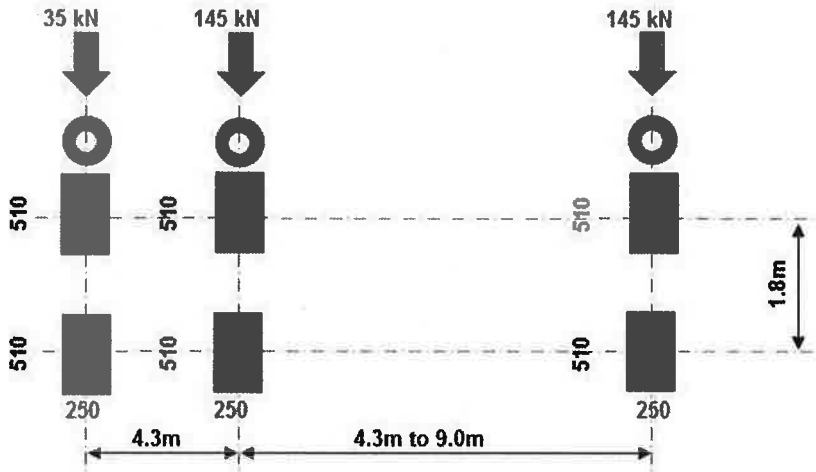
1- Design Truck (anteriormente HS20-44 Truck):

El Design Truck consta de tres ejes, delantero y dos ejes traseros con eje delantero con un peso 8kip (35 kN) y dos ejes traseros con un peso de 32kip (145 kN). La distancia entre los ejes delantero y trasero es de 14' (4,3 m) y la de los dos ejes traseros se puede variar entre el 14' (4,3 m) a 30 '(9,0m) para obtener el peor esfuerzo de cálculo. La distancia de neumático a neumático en todos los ejes es de 6 pies (1,8 m).





AASHTO HL-93 Truck Tire Plan

2- Design Tandem:

HL-93 Design Tandem constan de ejes dobles espaciados 4 '(1,2 m) de distancia, el peso de cada eje es 25kip (110 kN). La distancia entre los neumáticos de un eje es de 6 pies (1,8 m).

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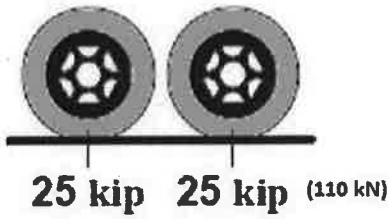
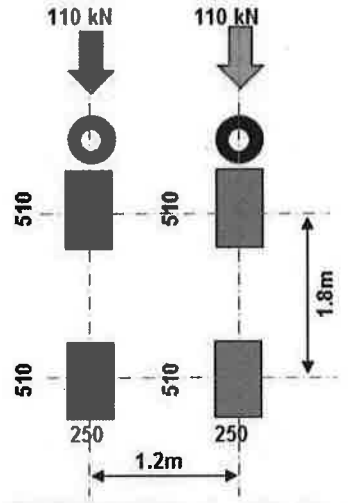


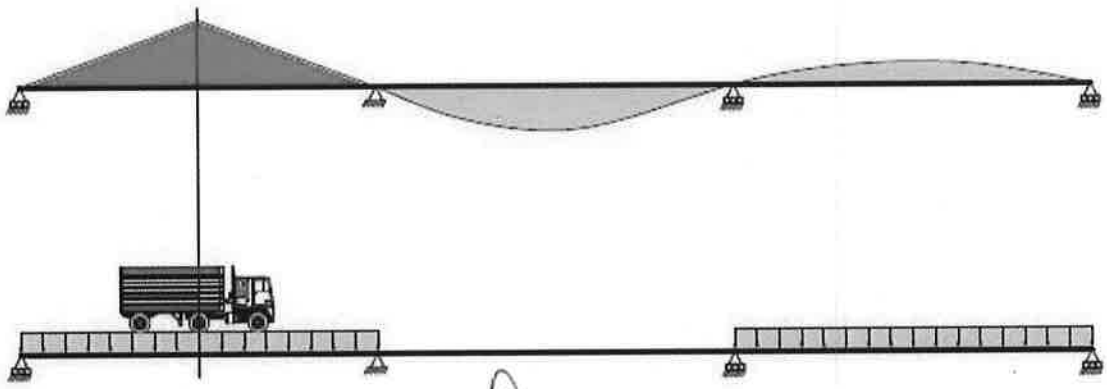


AASHTO HL-93 Tandem Tire Plan

3- Design Lane Load (LL):

El design lane load consistirá en una carga de 0,64 klf (9,34 kN/m) distribuidos de manera uniforme en la dirección longitudinal. En dirección transversal, se supondrá que la carga LL sea distribuida uniformemente sobre un ancho de 10,0' (3m).

El diseño de los vehículos y cargas de LL deben aplicarse de tal manera que se obtiene un efecto de fuerza extrema para el cálculo. En caso de múltiples carriles, se considerarán factores múltiples carriles descritos en el código.



Factor de impacto o de la asignación de carga dinámica se aplica sólo para los camiones y tándems, no para la carga de carril. La fuerza máxima de cálculo se toma como el mayor de los siguientes:

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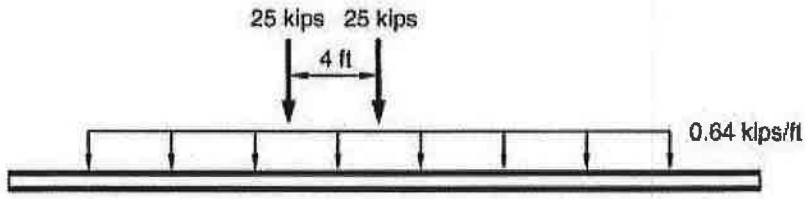
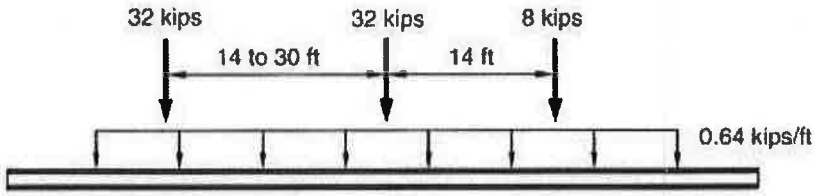
1. El efecto del Design Tandem + Design Lane Load;



2. El efecto del Design Truck (HL93) + el efecto de Design Lane Load;



7) Combinaciones de carga

Las combinaciones fueron calculadas utilizando el software *Autodesk® Structural Bridge Design 2019,* que se encarga de verificar las peores situaciones y proporciona el resultado de estas cargas.

Como se calcularon sólo las vigas, se utilizó nada más que las combinaciones con cargas permanentes y móviles.

Las combinaciones de los efectos de fuerzas extremas factoradas se especifican para este proyecto en los siguientes estados límites:

- **RESISTENCIA** I (Strength I): La combinación de carga básica relacionado con el uso de vehículos normales en el puente sin viento.

- **SERVICIO I** (Service I): Combinación de carga relacionada con el uso normal del puente con vientos de 88 kph, y con todas las cargas tomando sus valores nominales. También relacionado con el control de deflexión en estructuras metálicas y el control de ancho fisuras en estructuras de concreto reforzado.

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- **FATIGA** (Fatigue I): La combinación de cargas de fatiga y fractura está relacionada con la carga viva repetitiva vehicular y la respuesta dinámica debido a un camión de diseño simple.

Combinaciones y Factores de Carga

COMBINACIÓN DE CARGA / ESTADO LIMITE	CARGAS PERMANENTES	HL-93 (LANE LOAD, DESIGN TRUCK, DESIGN TANDEM), CARGA PEATONAL	ORTHO DECK
RESISTENCIA I	1,25	1,75	-
RESISTENCIA II	1,25	1,35	-
RESISTENCIA III	1,25	-	-
SERVICIO I	1,00	1,00	-
SERVICIO II	1,00	1,30	-
SERVICIO III	1,00	0,80	-
FATIGA	-	-	2,25

Carga de impacto (IM)

Los efectos estáticos del camión de diseño o tandem, las fuerzas centrífugas, y de frenado deben ser incrementadas en un porcentaje especificado en la tabla siguiente para los factores de impacto.

El factor que es aplicado a la carga estática debe ser tomado como:

(1 + IM /100)

La carga de impacto no puede ser aplicada a cargas peatonales o a la carga del carril de diseño.

Factores de impacto, IM

Componente	IM
Nudos de la losa -Todos los estados límites	75%
Todos los otros componentes	15%
Estado Límite de Fatiga y Fractura	33%
Todos los otros Estados Límites	

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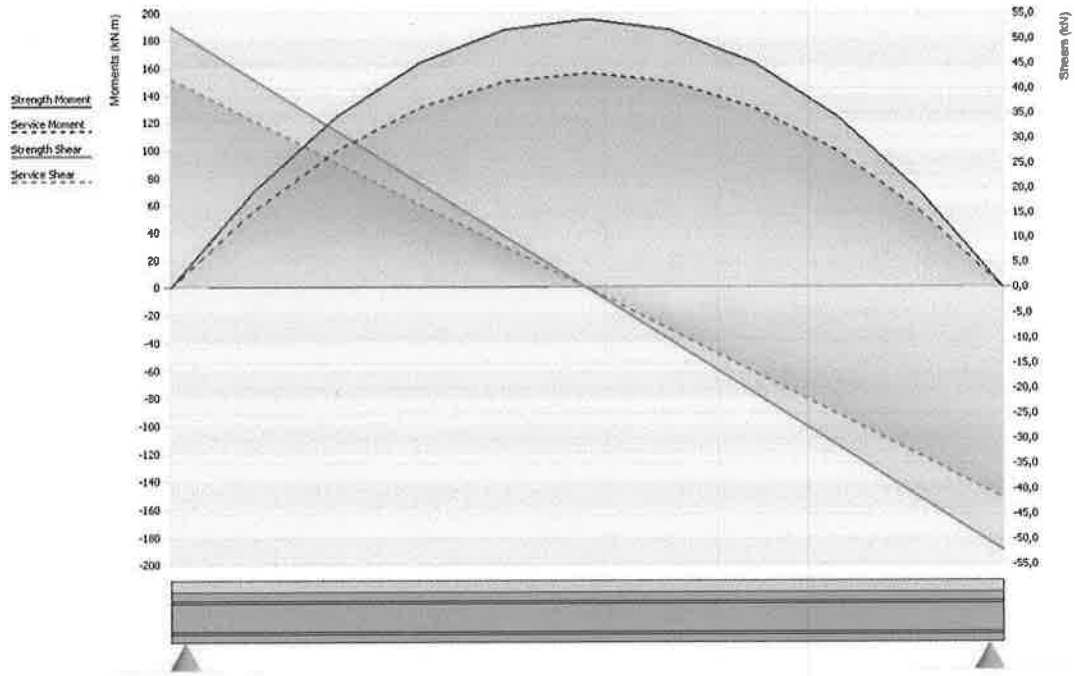
8) Esfuerzos característicos

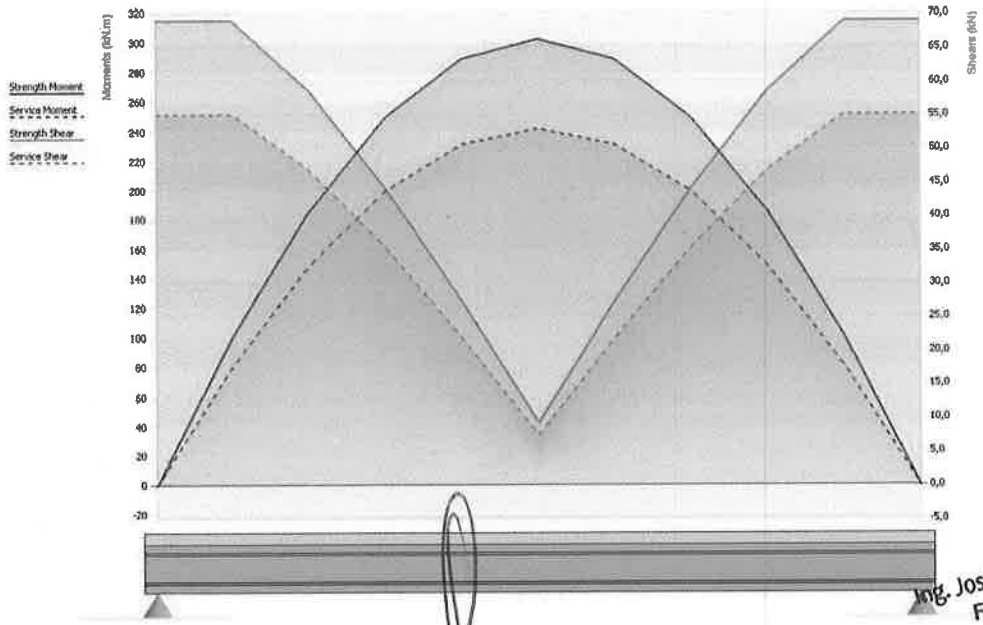
A partir del modelo descrito se encontraron los siguientes esfuerzos para las vigas del tablero de acuerdo con las cargas y combinaciones analizadas.

