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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:

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

$$= \frac{1}{197,0\ 1381,9 \left[1 + \frac{2,22E5*-334,0^2}{}\right]}$$

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$$1 + \frac{}{37,434}*\frac{}{2,22E5}\left[1+\frac{}{2,1E10}\right]*(1+0,7*2,9064)$$

$$= 0,82134$$

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: $\varepsilon_{bid} = k_s \cdot k_{hs} \cdot k_f \cdot k_{tds,d,i}$

*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 (about y = 0) = 35,9405*0,116
= 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

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Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:

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

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)

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

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

$$= 5{,}61989 \text{ MPa}$$

$$= \Delta f_{pR1}$$

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)
= -2,594 kN.m

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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:

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

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

$$= 0{,}84454$$

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

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$= \Delta f_{pSD}$

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

Corresponding gain of moment (about $y = 0$) $= 67,7968*0,116$
$= 7,86446$ 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:
$\psi_b (t_d , t_i) = 1,9 k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,d,i} \cdot t_i^{-0,118}$
$= 1,9*1,0625*0,96*0,7351*0,2816*14,0^{-0,118}$
$= 0,2938$

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

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
$\Delta f_{cd} = 10,8456$ MPa

$$\text{Loss of prestress} = \frac{E_p}{E_{ci}} * f_{cgp} * \left[\psi_b (t_f , t_i) - \psi_b (t_d , t_i) \right] * K_{df} + \frac{E_p}{E_c} * \Delta f_{cd} * \psi_b (t_f , t_d) * K_{df}$$

$$= \frac{197,0}{37,434} * 16,8981 * (0,78045 - 0,29382) * 0,90021 + \frac{197,0}{18,717} * 10,8456 * 0,69259 * 0,90021$$

$= 110,13$ 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 (about $y = 0$) $= 152,192*0,116$
$= 17,6544$ kN.m

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Moment about the centroid of the composite beam:

M_{cr} = 17,6544-(152,192*0,82762)
= -108,3 kN.m

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 = $\Delta f_{pR2} \cdot A_{ps}$
= 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*0,8276)
= -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: $\varepsilon_{ddf} = k_s \cdot k_{hs} \cdot k_f \cdot k_{tds,f,i}$
*0,00048

=

1,0*0,95*0,73*0,76*0,00048

= 2,568E-4

Creep coefficient at final time:

$\psi_b (t_f, t_d) = 1{,}9 k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,f,i} \cdot t_d^{-0{,}118}$
= 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

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

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

= -0,6508 MPa
E_p

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

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

$$= -9,1055 \text{ MPa}$$

$$= \Delta f_{pss}$$

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

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

SDL non-structural concrete etc. Loading

$M_{Applied} = 241,84309$ kN.m
$P_{Applied} = -76,21191$ kN

Corresponding stresses:

top stress $= -76,21191/709521,3 + 241,84309/1,0109E9$
$= -0,1074 + 0,23923$
$= 0,13182$ MPa

bottom stress $= -76,21191/709521,3 + 241,84309/-8,841E7$
$= -0,1074 + -2,736$
$= -2,843$ MPa

Transient Load BM 2 Loading

$M_{applied} = 798,366$ kN.m for Service-I
$638,692$ kN.m for Service-III

$N_{applied} = -309,73$ kN for Service-I
$-247,78$ kN for Service-III

Corresponding stresses:

top stress $= -309,73/7,095E5 + 798,366/1,0109E9$ Service-I
$= -0,4365 + 0,78976$
$= 0,35322$ MPa

bottom stress $= -247,78/7,095E5 + 638,692/-8,841E7$ Service-III
$= -0,3492 + -7,2245$
$= -7,5737$ MPa

PERMISSIBLE STRESSES IN PRECAST BEAM

Compression

Article 5.9.4.2.1 for permanent & transient loads

Permissible compression stress $= 0,6 * f'_c$
$= 0,6*40$
$= 24,0$ MPa

Tension

Article 5.9.4.2.2

Component is subject to normal exposure conditions.

Permissible tension stress $= -0,498 * \lambda * \sqrt{f'_c} >= -4,137$ MPa
$= -0,498 * 1,0 * \sqrt{40}$
$= -3,15469$ MPa

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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.

SERVICE LIMIT STATE - STRESS SUMMARY TABLE
minimum for girder bottom extreme fiber

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	Cast In Place bottom	Precast top	Precast 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
Utilities etc.		241,843	0,87889	0,21756	0,23923	-2,7356
	-76,212		-0,0977	-0,0977	-0,1074	-0,1074
(Diff. Shr. 1	-21,179	15,0716	0,0	0,0	-0,0149	-0,2003)
(Diff. Shr. 2		0,0	0,0	0,0	0,0	0,0)
(Differential shrinkage is not adverse or not specified						)
TOTAL PERMANENT			1,05066	-0,0126	6,54135	4,64578
(γ_{TG} = 0,5)						
Temp Grad P1	-932,71	-198,11	2,27188	-0,6867	-0,7550	0,92638
Temp Grad R1	186,543	39,6229	-0,4543	0,13734	0,15101	-0,1852
Temp Grad R2		0,01687	6,13E-5	1,52E-5	1,67E-5	-1,9E-4
(Temperature restraint is not adverse or not specified						)
TRANSIENT LOADS						
Transient Load BM 1- I		0,0	0,0		0,0	
- III		0,0		0,0		0,0
Transient Load BM 2- I	798,366	2,90137			0,78976	
	-309,73		-0,3969		-0,4365	
- III	638,692			0,57458		-7,234

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	-247,78		-0,3176		-0,3492
Transient Load BM 3- I	798,366	2,90137		0,78976	
	-310,51	-0,398		-0,4376	
- III	638,692		0,57458		-7,2245
	-248,41		-0,3184		-0,3501
Transient Load BM 4- I	-1,4268	-0,0052		-0,0014	
	1,69255	0,00217		0,00239	
- III	-1,1414		-0,001		0,01291
	1,35404		0,00174		0,00191
Transient Load BM 5- I	571,09	2,07542		0,56493	
	-284,26	-0,3643		-0,4006	
- III	456,872		0,41101		-5,1679
	-227,4		-0,2914		-0,3205
Transient Load BM 6- I	-0,7683	-0,0028		-7,6E-4	
	2,59067	0,00332		0,00365	
- III	-0,6146		-5,5E-4		0,00695
	2,07253		0,00266		0,00292

TOTAL PERMANENT LOAD + TRANSIENT LOAD

Transient Load BM 2	3,10071	-0,4423	6,13949	-3,1134
Transient Load BM 3	3,09971	-0,4431	6,13839	-3,1143
Transient Load BM 5	2,30741	-0,5798	5,95057	-1,0281

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:
(Note that the reported values of deflections for live loadings are exaggerated when these are derived from enveloped effects)

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)]

Effective Elastic Modulus for time dependant effects, E_E = 28,4624 kN/mm²

$E_E = E_S . (\psi_{tc\%} + ((1-\psi_{tc\%}) / (1 + \chi.\psi)))$

ψ = creep factor = 0,78045

$\psi_{tc\%}$ = percentage of creep occurring before c.i.p. deck added = 0,29382/0,78045 = 37,6482%

χ = ageing factor, taken as 0,8

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	16,6555	Cracked
	B	18,3483		490,15			
At Erection	T	-1,4746	E_I	-50,966	-727,63	19,2112	19,2112
	B	17,4726		603,904			
Long-term Dead	T	6,54135					
	B	4,64578					
Dead Stress Increment	T	8,21992	E_E	288,799	855,804	-19,08	-19,08
	B	-13,702		-481,42			
Total Dead Load deflection					261,369	-2,4241	0,0

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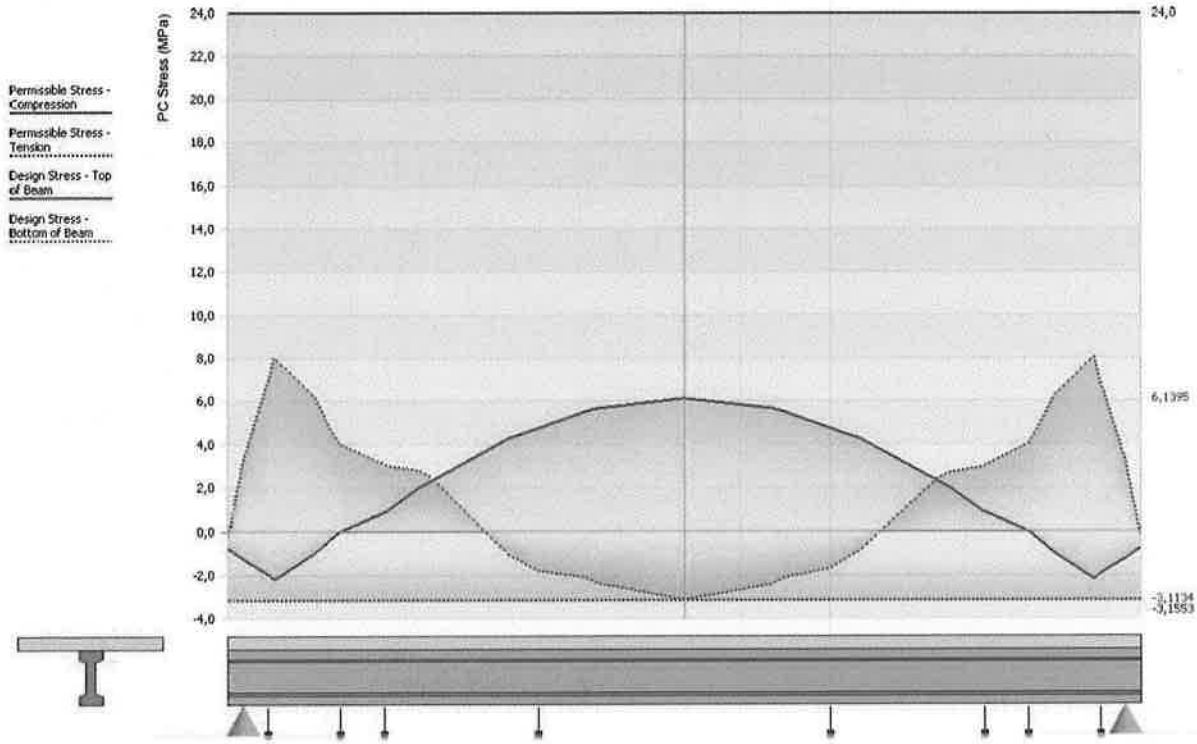
Transient Load BM 2-I	T	0,35322	E_s	9,43606	291,488	-6,5937	-6,5937
	B	-9,4672		-252,9			
Extreme in-service		-4,8214			552,858	-9,0178	Cracked