Esfuerzos en las vigas pretensadas

Esfuerzos debidos al izaje de la viga



Esfuerzos debidos a las cargas muertas



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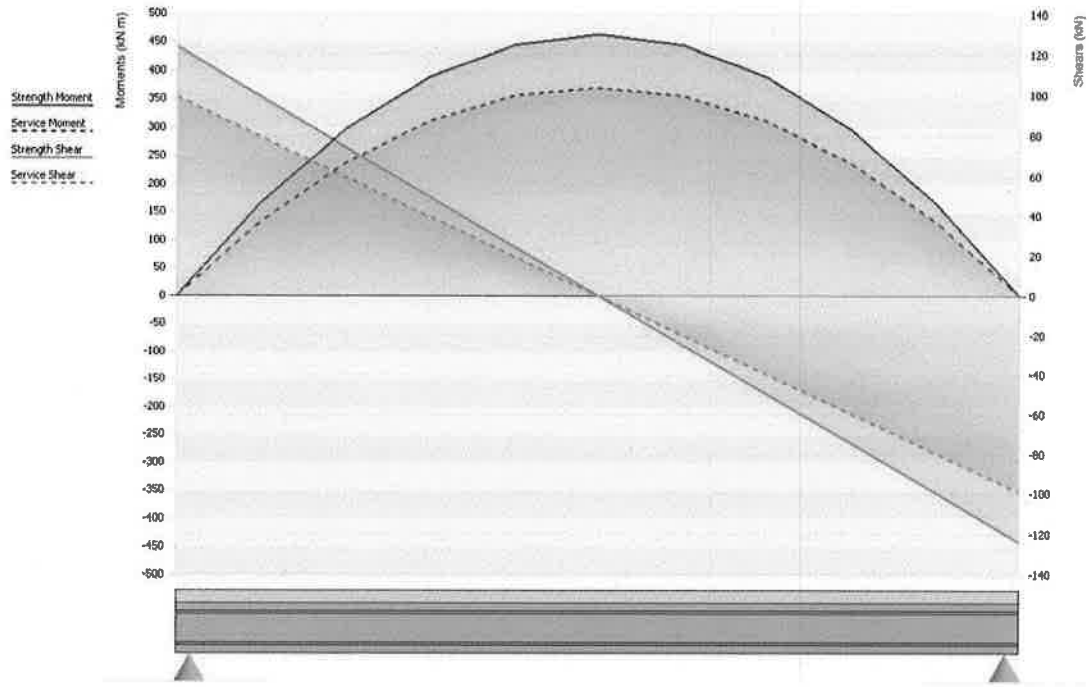
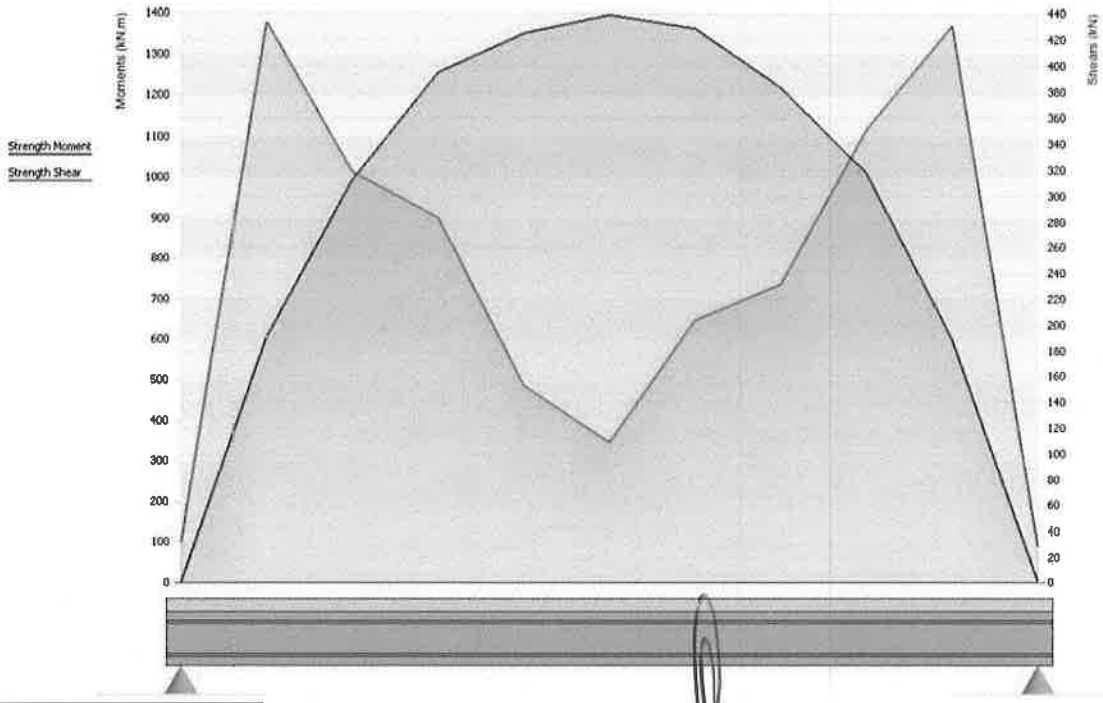
Esfuerzos debidos a etapa de construcción 1A



Envolvente de Momentos My+ en estado de límite de resistencia I (kN.m)



$My_{máx}$=1398kN.m

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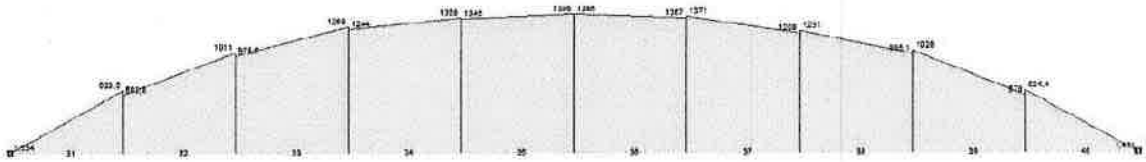
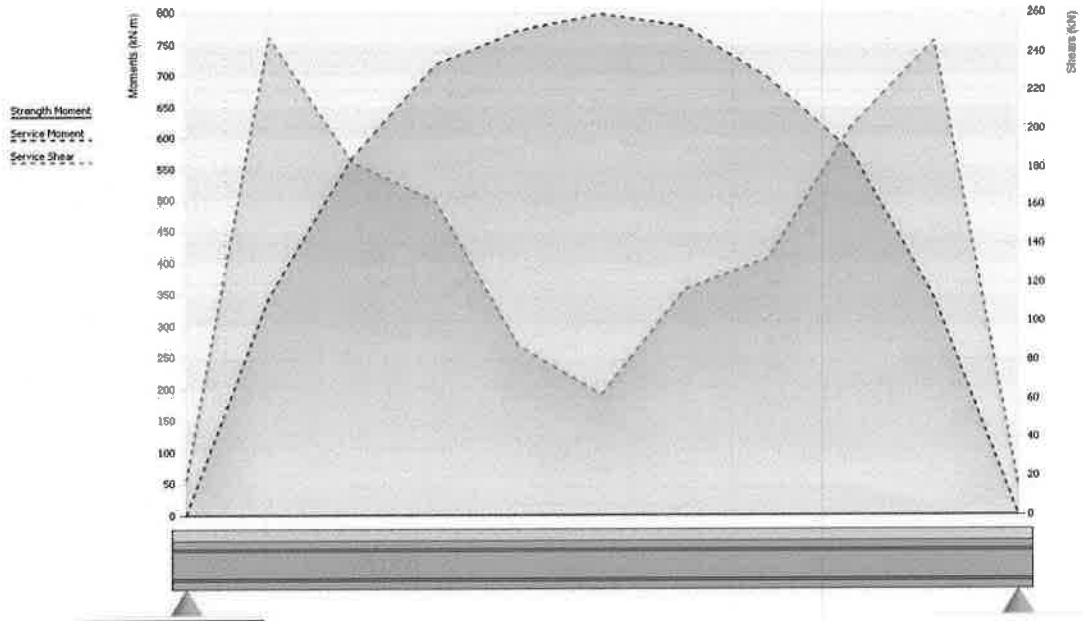
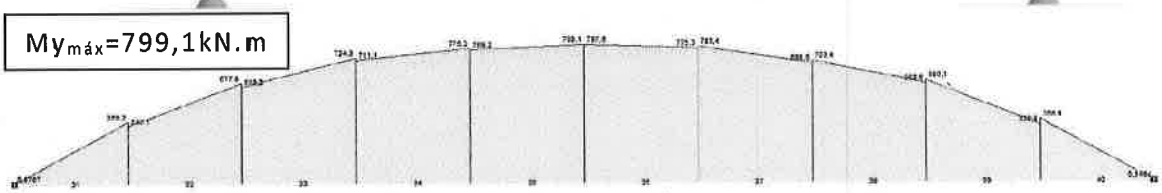


Envolvente de Momentos My+ en estado de límite de servicio I (kN.m)



$My_{máx}$=799,1kN.m



Envolvente de Cortantes Fz+ en estado de límite de resistencia I (kN)