[1] Deflection 'Cracked' indicates tensile stress at one or more sections along the beam exceed the stress limits for concrete both in service and at transfer.



Momento en estado límite de resistencia I

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 1
Limit State - Strength I

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Strength Limit State Stress/Strain summary for section with NEGATIVE moment

height of neutral axis = 146,762 mm
depth in compression = 146,762 mm

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)

$$\varepsilon_{cl} = -0,002$$

Tension-controlled strain limit in the extreme tension steel (5,7,2,1)

$$\varepsilon_{tl} = -0,005$$

Net tensile strain in extreme steel at nominal resistance

$$\varepsilon_t = -0,00222$$

prestressed section:

$$0,75 <= \varphi = 0,75 + 0,25\left[\frac{eps_t - eps_{cl}}{eps_{tl} - eps_{cl}}\right] <= 1,0 \quad \text{(Eq.: 5,5,4,2,1-1)}$$

$$0,75 + 0,25\left[\frac{-0,0022 - -0,002}{-0,005 - -0,002}\right]$$

$$= 0,76793$$

$$\varphi = 0,76793$$

Location		strain	stress MPa	height mm
Precast beam	- *top*	-0,015397	0,0	900,0
	- *bottom*	0,003	33,9999	0,0
C.I.P. stage 1	- *top*	-0,019894	0,0	1120,0
	- *bottom*	-0,015397	0,0	900,0
Tendon	- *top*	-0,009107	-1596,3	255,0
	- *bottom*	-0,0049	-965,32	55,0
Reinforcement	- *top*	-0,014579	-500,0	860,0
	- *bottom*	0,0021824	400,0	40,0

Ultimate Strength capacity of section = 234,567*0,767492
= 180,029 kN.m
In conjunction with axial load of -14,9632 kN

Strength Limit State Stress/Strain summary for section with POSITIVE moment

height of neutral axis = 1053,41 mm
depth in compression = 66,5896 mm

Resistance Factor (φ) from Article 5.5.4.2.1
Type of force effect is:
Tension-controlled prestressed concrete:
φ = 1,00

Location		strain	stress MPa	height mm
Precast beam	- *top*	-0,006911	0,0	900,0
	- *bottom*	-0,047458	0,0	0,0
C.I.P. stage 1	- *top*	0,003	25,4999	1120,0
	- *bottom*	-0,006911	0,0	900,0
Tendon	- *top*	-0,042865	-1900,0	255,0
	- *bottom*	-0,051756	-1900,0	55,0
Reinforcement	- *top*	-0,008714	-500,0	860,0
	- *bottom*	-0,045656	-500,0	40,0