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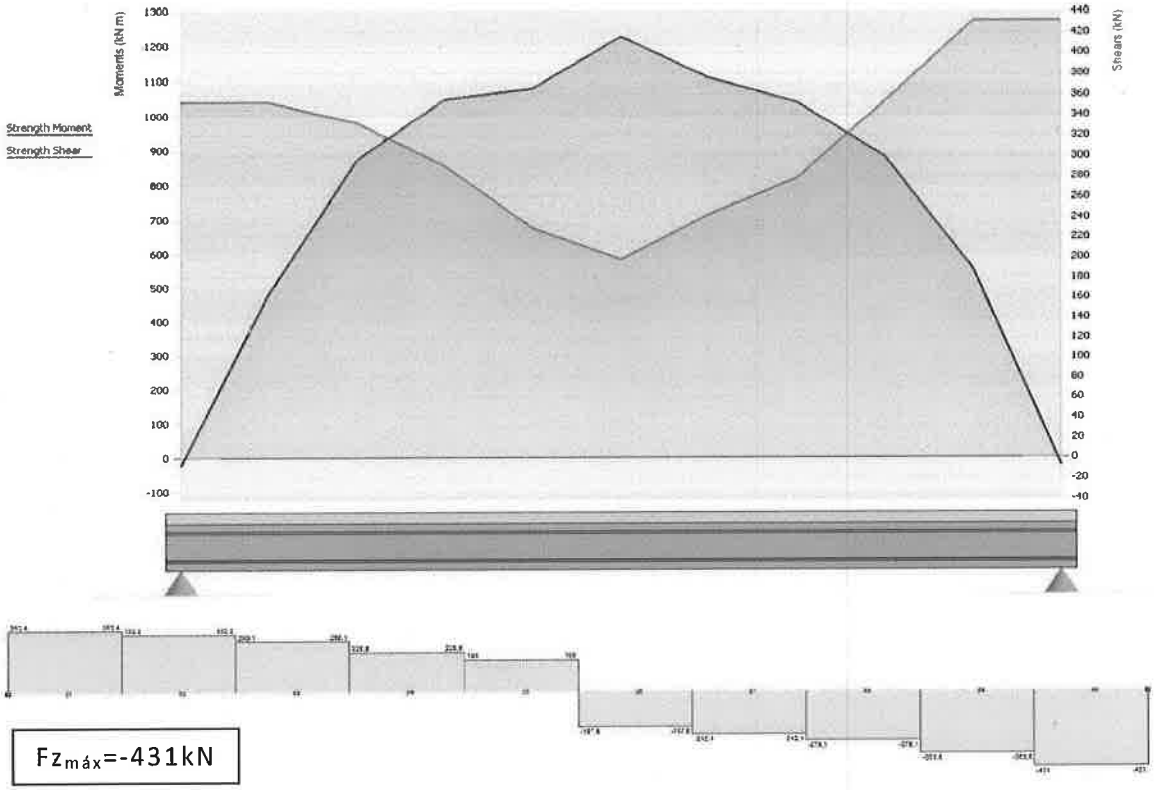
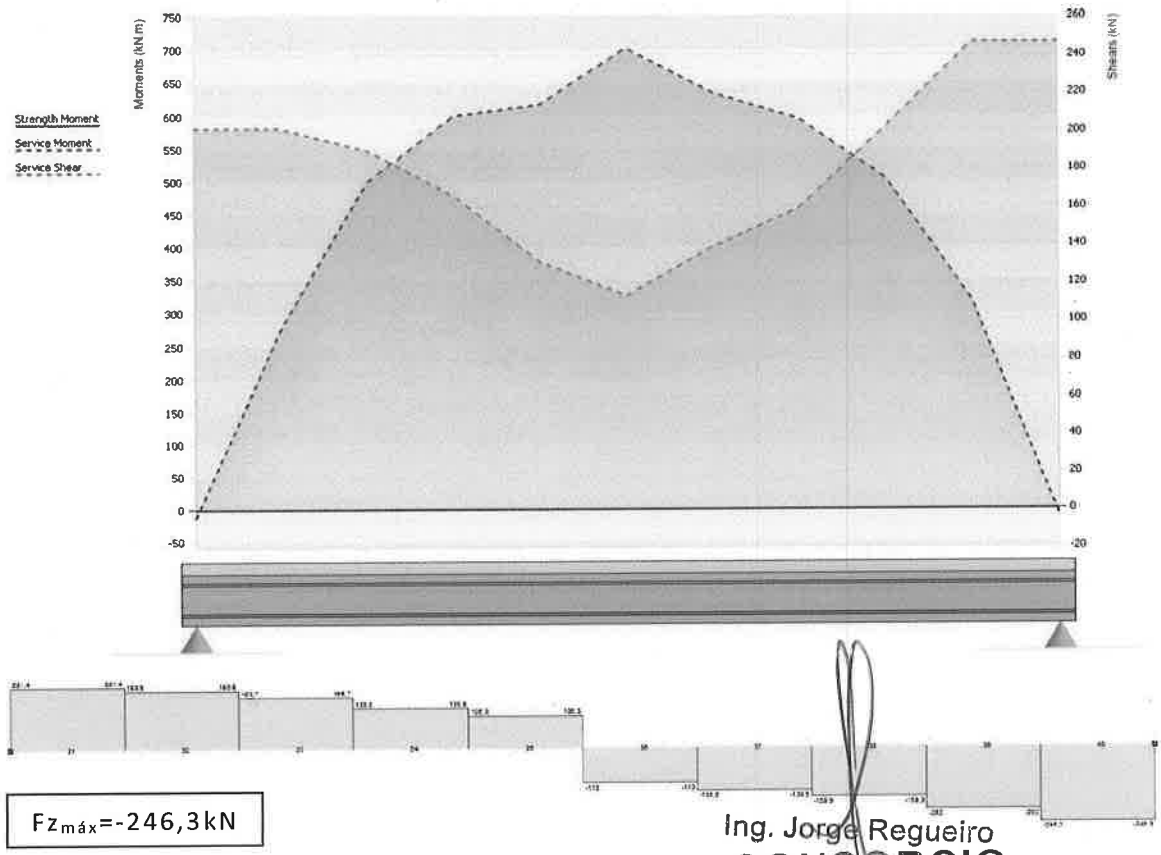


$Fz_{máx}$=-431kN

Envolvente de Cortantes Fz+ en estado de límite de servicio I (kN)



$Fz_{máx}$=-246,3kN

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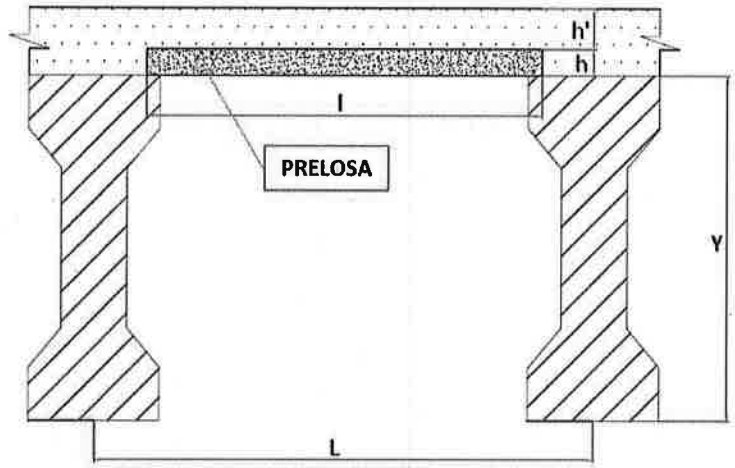
9) Dimensionamiento de las estructuras

Prelosas

Entrada de datos	
Dados de Materiales	
fck	30 MPa
fyk	500 MPa
Dados de Obra	
h	8,00 cm
h'	14,00 cm
l	220,00 cm
L	240,00 cm
Y	152,00 cm
Extensión Longitudinal	1500,00 cm
Recubrimiento	3,00 cm



- Ancho de prelosa: 36,50 cm
- Armadura inferior: 3 Ø 12,50 mm
- As = 3 x 1,23 cm² / 0,3650 m = 10,11 cm²/m ∴ Ø 12,50 c/ 12,16 cm
- Armadura negativa transversal: Ø 12,50 c/ 15 ∴ a_s = 8,33 cm²/m
- Armadura negativa longitudinal: Ø 10,00 c/ 15 ∴ a_s = 5,33 cm²/m
- Armadura transversal sobre prelosas: Ø 8,00 c/ 15 ∴ a_s = 3,33 cm²/m
- Armadura longitudinal sobre prelosas: Ø 10,00 c/ 15 ∴ a_s = 5,33 cm²/m

<u>Verificación prelosas premoldeadas – Montaje y hormigonado</u>

<u>Momento solicitante:</u>

De acuerdo con la hoja de cálculo:

Dimensionamiento de tablero										
Dados de los materiales		Dados del Tablero (cm)								
Fck (kgf/cm2)	Fyk (kgf/cm2)	h	h'	b	Recubri-miento	d'	d	l	L	Extensión Longitudinal
300	5000	8,00 cm	14,00 cm	100,00 cm	3,00 cm	3,90 cm	3,10 cm	220,00 cm	240,00 cm	1500,00 cm

Dimensionamiento de Prelosa										
Cargas			Verificación de Flexión			Verificación de Cortante				
g1	g2	p - acidenta	Mg	Mp	Mk	Vg	Vp	V_{sd}	V_{rd1}	Sin armadura transversal
0,20	0,35	0,2	0,33	0,12	0,60	0,61	0,22	1,15	3,09	

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$$g_1 = \frac{h}{100} \times 2{,}50 = 0{,}08 \times 2{,}50 = 0{,}20\ \frac{tf}{m^2}$$

$$g_2 = \frac{h'}{100} \times 2{,}50 = 0{,}14 \times 2{,}50 = 0{,}35\ \frac{tf}{m^2}$$

$$M_g = \frac{(g_1 + g_2) \times I^2}{8} = \frac{(0{,}20 + 0{,}35) \times 2{,}20^2}{8} = 0{,}33\ tf.m/m$$

$$M_p = \frac{p \times I^2}{8} = \frac{0{,}20 \times 2{,}20^2}{8} = 0{,}12\ tf.m/m$$

$$M_k = M_g \times 1{,}25 + M_p \times 1{,}50$$

$$\boldsymbol{M_k} = 0{,}33 \times 1{,}25 + 0{,}12 \times 1{,}50 = 0{,}60\ tf.\frac{m}{m} = \mathbf{6\ kN.m}$$

Momento resistente

De acuerdo con el cálculo del software:

```
Neutral axis free to rotate
Neutral axis angle (applied loads) = 0,0°  (clockwise from xx)
          depth in compression, c = 0,0223623 m

        Resistance Factor used, φ = 0,850135
```

Resistance Factor (φ) from Article 5.5.4.2.1

```
Type of force effect is:
  Non-tension-controlled and non-compression-controlled
    Compression-controlled strain limit in the extreme tension steel
(5,7,2,1)
                                                  εcl = -0,0026
    Tension-controlled strain limit in the extreme tension steel (5,7,2,1)
                                                  εtl =  -0,005
    Net tensile strain in extreme steel at nominal resistance
                                                  εt  = -0,00421

    nonprestressed section:

      0,75 <= φ = 0,75 + 0,15[ eps_t - eps_cl ]   <= 0,9     (Eq.: 5,5,4,2,1-
2)
                              [ eps_tl - eps_cl ]

                 0,75 + 0,15[ -0,0042 - -0,0026 ]
                            [ -0,005 - -0,0026 ]

              = 0,85013

            φ = 0,85013
```

MAXIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y
C40	(not used)			
CA50	-0,003976	-479,0406	28,0	28,0
CP190 RB	(not used)			
C30	0,003	25,499949	1000,0	80,0

MINIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y

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C40	(not used)			
CA50	-0,004211	-488,4314	973,75	26,25
CP190 RB	(not used)			
C30	-0,007732	0,0	0,0	0,0

LOADING relative to centroidal axes:

	Applied Loads	Required Strength	Spare Resistance	(Equilibrium Forces)
Mx kN.m	0,0	0,0	20,613934	20,613934
My kN.m	0,0	0,0	n/a	-1,3497775
Ax kN	0,0	0,0	n/a	0,61687619

Spare resistance for X Moment - Positive = 20,613934 kN.m

therefore limiting additional load = 20,6139 * 0,85013
= 17,5246 kN.m

Check Minimum Reinforcement

The section is noncompression-controlled, therefore Article 5.7.3.3.2 applies
for minimum reinforcement.

$M_{cr} = \gamma_3 .(\gamma_1 .f_r + \gamma_2 .f_{cpe}).S_c$ (5,7,3,3,2-1)

For Monolithic sections with no prestress,

$M_{cr} = \gamma_3 .\gamma_1 .f_r .S_c$ (5,7,3,3,2-1)

γ_1 = 1,6

γ_3 = 0,67 for A615, Grade 60 reinforcement
0,75 for A706, Grade 60 reinforcement

$f_r = 0,24\sqrt{f'_c}$
= 3,45169 MPa

S_c = 1106695,1 mm³

hence,

M_{cr} = 1,072 * 3,45169 * 1,1067E+06 = 4,095 kN.m for A615

M_{cr} = 1,2 * 3,45169 * 1,1067E+06 = 4,58396 kN.m for A706

M_r must be not less than the lesser of M_{cr} and $1,33M_u$

Summary of Internal Forces

	Force kN	Acting Height mm
compression force, F_{cd}		
- concrete	475,52358	70,4871
- reinforcement	*(no compression reinforcement)*	
- combined	475,52358	70,4871
tension force, F_{td}		
- reinforcement	-474,9067	27,1214

inner lever arm, z = 70,4871 - 27,1214 = 43,365735 mm

Note: Moments arising from eccentricity of axial loads are relative to the centroidal axis of the section.
The transformed section centroid is used.

$$\therefore M_r = 17,5246\ kN.m > M_k = 6\ kN.m$$

Verificación OK!

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Verificación losas del tablero – Utilización y homogeneizada

- Altura de losa: 22,00 cm

Momento resistente positivo

De acuerdo con el cálculo del software:

```
Neutral axis free to rotate
Neutral axis angle (applied loads) = 0,0°  (clockwise from xx)
          depth in compression, c = 0,0245246 m

        Resistance Factor used, φ = 0,9
```

Resistance Factor (φ) from Article 5.5.4.2.1

```
Type of force effect is:
  Tension-controlled reinforced concrete:
      φ = 0,90
```

MAXIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y
C40	(not used)			
CA50	-2,111E-4	-42,21252	26,25	193,75
CP190 RB	(not used)			
C30	0,003	25,499949	1000,0	220,0

MINIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y
C40	(not used)			
CA50	-0,020701	-500,0	973,75	26,25
CP190 RB	(not used)			
C30	-0,023912	0,0	0,0	0,0

LOADING relative to centroidal axes:

	Applied Loads	Required Strength	Spare Resistance	(Equilibrium Forces)
Mx kN.m	0,0	0,0	90,03915	90,03915
My kN.m	0,0	0,0	n/a	-0,4295147
Ax kN	0,0	0,0	n/a	0,00741976

```
Spare resistance for X Moment - Positive =  90,03915 kN.m

therefore limiting additional load = 90,0392 *      0,9
                                   = 81,0352 kN.m
```

Check Minimum Reinforcement

The section is noncompression-controlled, therefore Article 5.7.3.3.2 applies for minimum reinforcement.

$M_{cr} = \gamma_3 .(\gamma_1 .f_r + \gamma_2 .f_{cpe}).S_c$ (5,7,3,3,2-1)

For Monolithic sections with no prestress,

$M_{cr} = \gamma_3 .\gamma_1 .f_r .S_c$ (5,7,3,3,2-1)

$\gamma_1 = 1,6$

$\gamma_3 = 0,67$ for A615, Grade 60 reinforcement

$0,75$ for A706, Grade 60 reinforcement

$f_r = 0,24\sqrt{f'_c}$

$= 3,45169$ MPa

$S_c = 8627728,8$ mm³

hence,

$M_{cr} = 1,072 * 3,45169 * 8,62773E+06 = 31,9244$ kN.m for A615

$M_{cr} = 1,2 * 3,45169 * 8,62773E+06 = 35,7363$ kN.m for A706

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M_r must be not less than the lesser of M_{cr} and $1,33M_u$

Summary of Internal Forces

	Force kN	Acting Height mm
compression force, F_{cd}		
- concrete	521,9628	209,549
- reinforcement	*(no compression reinforcement)*	
- combined	521,9628	209,549
tension force, F_{td}		
- reinforcement	-521,9554	37,0472

inner lever arm, z = 209,549 - 37,0472 = 172,50212 mm

Note: Moments arising from eccentricity of axial loads are relative to the centroidal axis of the section.
The transformed section centroid is used.

$$\therefore M_{r\,(pos)} = 81,0352\ kN.m$$

Momento resistente negativo

De acuerdo con el cálculo del software:

Neutral axis free to rotate
Neutral axis angle (applied loads) = -179,93° (clockwise from xx)
depth in compression, c = 0,0226879 m

Resistance Factor used, φ = 0,9

Resistance Factor (φ) from Article 5.5.4.2.1
Type of force effect is:
Tension-controlled reinforced concrete:
φ = 0,90

MAXIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y
C40	(not used)			
CA50	-5,32E-4	-106,3933	973,75	26,25
CP190 RB	(not used)			
C30	0,003	25,499949	0,0	0,0

MINIMUM Strains:

Material Property	Strain	Stress MPa	location x	location y
C40	(not used)			
CA50	-0,022111	-500,0	973,75	193,75
CP190 RB	(not used)			
C30	-0,025497	0,0	1000,0	220,0

LOADING relative to centroidal axes:

	Applied Loads	Required Strength	Spare Resistance	(Equilibrium Forces)
Mx kN.m	0,0	0,0	-69,762397	-69,762397
My kN.m	0,0	0,0	n/a	-1,0578756
Ax kN	0,0	0,0	n/a	0,14742159

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Spare resistance for X Moment - Negative = -69,7624 kN.m

therefore limiting additional load = -69,762 * 0,9
= -62,786 kN.m

Check Minimum Reinforcement

The section is noncompression-controlled, therefore Article 5.7.3.3.2 applies
for minimum reinforcement.

$M_{cr} = \gamma_3 . (\gamma_1 . f_r + \gamma_2 . f_{cpe}) . S_c$ (5,7,3,3,2-1)

For Monolithic sections with no prestress,

$M_{cr} = \gamma_3 . \gamma_1 . f_r . S_c$ (5,7,3,3,2-1)

γ_1 = 1,6
γ_3 = 0,67 for A615, Grade 60 reinforcement
0,75 for A706, Grade 60 reinforcement
f_r = $0,24\sqrt{f'_c}$
= 3,45169 MPa
S_c = 8516136,2 mm³

hence,

M_{cr} = 1,072 * 3,45169 * 8,51614E+06 = 31,5115 kN.m for A615
M_{cr} = 1,2 * 3,45169 * 8,51614E+06 = 35,274 kN.m for A706

M_r must be not less than the lesser of M_{cr} and $1,33M_u$

Summary of Internal Forces

	Force kN	Acting Height mm
compression force, F_{cd}		
- concrete	480,57001	210,966
- reinforcement	(no compression reinforcement)	
- combined	480,57001	210,966
tension force, F_{td}		
- reinforcement	-480,4226	65,7837

inner lever arm, z = 210,966 - 65,7837 = 145,18235 mm

Note: Moments arising from eccentricity of axial loads are relative to the centroidal axis of the section.
The transformed section centroid is used.

$$\therefore M_{r\,(neg)} = -62,786\ kN.m$$

Momento solicitante positivo

Momento debido al peso proprio:

$$g = \frac{h}{100} \times 2,50 = 0,22 \times 2,50 = 0,55\ \frac{tf}{m^2}$$

$$M_g = \frac{g \times l^2}{8} = \frac{0,55 \times 2,20^2}{8} = 0,33\ tf.\frac{m}{m} = 3,30\ kN.m/m$$

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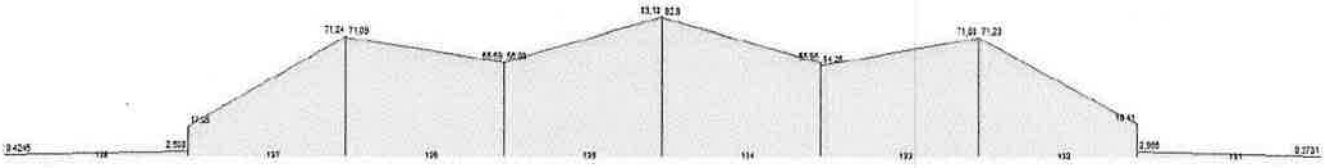
Momento máximo debido al envolvente de cargas (calculado pelo software):



$$\boldsymbol{M_{max\,total\,(pos)} = 83,18\ kN.m}$$

Momento en una losa:

$$\boldsymbol{M_{max\,(pos)} = \frac{83,18}{15/10} = 55,45\ kN.m/m}$$

$$M_k = M_g \times 1,25 + M_{max}$$

$$\boldsymbol{M_{k\,(pos)}} = 3,30 \times 1,25 + 55,45 = \boldsymbol{59,58\ kN.m}$$

$$\therefore \boldsymbol{M_{r\,(pos)} = 81,0352\ kN.m > M_{k\,(pos)} = 59,58\ kN.m}$$

Verificación OK!

<u>*Momento solicitante negativo*</u>

Momento debido al peso proprio para losa en voladizo:

$$\boldsymbol{M_g} = \frac{g \times l^2}{2} = \frac{(0,22 \times 2,5) \times 1,40^2}{2} = 0,54\ tf.\frac{m}{m} = \boldsymbol{5,39\ kN.m/m}$$

Momento máximo debido al envolvente de cargas (calculado pelo software):

$$\boldsymbol{M_{max\,total\,(neg)} = 5,491\ kN.m}$$

Momento en una losa:

$$\boldsymbol{M_{max\,(neg)} = \frac{5,491}{15/10} = 3,66\ kN.m/m}$$

$$M_k = M_g \times 1,25 + M_{max}$$

$$\boldsymbol{M_{k\,(neg)}} = 5,39 \times 1,25 + 3,66 = \boldsymbol{10,40\ kN.m}$$

$$\therefore \boldsymbol{M_{r\,(neg)} = 62,78\ kN.m > M_{k\,(neg)} = 10,40\ kN.m}$$

Verificación OK!

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Momento máximo debido al envolvente de cargas (para losas internas):

$$M_{max\,total\,(neg)} = 36,31\ kN.m$$

Momento en una losa:

$$M_{max\,(neg)} = \frac{36,31}{15/10} = 24,21\ kN.m/m$$

$$\therefore M_{r\,(neg)} = 62,78\ kN.m > M_{max\,(neg)} = 24,21\ kN.m$$

Verificación OK!

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Viga Pretensada

Tensión en estado límite de servicio I – izaje de la viga

Section details:
Ref 1 "Section 1"
at 0,5 x span = 0,5 x 15 = 7,5 m from left end of beam

Design Code:
AASHTO LRFD Bridge Design Specifications
Seventh Edition with 2016 Interim Revisions

Analysis:
Erection stage stresses
Limit State - Service I

ACTUAL STRESSES IN PRECAST BEAM

14 tendons are fully bonded at this section
0 tendons are fully debonded at this section
0 tendons are harped at this section

INITIAL PRESTRESS

height mm	No of tendons	f_{pu} MPa	area mm²	% of f_{pu}	initial force kN
55,0	4	1900,0	98,71	75,0	562,647
105,0	6	1900,0	98,71	75,0	843,9705
155,0	2	1900,0	98,71	75,0	281,3235
205,0	1	1900,0	98,71	75,0	140,66175
255,0	1	1900,0	98,71	75,0	140,66175
TOTAL	14				1969,2645

Transverse Eccentricity
Tendon layout is symmetrical about the vertical centroidal axis.

INITIAL RELAXATION - Article 5.9.5.4.4b
Data has the time from stressing to transfer set to zero, indicating that initial relaxation is not required to be calculated here.

height mm	No of tendons	f_{pj}	f_{py} MPa	relax. loss	area mm²	After relaxation force kN	moment kN.m
55,0	4	1425,0	1710,0	0,0	98,71	562,647	30,945585
105,0	6	1425,0	1710,0	0,0	98,71	843,9705	88,616902
155,0	2	1425,0	1710,0	0,0	98,71	281,3235	43,605142
205,0	1	1425,0	1710,0	0,0	98,71	140,66175	28,835659
255,0	1	1425,0	1710,0	0,0	98,71	140,66175	35,868746
TOTAL	14					1969,2645	227,87203

Moment about the centroid of the precast beam:

M_r = 227,87203-(1969,2645*0,45)
= -658,297 kN.m

Corresponding stresses:

top stress = 1969,26/222,5+-658,297/4,6234E7
= 8,850627+-14,23844
= -5,38781 MPa

bottom stress = 1969,26/222,5+-658,297/-4,623E7
= 8,850627+14,238437
= 23,0891 MPa

Self weight moment:

c.s.a. = 2,225E5 mm²
density = 25,0 kN/m³
self weight = 2,225E5*25,0
= 5,5625 kN/m
beam length = 15,0 m
distance = 7,5 m

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M_{sw} = 0,5*5,5625*7,5*(15,0-7,5)
= 156,445 kN.m

Corresponding stresses:
top stress = 156,445/4,6234E7
= 3,38379 MPa
bottom stress = 156,445/-4,623E7
= -3,3838 MPa

ELASTIC SHORTENING LOSSES - Article 5.9.5.2.3a

concrete stresses at mid-span:
stress at top of precast beam = -2,004 MPa
stress at bottom of precast beam = 19,7053 MPa
depth of precast beam = 900,0 mm

height mm	No of tendons	conc stress MPa	E_p / E_{ci}	tendon force kN	tendon moment kN.m
55,0	4	18,3786	5,2626	38,188612	2,1003737
105,0	6	17,17253	5,2626	53,523799	5,6199989
155,0	2	15,96645	5,2626	16,588227	2,5711751
205,0	1	14,76038	5,2626	7,6675934	1,5718567
255,0	1	13,55431	5,2626	7,0410736	1,7954738
TOTAL	14			123,0093	13,658878

Moment about the centroid of the precast beam:
M_{ed} = 13,658878-(123,0093*0,45)
= -41,69531 kN.m

hence top stress = -2,004-123,009/222,5--41,6953/4,6234E7
= -2,004-0,5528508--0,901836
= -1,65504 MPa
bottom stress = 19,705-123,009/222,5--41,6953/-4,623E7
= 19,705-0,5528508-0,9018361
= 18,2506 MPa

After a further 2 iterations of the above process, the top and bottom stresses are as follows:
top stress = -1,67857 MPa
bottom stress = 18,3483 MPa

REFINED ESTIMATES OF TIME-DEPENDENT LOSSES - Article 5.9.5.4

Elastic modulus of prestressing steel: E_p = 197,0 kN/mm²
Total effective tendon area: A_{ps} = 1381,9 mm²
Elastic modulus of concrete at transfer: E_{ci} = 37,434 kN/mm²
Percentage relative humidity: H = 75,0 %
Humidity factor for creep: k_{hc} = 1,56-0,008H = 0,96
Humidity factor for shrinkage: k_{hs} = 2,00-0,014H = 0,95
Age at the end of curing: t_c = 3,0 day
Age at transfer: t_i = 14,0 day
Age at in-situ casting: t_d = 28,0 day
Final age: t_f = 120,0 day
Comp. strength of conc. at 1st loading: f'_{ci} = 40,0 MPa (= 5,802 KSI)

Factor for the effect of concrete strength: $k_f = \frac{35,0}{7+f'_{ci}} = \frac{35,0}{7+40,0} =$
2,7341

Time factor for creep at final time: $k_{tdc,f,i} = \frac{t_f - t_i}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_i}$

$= \frac{120,0-14,0}{12*\left[\frac{100-4*5,8}{5,8+20}\right] + 120,0-14,0}$

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$$= 0,7479$$

Time fac. for shrinkage at final time: $k_{tds,f,i} = \dfrac{t_f - t_c}{12*\left[\dfrac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_c}$

$$= \frac{120,0-3,0}{12*\left[\dfrac{100-4*5,8}{5,8+20}\right] + 120,0-3,0}$$

$$= 0,7661$$

Losses: Time of Transfer to Time of Deck Placement - Article 5.9.5.4.2

Gross area of section: $A_g = 2,22E5$ mm²

Perimeter of section: $p = 2938,5$ mm

Volume to surface ratio: $V/S = A_g/p = 75,718$ mm

Factor for the effect of the volume: $k_s = 1,45-0,0051*(V/S) = 1,0638$

Creep coefficient at final time:

$$\psi_b(t_f, t_i) = 1,9*k_s.k_{hc}.k_f.k_{tdc,f,i}.t_i^{-0,118}$$
$$= 1,9*1,0638*0,96*2,7341*0,7479*14,0^{-0,118}$$
$$= 2,9064$$

Inertia of the gross concrete section: $I_g = 2,08E10$ mm⁴

Eccentricity of prestressing force: $e_{pg} = -334,0$ mm

Transformed section coefficient:

$$K_{id} = \frac{1}{1 + \dfrac{E_p}{E_{ci}}.\dfrac{A_{ps}}{A_g}\left[1+\dfrac{A_g.e_{pg}^2}{I_g}\right].[1+0,7*\psi_b(t_f, t_i)]}$$

$$= \frac{1}{1 + \dfrac{197,0}{37,434}*\dfrac{1381,9}{2,22E5}\left[1+\dfrac{2,22E5*-334,0^2}{2,1E10}\right]*(1+0,7*2,9064)}$$

$$= 0,82134$$

Time factor for creep at deck placement:

$$k_{tdc,d,i} = \frac{t_d - t_i}{12*\left[\dfrac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_d - t_i}$$

$$= \frac{28,0-14,0}{12*\left[\dfrac{100-4*5,8}{5,8+20}\right] + 28,0-14,0}$$

$$= 0,281$$

Time factor for shrinkage at deck placement:

$$k_{tds,d,i} = \frac{t_d - t_c}{12*\left[\dfrac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_d - t_c}$$

$$= \frac{28,0-3,0}{12*\left[\dfrac{100-4*5,8}{5,8+20}\right] + 28,0-3,0}$$

$$= 0,411$$

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Shrinkage of Girder Concrete - Article 5.9.5.4.2a

Concrete shrinkage strain of girder: $\varepsilon_{bid} = k_s \cdot k_{hs} \cdot k_f \cdot k_{tds,d,i}$ *0,00048

=

1,062*0,95*0,73*0,41*0,00048

= 1,466E-4

Loss of prestress = $\varepsilon_{bid} \cdot E_p \cdot K_{id}$
= 1,47E-4*197,0*0,90021
= 26,007 MPa
= Δf_{pSR}

Total shrinkage loss of force = $\Delta f_{pSR} \cdot A_{ps}$
= 26,0073*1381,94
= 35,9405 kN

Corresponding gain of moment = 35,9405*0,116
(about y = 0) = 4,16911 kN.m

Moment about the centroid of the composite beam:
M_{sh} = 4,16911-(35,9405*0,45)
= -12,004 kN.m

Corresponding stresses:
top stress = 35,9405/222500+-12,0041/4,623E7
= 0,1615302+-0,259639
= -0,0981089 MPa

bottom stress = 35,9405/222500+-12,0041/-4,623E7
= 0,1615302+0,2596391
= 0,421169 MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:
$\psi_b(t_d, t_i) = 1,9 k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,d,i} \cdot t_i^{-0,118}$
$= 1,9*1,0*0,96*0,7351*0,2816*14,0^{-0,118}$
= 0,2765

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,9918 MPa
at bottom of precast beam = 19,6931 MPa
at center of gravity of prestress: f_{cgp} = 16,8981 MPa

Loss of prestress = $\frac{E_p}{E_{ci}} \cdot f_{cgp} \cdot \psi_b(t_d, t_i) \cdot K_{id}$
= 197,0/37,434*16,898*0,2765*0,9126
= 22,4453 MPa
= Δf_{pCR}

Total creep loss of force = $\Delta f_{pCR} \cdot A_{ps}$
= 22,4453*1381,94
= 31,0181 kN

Corresponding loss of moment = 31,0181*0,116
= 3,59811 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{cr} = 3,59811-(31,0181*0,45)
= -10,36 kN.m

Corresponding stresses:
top stress = 31,0181/2,225E5+-10,36/4,62338E7
= 0,13940+-0,2240
= -0,0847 MPa

bottom stress = 31,0181/2,225E5+-10,36/-4,6234E7

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= 0,13940+0,22407
= 0,36348 MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.2c

Stress in tendons after transfer: f_{pt} = 1341,96 MPa
Yield strength of prestressing steel: f_{py} = 1710,0 MPa
Time between tensioning and deck placement: $t = t_s - t_i$ = 28,0-14,0 = 14,0day
Factor for relaxation steel type: K'_L = 45,0 (low relaxation)

$$\text{Loss of prestress} = \frac{f_{pt}}{K'_L} \cdot \frac{\log(24t)}{\log(24t_i)} \cdot \left[\frac{f_{pt}}{f_{py}} - 0,55\right] \cdot \left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right] \cdot K_{id}$$

$$= \frac{1342,0}{45,0} * \frac{\log(24*14,0)}{\log(24*14,0)} * \left[\frac{1342,0}{1710,0} - 0,55\right] \left[1 - \frac{3*(26,007+22,445)}{1342,0}\right] * 0,9$$

= 5,61989 MPa

$= \Delta f_{pR1}$

Total relaxation loss of force = $\Delta f_{pR1} \cdot A_{ps}$
= 5,6199*1381,9
= 7,7663445 kN

Corresponding loss of moment = 7,7663* 0,116
= 0,9008996 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{rx} = 0,9009-(7,76634*0,45)
= -2,594 kN.m

Corresponding stresses:
top stress = 7,76634/222500+-2,59396/4,623E7
= 0,0349049+-0,056105
= -0,0212003 MPa

bottom stress = 7,76634/222500+-2,59396/-4,623E7
= 0,0349049+0,0561052
= 0,0910101 MPa

Erection of beam Loading

Bending moment from erection loadcase at current span location:
$M_{Applied}$ = 156,44531 kN.m
Corresponding stresses:
top stress = 156,44531/4,6234E7
= 3,38379 MPa
bottom stress = 156,44531/-4,623E7
= -3,3838 MPa

Remove the dead load applied for transfer calculations
M_{sw} = -156,45 kN.m
Corresponding stresses:
top stress = -156,45/4,6234E7
= -3,3838 MPa
bottom stress = -156,45/-4,623E7
= 3,38379 MPa

PERMISSIBLE STRESSES IN PRECAST BEAM

Compression
Article 5.9.4.2.1 for permament loads
Permissible compression stress = 0,45 * f'_c
= 0,45 * 40

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= 18 MPa

Tension

Article 5.9.4.2.2

Permissible tension stress = $-0,5 * \lambda * \sqrt{f'_c}$

= $-0,5 * 1,0 * \sqrt{40}$

= -3,15469 MPa

SERVICE LIMIT STATE - STRESS SUMMARY TABLE

Note: The reinforcement has only been included in the calculation of the stresses after the cast in place 1A/1B stage.

	force kN	moment kN.m	Concrete Stresses (MPa) Cast In Place top	bottom	Precast top	bottom
Prestress	1969,26	-658,3			-5,3878	23,0891
Self Weight		156,445			3,38379	-3,3838
Prestress + Self Wt.					-2,004	19,7053
Elastic Def	-114,76	38,8934			0,32545	-1,357
TRANSFER	1854,5	-462,96			-1,6786	18,3483
Erection		0,0			0,0	0,0
TOTAL PERMANENT					-1,6786	18,3483

SLS FLEXURE
calculated using curvature of uncracked sections

The deflections are calculated from integration of the curvatures along the beam, using the parameters detailed below:

Elastic Modulus at Transfer, E_T = 37,434 kN/mm²

Intermediate Term Elastic Modulus, E_I = 28,9327 kN/mm²

$E_I = E_c / (\psi(t,t_i)+1)$

where $\psi(t,t_i)$ = 0,29382 (Article 5.14.2.3.6)]

Precast section height = 900,0 mm

		Precast Stress (MPa)	E	Strain (x10⁻⁶)	Curvature (x10⁻⁶) (rad/m)	Deflection[1] (mm) Here	Max.
At Transfer	T	-1,6786	E_T	-44,841	-594,43	16,6609	Cracked
	B	18,3483		490,15			
At Erection	T	-1,4746	E_I	-50,966	-727,63	19,2132	19,2132
	B	17,4726		603,904			

[1] Deflection 'Cracked' indicates tensile stress at one or more sections along the beam exceeds the stress limit at transfer.

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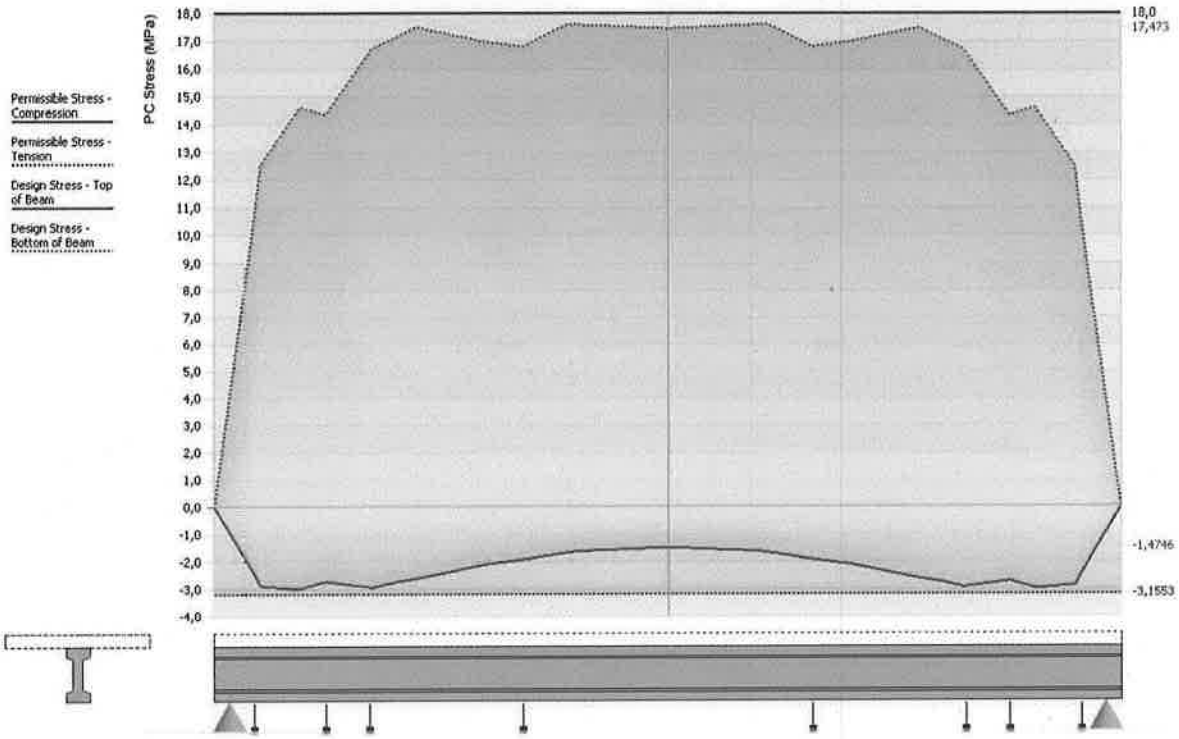

Tensión en estado límite de servicio I – etapa de construcción 1A

Section details:

Ref 1 "Section 1"
at 0,5 x span = 0,5 x 15 = 7,5 m from left end of beam

Design Code:

AASHTO LRFD Bridge Design Specifications
Seventh Edition with 2016 Interim Revisions

Analysis:

Construction stage stresses
Limit State - Service I

ACTUAL STRESSES IN PRECAST BEAM

---

14 tendons are fully bonded at this section
0 tendons are fully debonded at this section
0 tendons are harped at this section

INITIAL PRESTRESS

height mm	No of tendons	f_{pu} MPa	area mm²	% of f_{pu}	initial force kN
55,0	4	1900,0	98,71	75,0	562,647
105,0	5	1900,0	98,71	75,0	843,9705
155,0	2	1900,0	98,71	75,0	281,3235
205,0	1	1900,0	98,71	75,0	140,66175
255,0	1	1900,0	98,71	75,0	140,66175
TOTAL	14				1969,2645

Transverse Eccentricity

Tendon layout is symmetrical about the vertical centroidal axis.

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INITIAL RELAXATION - Article 5.9.5.4.4b

Data has the time from stressing to transfer set to zero, indicating that initial relaxation is not required to be calculated here.

height mm	No of tendons	f_{pj}	f_{py} MPa	relax. loss	area mm²	After relaxation force kN	moment kN.m
55,0	4	1425,0	1710,0	0,0	98,71	562,647	30,945585
105,0	6	1425,0	1710,0	0,0	98,71	843,9705	88,616902
155,0	2	1425,0	1710,0	0,0	98,71	281,3235	43,605142
205,0	1	1425,0	1710,0	0,0	98,71	140,66175	28,835659
255,0	1	1425,0	1710,0	0,0	98,71	140,66175	35,868746
TOTAL	14					1969,2645	227,87203

Moment about the centroid of the precast beam:

M_r = 227,87203-(1969,2645*0,45)
= -658,297 kN.m

Corresponding stresses:

top stress = 1969,26/222,5+-658,297/4,6234E7
= 8,850627+-14,23844
= -5,38781 MPa

bottom stress = 1969,26/222,5+-658,297/-4,623E7
= 8,850627+14,238437
= 23,0891 MPa

Self weight moment:

c.s.a. = 2,225E5 mm²
density = 25,0 kN/m³
self weight = 2,225E5*25,0
= 5,5625 kN/m
beam length = 15,0 m
distance = 7,5 m
M_{sw} = 0,5*5,5625*7,5*(15,0-7,5)
= 156,445 kN.m

Corresponding stresses:

top stress = 156,445/4,6234E7
= 3,38379 MPa
bottom stress = 156,445/-4,623E7
= -3,3838 MPa

ELASTIC SHORTENING LOSSES - Article 5.9.5.2.3a

concrete stresses at mid-span:
stress at top of precast beam = -2,004 MPa
stress at bottom of precast beam = 19,7053 MPa
depth of precast beam = 900,0 mm

height mm	No of tendons	conc stress MPa	E_p / E_{ci}	tendon force kN	tendon moment kN.m
55,0	4	18,3786	5,2626	38,188612	2,1003737
105,0	6	17,17253	5,2626	53,523799	5,6199989
155,0	2	15,96645	5,2626	16,588227	2,5711751
205,0	1	14,76038	5,2626	7,6675934	1,5718567
255,0	1	13,55431	5,2626	7,0410736	1,7954738
TOTAL	14			123,0093	13,658878

Moment about the centroid of the precast beam:

M_{ed} = 13,658878-(123,0093*0,45)
= -41,69531 kN.m

hence top stress = -2,004-123,009/222,5--41,6953/4,6234E7
= -2,004-0,5528508--0,901836
= -1,65504 MPa

bottom stress = 19,705-123,009/222,5--41,6953/-4,623E7
= 19,705-0,5528508-0,9018361
= 18,2506 MPa

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After a further 2 iterations of the above process, the top and bottom stresses are as follows:

top stress = -1,67857 MPa
bottom stress = 18,3483 MPa

REFINED ESTIMATES OF TIME-DEPENDENT LOSSES - Article 5.9.5.4

Elastic modulus of prestressing steel: E_p = 197,0 kN/mm²
Total effective tendon area: A_{ps} = 1381,9 mm²
Elastic modulus of concrete at transfer: E_{ci} = 37,434 kN/mm²
Percentage relative humidity: H = 75,0 %
Humidity factor for creep: k_{hc} = 1,56-0,008H = 0,96
Humidity factor for shrinkage: k_{hs} = 2,00-0,014H = 0,95
Age at the end of curing: t_c = 3,0 day
Age at transfer: t_i = 14,0 day
Age at in-situ casting: t_d = 28,0 day
Final age: t_f = 120,0 day
Comp. strength of conc. at 1st loading: f'_{ci} = 40,0 MPa (= 5,802 KSI)

Factor for the effect of concrete strength: $k_f = \frac{35,0}{7+f'_{ci}} = \frac{35,0}{7+40,0} =$ 2,7341

Time factor for creep at final time: $k_{tdc,f,i}$

$$= \frac{t_f - t_i}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_i}$$

$$= \frac{120,0-14,0}{12*\left[\frac{100-4*5,8}{5,8+20}\right] + 120,0-14,0}$$

$$= 0,7479$$

Time fac. for shrinkage at final time: $k_{tds,f,i}$

$$= \frac{t_f - t_c}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_c}$$

$$= \frac{120,0-3,0}{12*\left[\frac{100-4*5,8}{5,8+20}\right] + 120,0-3,0}$$

$$= 0,7661$$

Losses: Time of Transfer to Time of Deck Placement - Article 5.9.5.4.2

Gross area of section: A_g = 2,22E5 mm²
Perimeter of section: p = 2938,5 mm
Volume to surface ratio: $V/S = A_g/p$ = 75,718 mm
Factor for the effect of the volume: k_s = 1,45-0,0051*(V/S) = 1,0638
Creep coefficient at final time:

$$\psi_b(t_f, t_i) = 1,9*k_s.k_{hc}.k_f.k_{tdc,f,i}.t_i^{-0,118}$$
$$= 1,9*1,0638*0,96*2,7341*0,7479*14,0^{-0,118}$$
$$= 2,9064$$

Inertia of the gross concrete section: I_g = 2,08E10 mm⁴
Eccentricity of prestressing force: e_{pg} = -334,0 mm
Transformed section coefficient:

$$K_{id} = \frac{1}{1 + \frac{E_p}{E_{ci}}.\frac{A_{ps}}{A_g}\left[1+\frac{A_g.e_{pg}^2}{I_g}\right].[1+0,7*\psi_b(t_f, t_i)]}$$

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$$= \frac{1}{1 + \frac{197,0}{37,434} * \frac{1381,9}{2,22E5}\left[1 + \frac{2,22E5*-334,0^2}{2,1E10}\right]*(1+0,7*2,9064)}$$

$= 0,82134$

Time factor for creep at deck placement:

$$k_{tdc,d,i} = \frac{t_d - t_i}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_d - t_i}$$

$$= \frac{28,0-14,0}{12*\left[\frac{100-4*5,8}{5,8+20}\right] + 28,0-14,0}$$

$= 0,281$

Time factor for shrinkage at deck placement:

$$k_{tds,d,i} = \frac{t_d - t_c}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_d - t_c}$$

$$= \frac{28,0-3,0}{12*\left[\frac{100-4*5,8}{5,8+20}\right] + 28,0-3,0}$$

$= 0,411$

Shrinkage of Girder Concrete - Article 5.9.5.4.2a

Concrete shrinkage strain of girder: $\varepsilon_{bid} = k_s . k_{hs} . k_f . k_{tds,d,i}$
*0,00048

=

1,062*0,95*0,73*0,41*0,00048

$= 1,466E-4$

Loss of prestress $= \varepsilon_{bid} . E_p . K_{id}$
$= 1,47E-4*197,0*0,90021$
$= 26,007$ MPa
$= \Delta f_{pSR}$

Total shrinkage loss of force $= \Delta f_{pSR} . A_{ps}$
$= 26,0073*1381,94$
$= 35,9405$ kN

Corresponding gain of moment $= 35,9405*0,116$
(about $y = 0$) $= 4,16911$ kN.m

Moment about the centroid of the composite beam:
$M_{sh} = 4,16911-(35,9405*0,45)$
$= -12,004$ kN.m

Corresponding stresses:
top stress $= 35,9405/222500+-12,0041/4,623E7$
$= 0,1615302+-0,259639$
$= -0,0981089$ MPa

bottom stress $= 35,9405/222500+-12,0041/-4,623E7$
$= 0,1615302+0,2596391$
$= 0,421169$ MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:

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$\psi_b\ (t_d\ ,t_i\ )\ =\ 1,9k_s\ .k_{hc}\ .k_f\ .k_{tdc,d,i}\ \ .t_i^{\ -0,118}$
$=\ 1,9*1,0*0,96*0,7351*0,2816*14,0^{-0,118}$
$=\ 0,2765$

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,9918 MPa
at bottom of precast beam = 19,6931 MPa
at center of gravity of prestress: f_{cgp} = 16,8981 MPa

$$\text{Loss of prestress} = \frac{E_p}{E_{ci}}.f_{cgp}\ .\psi_b\ (t_d\ ,t_i\ ).K_{id}$$
$$= \frac{197,0}{37,434}*16,898*0,2765*0,9126$$
$$= 22,4453 \text{ MPa}$$
$$= \Delta f_{pCR}$$

Total creep loss of force $= \Delta f_{pCR}\ .A_{ps}$
$= 22,4453*1381,94$
$= 31,0181$ kN

Corresponding loss of moment = 31,0181*0,116
= 3,59811 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{cr} = 3,59811-(31,0181*0,45)
= -10,36 kN.m

Corresponding stresses:
top stress = 31,0181/2,225E5+-10,36/4,62338E7
= 0,13940+-0,2240
= -0,0847 MPa

bottom stress = 31,0181/2,225E5+-10,36/-4,6234E7
= 0,13940+0,22407
= 0,36348 MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.2c

Stress in tendons after transfer: f_{pt} = 1341,96 MPa
Yield strength of prestressing steel: f_{py} = 1710,0 MPa
Time between tensioning and deck placement: $t = t_s - t_i$ = 28,0-14,0 = 14,0day
Factor for relaxation steel type: K'_L = 45,0 (low relaxation)

$$\text{Loss of prestress} = \frac{f_{pt}}{K'_L}.\frac{\log(24t)}{\log(24t_i)}.\left[\frac{f_{pt}}{f_{py}} - 0,55\right].\left[1-\frac{3(\Delta f_{pSR}+\Delta f_{pCR})}{f_{pt}}\right].K_{id}$$

$$= \frac{1342,0}{45,0}*\frac{\log(24*14,0)}{\log(24*14,0)}*\left[\frac{1342,0}{1710,0}-0,55\right]\left[1-\frac{3*(26,007+22,445)}{1342,0}\right]*0,9$$

$= 5,61989$ MPa

$= \Delta f_{pR1}$

Total relaxation loss of force $= \Delta f_{pR1}\ .A_{ps}$
$= 5,6199*1381,9$
$= 7,7663445$ kN

Corresponding loss of moment = 7,7663* 0,116
= 0,9008996 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{rx} = 0,9009-(7,76634*0,45)

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= -2,594 kN.m

Corresponding stresses:
top stress = 7,76634/222500+-2,59396/4,623E7
= 0,0349049+-0,056105
= -0,0212003 MPa

bottom stress = 7,76634/222500+-2,59396/-4,623E7
= 0,0349049+0,0561052
= 0,0910101 MPa

Erection of beam Loading

Bending moment from erection loadcase at current span location:
$M_{Applied}$ = 156,44531 kN.m
Corresponding stresses:
top stress = 156,44531/4,6234E7
= 3,38379 MPa
bottom stress = 156,44531/-4,623E7
= -3,3838 MPa

Remove the dead load applied for transfer calculations
M_{sw} = -156,45 kN.m
Corresponding stresses:
top stress = -156,45/4,6234E7
= -3,3838 MPa
bottom stress = -156,45/-4,623E7
= 3,38379 MPa

Construction stage 1A Loading

$M_{Applied}$ = 371,25 kN.m
Corresponding stresses:
top stress = 371,25/4,6234E7
= 8,02984 MPa
bottom stress = 371,25/-4,623E7
= -8,0298 MPa

Losses: Time of Deck Placement to Final Time - Article 5.9.5.4.3

Gross composite section area: A_c = 7,51E5 mm²
Perimeter of section: p = 7378,5 mm
Volume to surface ratio: $V/S = A_c/p$ = 101,71 mm
Factor for the effect of the volume: k_s = 1,45-0,0051*(V/S) = 1,0
Creep coefficient at final time:

$$\psi_b(t_f, t_i) = 1{,}9 * k_s . k_{hc} . k_f . k_{tdc,f,i} . t_i^{-0{,}118}$$
$$= 1{,}9*1{,}0*0{,}96*2{,}734*0{,}748*14{,}0^{-0{,}118}$$
$$= 2{,}732$$

Inertia of the net concrete section: I_c = 7,2E10 mm⁴
Eccentricity of prestressing force: e_{pc} = 727,98 mm
Transformed section coefficient:

$$K_{df} = \frac{1}{1 + \frac{E_p}{E_{ci}} . \frac{A_{ps}}{A_c} \left[1 + \frac{A_c . e_{pc}^2}{I_c}\right] . [1+0{,}7*\psi_b(t_f, t_i)]}$$

$$= \frac{1}{1 + \frac{197{,}0}{37{,}434} * \frac{1381{,}9}{7{,}51E5} \left[1 + \frac{7{,}51E5*727{,}98^2}{7{,}2E10}\right] * (1+0{,}7*2{,}732)}$$

$$= 0{,}84454$$

Shrinkage of Girder Concrete - Article 5.9.5.4.3a

Concrete shrinkage strain of girder: $\varepsilon_{bdf} = k_s . k_{hs} . k_f . k_{tds,f,i}$
*0,00048
=
1,062*0,95*0,73*0,76*0,00048
= 2,729E-4

Loss of prestress = $\varepsilon_{bdf} . E_p . K_{df}$
= 2,73E-4*1,97E8*0,91265
= 49,0591 MPa
= Δf_{pSD}

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Total shrinkage loss of force = $\Delta f_{pSD} \cdot A_{ps}$
= 49,0591*1381,94
= 67,7968 kN

Corresponding gain of moment = 67,7968*0,116
(about y = 0) = 7,86446 kN.m

Moment about the centroid of the composite beam:
M_{sh} = 7,86446-(67,7968*0,82762)
= -48,246 kN.m

Corresponding stresses:
top stress = 67,7968/709521+-48,2456/1,011E9
= 0,0955528+-0,047726
= 0,0478273 MPa

bottom stress = 67,7968/709521+-48,2456/-8,841E7
= 0,0955528+0,5457267
= 0,64128 MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:
$\psi_b\ (t_d\ ,t_i\ )$ = $1{,}9k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,d,i} \cdot t_i^{-0{,}118}$
= $1{,}9*1{,}0625*0{,}96*0{,}7351*0{,}2816*14{,}0^{-0{,}118}$
= 0,2938

Creep coefficient at final time:
$\psi_b\ (t_f\ ,t_d\ )$ = $1{,}9k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,f,i} \cdot t_d^{-0{,}118}$
= $1{,}9*1{,}0625*0{,}96*0{,}7351*0{,}7203*28{,}0^{-0{,}118}$
= 0,6926

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,9918 MPa
at bottom of precast beam = 19,6931 MPa
at center of gravity of prestress: f_{cgp} = 16,8981 MPa

Change in concrete stress at centroid of prestressing strands:
concrete stress change after permanent loads:
at top of precast beam = 3,37545 MPa
at bottom of precast beam = 11,9509 MPa
Δf_{cd} = 10,8456 MPa

$$\text{Loss of prestress} = \frac{E_p}{E_{ci}} * f_{cgp} * \left[\psi_b\ (t_f\ ,t_i\ ) - \psi_b\ (t_d\ ,t_i\ )\right] * K_{df}$$
$$+ \frac{E_p}{E_c} * \Delta f_{cd} * \psi_b\ (t_f\ ,t_d\ ) * K_{df}$$
$$= \frac{197{,}0}{37{,}434} * 16{,}8981*(0{,}78045-0{,}29382)*0{,}90021$$
$$+ \frac{197{,}0}{18{,}717} * 10{,}8456*0{,}69259*0{,}90021$$
$$= 110{,}13 \text{ MPa}$$
$$= \Delta f_{pCD}$$

Total creep loss of force = $\Delta f_{pCD} * A_{ps}$
= 110,13*1381,94
= 152,192 kN

Corresponding loss of moment = 152,192*0,116
= 17,6544 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{cr} = 17,6544-(152,192*0,82762)
= -108,3 kN.m

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Corresponding stresses:
top stress = 152,192/7,095E5+-108,3/1,0109E9
= 0,2145+-0,1071
= 0,10736 MPa

bottom stress = 152,192/7,095E5+-108,3/-8,8406E7
= 0,2145+1,22507
= 1,43957 MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.3c

Loss of prestress = Δf_{pR1}
= 5,61989 MPa
= Δf_{pR2}

Total relaxation loss of force = Δf_{pR2} . A_{ps}
= 5,6199*1381,9
= 7,7663445 kN

Corresponding loss of moment = 7,7663* 0,116
= 0,9008996 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{rx} = 0,9009-(7,76634*0,8276)
= -5,5267 kN.m

Corresponding stresses:
top stress = 7,76634/709521+-5,5267/1,011E9
= 0,0109459+-0,005467
= 0,00547877 MPa

bottom stress = 7,76634/709521+-5,5267/-8,841E7
= 0,0109459+0,0625148
= 0,0734607 MPa

Shrinkage of Deck Concrete - Article 5.9.5.4.3d

Concrete shrinkage strain of girder: ε_{ddf} = k_s . k_{hs} . k_f . $k_{tds,f,i}$ *0,00048
=
1,0*0,95*0,73*0,76*0,00048
= 2,568E-4

Creep coefficient at final time:
$\psi_b\ (t_f\ ,t_d\ )$ = 1,9k_s . k_{hc} . k_f . $k_{tdc,f,i}$. $t_d^{-0,118}$
= 1,9*1,0*0,96*0,7351*0,7203*28,0$^{-0,118}$
= 0,6518

Change in concrete stress at centroid of prestressing strands:

Area of deck concrete: A_d = 5,28E5 mm²
Perimeter of section: p = 5240,0 mm
Volume to surface ratio: V/S = A_d /p = 100,76 mm
Factor for the effect of the volume: k_s = 1,45-0,0051*(V/S) = 1,4298
Modulus of elasticity of deck concrete: E_{cd} = 34,0436 mm²
Eccentricity of the deck: e_d = 166,023 mm³

$$\Delta f_{cdf} = \frac{\varepsilon_{ddf}\ .A_d\ .E_{cd}}{[1+0,7\psi_b\ (t_f\ ,t_d\ )]}\left[\frac{1}{A_c}\ \frac{e_{pc}\ .e_d}{I_c}\right]$$

$$= \frac{2,6E\text{-}4*5,28E5*34,044*1000}{[1+0,7*0,6518]}\left[\frac{1}{7,1E5}\ \frac{711,621*166,023}{7,3E10}\right]$$

= -0,6508 Mpa

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Loss of prestress $= \frac{E_p}{E_c} . \Delta f_{cdf} \ . K_{df} \ . [1+0,7\psi_b \ (t_f \ , t_d \ )]$

$= \frac{197,0}{18,717} * -0,6508 * 0,91265 \ * (1+0,7*0,65187)$

$= -9,1055$ MPa

$= \Delta f_{pss}$

Total shrinkage gain of force $= \Delta f_{pss} \ . A_{ps}$
$= -9,1055*1381,94$
$= -12,583$ kN

Corresponding gain of moment $= -12,583*0,116$
(about $y = 0$) $= -1,4597$ kN.m

Moment about the centroid of the composite beam:
$M_{sh} = -1,4597-(-12,583*0,82762)$
$= 8,95452$ kN.m

Corresponding stresses:
top stress = -12,5833/709521+8,95452/1,011E9
= -0,017735+0,008858
= -0,00887687 MPa

bottom stress = -12,5833/709521+8,95452/-8,841E7
= -0,017735+-0,101288
= -0,119023 MPa

PERMISSIBLE STRESSES IN PRECAST BEAM

Compression

Article 5.9.4.2.1 for permament loads
Permissible compression stress $= 0,45 * f'_c$
$= 0,45 * 40$
$= 18$ MPa

Tension

Article 5.9.4.2.2
Permissible tension stress $= -0,5 * \lambda * \sqrt{f'_c}$
$= -0,5 * 1,0 * \sqrt{40}$
$= -3,15469$ MPa

PERMISSIBLE STRESSES FOR CAST IN PLACE CONCRETE

Compression

Article 5.9.4.2
char. strength of c.i.p. concrete = 30,0 MPa
stress limitation factor = 0,6
permissible compression stress = $0,6*f_{cu}$
$= 0,6 * 30$
$= 18,0$ MPa

Tension

Article 5.7.3.4
Flexural tensile stresses may need to be limited by cracking considerations
in accordance with article 5.7.3.4.

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SERVICE LIMIT STATE - STRESS SUMMARY TABLE

Note: The reinforcement has only been included in the calculation of the stresses after the cast in place 1A/1B stage.

	force kN	moment kN.m	Concrete Stresses (MPa) Cast In Place top	bottom	Precast top	bottom
Prestress	1969,26	-658,3			-5,3878	23,0891
Self Weight		156,445			3,38379	-3,3838
Prestress + Self Wt.					-2,004	19,7053
Elastic Def	-114,76	38,8934			0,32545	-1,357
TRANSFER	1854,5	-462,96			-1,6786	18,3483
Erection		0,0			0,0	0,0
C.I.P. Stage 1A		371,25			8,02984	-8,0298
Time-Dependent Losses						
Time of Transfer to Time of Deck Placement						
Shrinkage	-35,94	12,0041			0,09811	-0,4211
Creep	-31,018	10,36			0,08467	-0,3634
Relaxation	-7,7663	2,59396			0,0212	-0,091
Time of Deck Placement to Final Time						
Shrinkage						
Girder	-67,797	48,2456	0,08843	-0,0435	-0,0478	-0,6412
Deck	21,1793	-15,072	-0,0276	0,01359	0,01494	0,20033
Creep	-152,19	108,303	0,19851	-0,0976	-0,1073	-1,4396
Relaxation	-7,7663	5,5267	0,01013	-0,005	-0,0055	-0,0735
TOTAL PERMANENT			0,26945	-0,1325	6,40953	7,48879

SLS FLEXURE
calculated using curvature of uncracked sections

The deflections are calculated from integration of the curvatures along the beam, using the parameters detailed below:

Elastic Modulus at Transfer, E_T = 37,434 kN/mm²

Intermediate Term Elastic Modulus, E_I = 28,9327 kN/mm²

$E_I = E_c / (\psi(t,t_i)+1)$

where $\psi(t,t_i)$ = 0,29382 (Article 5.14.2.3.6)]

Short Term Elastic Modulus, E_S = 37,434 kN/mm²

Precast section height = 900,0 mm

		Precast Stress (MPa)	E	Strain (x10⁻⁶)	Curvature (x10⁻⁶) (rad/m)	Deflection[1] (mm) Here	Max.
At Transfer	T	-1,6786	E_T	-44,841	-594,43	16,6609	Cracked
	B	18,3483		490,15			
At Erection	T	-1,4746	E_I	-50,966	-727,63	19,2132	19,2132
	B	17,4726		603,904			
From in situ 1A	T	8,02984	E_S	214,507	476,682	-10,505	-10,505
	B	-8,0298		-214,51			

[1] Deflection 'Cracked' indicates tensile stress at one or more sections along the beam exceeds the stress limit at transfer.

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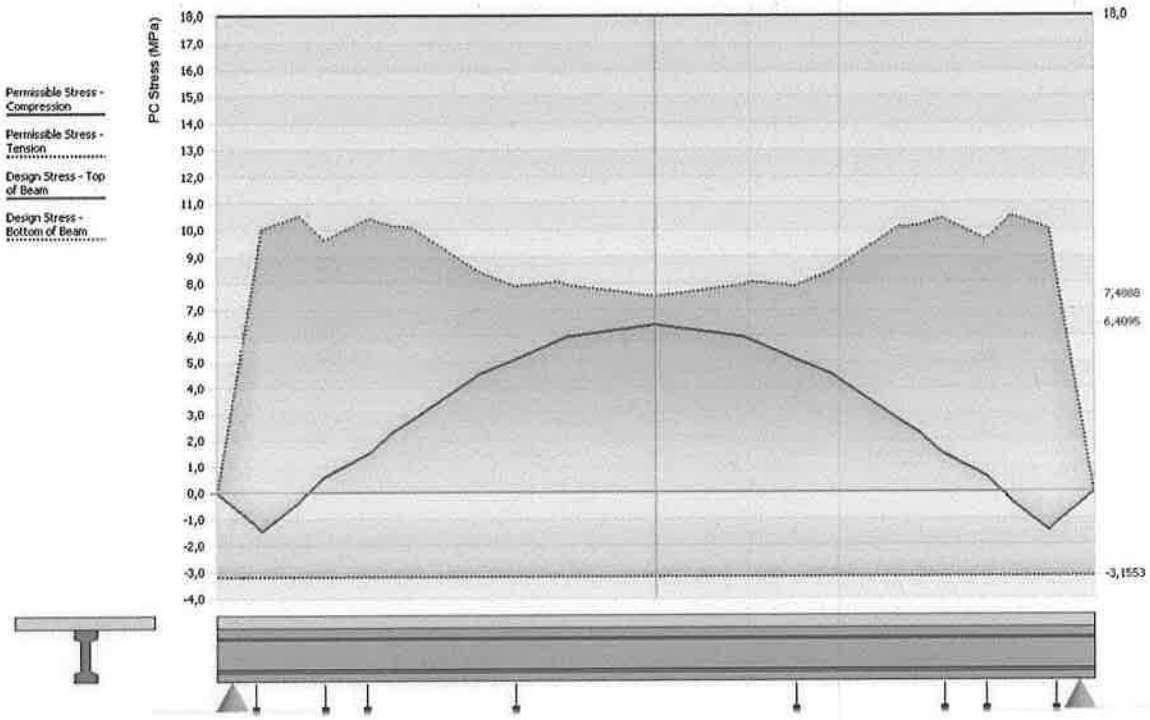

Tensión en estado límite de servicio I – cargas vivas

Section details:
Ref 1 "Section 1"
at 0,5 x span = 0,5 x 15 = 7,5 m from left end of beam

Design Code:
AASHTO LRFD Bridge Design Specifications
Seventh Edition with 2016 Interim Revisions

Analysis:
Live Load Bending Moments
Transient Load BM 2
Limit State - Service I
& Service III
for Normal environment conditions [A5,9,4,2,2b]

ACTUAL STRESSES IN PRECAST BEAM

14 tendons are fully bonded at this section
0 tendons are fully debonded at this section
0 tendons are harped at this section

INITIAL PRESTRESS

height mm	No of tendons	f_{pu} MPa	area mm²	% of f_{pu}	initial force kN
55,0	4	1900,0	98,71	75,0	562,647
105,0	6	1900,0	98,71	75,0	843,9705
155,0	2	1900,0	98,71	75,0	281,3235
205,0	1	1900,0	98,71	75,0	140,66175
255,0	1	1900,0	98,71	75,0	140,66175
TOTAL	14				1969,2645

Transverse Eccentricity
Tendon layout is symmetrical about the vertical centroidal axis.

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INITIAL RELAXATION - Article 5.9.5.4.4b

Data has the time from stressing to transfer set to zero, indicating that initial relaxation is not required to be calculated here.

height mm	No of tendons	f_{pj}	f_{py} MPa	relax. loss	area mm²	After relaxation force kN	moment kN.m
55,0	4	1425,0	1710,0	0,0	98,71	562,647	30,945585
105,0	6	1425,0	1710,0	0,0	98,71	843,9705	88,616902
155,0	2	1425,0	1710,0	0,0	98,71	281,3235	43,605142
205,0	1	1425,0	1710,0	0,0	98,71	140,66175	28,835659
255,0	1	1425,0	1710,0	0,0	98,71	140,66175	35,868746
TOTAL	14					1969,2645	227,87203

Moment about the centroid of the precast beam:

M_r = 227,87203-(1969,2645*0,45)
= -658,297 kN.m

Corresponding stresses:

top stress = 1969,26/222,5+-658,297/4,6234E7
= 8,850627+-14,23844
= -5,38781 MPa

bottom stress = 1969,26/222,5+-658,297/-4,623E7
= 8,850627+14,238437
= 23,0891 MPa

Self weight moment:

c.s.a. = 2,225E5 mm²
density = 25,0 kN/m³
self weight = 2,225E5*25,0
= 5,5625 kN/m
beam length = 15,0 m
distance = 7,5 m
M_{sw} = 0,5*5,5625*7,5*(15,0-7,5)
= 156,445 kN.m

Corresponding stresses:

top stress = 156,445/4,6234E7
= 3,38379 MPa
bottom stress = 156,445/-4,623E7
= -3,3838 MPa

ELASTIC SHORTENING LOSSES - Article 5.9.5.2.3a

concrete stresses at mid-span:

stress at top of precast beam = -2,004 MPa
stress at bottom of precast beam = 19,7053 MPa
depth of precast beam = 900,0 mm

height mm	No of tendons	conc stress MPa	E_p / E_{ci}	tendon force kN	tendon moment kN.m
55,0	4	18,3786	5,2626	38,188612	2,1003737
105,0	6	17,17253	5,2626	53,523799	5,6199989
155,0	2	15,96645	5,2626	16,588227	2,5711751
205,0	1	14,76038	5,2626	7,6675934	1,5718567
255,0	1	13,55431	5,2626	7,0410736	1,7954738
TOTAL	14			123,0093	13,658878

Moment about the centroid of the precast beam:

M_{sd} = 13,658878-(123,0093*0,45)
= -41,69531 kN.m

hence top stress = -2,004-123,009/222,5--41,6953/4,6234E7
= -2,004-0,5528508--0,901836
= -1,65504 MPa

bottom stress = 19,705-123,009/222,5--41,6953/-4,623E7
= 19,705-0,5528508-0,9018361
= 18,2506 MPa

After a further 2 iterations of the above process, the top and bottom stresses are as follows:

top stress = -1,67857 MPa

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