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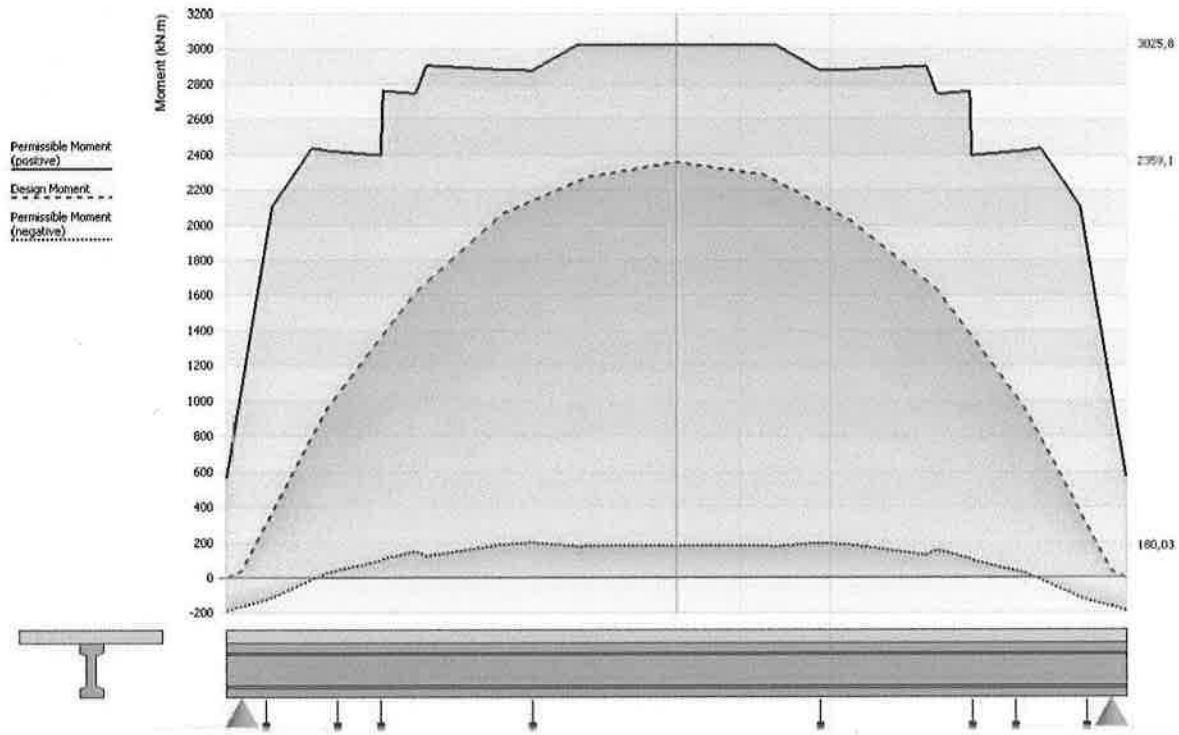
Ultimate Strength capacity of section = 3025,82*1
= 3025,82 kN.m
In conjunction with axial load of -14,9632 kN

SUMMARY OF APPLIED LOADINGS

	BENDING MOMENT kN.m	AXIAL kN
Erection loading before composite	= 195,55664	0,0
Erection loading to stage 1A	= 464,0625	0,0
Utility loading etc.	= 302,30387	-95,26488
Transient loading	= 1397,1398	-542,0237
TOTAL	2359,0628	-637,2885



My_{total} = 2359,06 kN.m < 3025,80 kN.m

Cortante

Section details:
Ref 1 "Section 1"
at 0,113 x span = 0,113 x 15 = 1,7 m from left end of beam

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

Analysis:
Shear Forces and associated Bending Moments
Transient Load SF + assoc. BM 1
Limit State - Strength I

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SUMMARY OF APPLIED LOAD EFFECTS

	SHEAR FORCE kN	BENDING MOMENT kN.m	AXIAL LOAD kN
Erection loading before composite =	40,328125	77,701172	0,0
Erection loading to stage 1A =	95,7	184,3875	0,0
Wearing surface loading =	0,0	0,0	0,0
Utility loading etc =	68,844553	100,69574	-43,1964
Transient loading:			
Max SF + associated BM =	352,41213	483,50326	-272,3785
Max BM + associated SF =	433,59993	605,68977	-277,69
TOTAL - Max SF + associated BM:	557,28481	846,28768	-315,5749
- Max BM + associated SF:	638,47261	968,47418	-320,8864

Note:
The following effects are not considered (article 3.4.1) -
differential settlement
temperature gradient

Article 5.8.3
Transverse Shear Reinforcement

The *'BM with associated SF'* loadcase is found to be critical at this section.

factored shear force V_u = 638,473 kN
factored moment M_u = 968,474 kN.m

The factored moment at this section is positive, and so the following calculations relate to the section as defined

Calculate effective shear depth, d_v - Article 5.8.2.9

overall depth of section h = 1120,0 mm
compression fiber to tension force centroid d_e = 961,947 mm
resistance moment of section (no axial load) M_r = 2502,71 kN.m
total tension force T = 2663,15 kN
distance from resultant tension to compression = M_r / T
= 0,93975 m

$0,9*d_e$ = 0,9*961,947 = 865,752 mm
$0,72*h$ = 0,72*1120,0 = 806,4 mm
hence, effective shear depth, d_v = 939,757 mm

Critical section for shear - Article 5.8.3.2

angle of compressive stress (see below) θ = 33,4256°

$0,5*d_v .\cot(\theta)$ = 0,5*939,757*1,5151
= 711,915 mm
critical section for shear is larger of d_v and 711,915 mm
hence, critical section for shear:
from face of support = 0,93975 m
therefore the section at the current point of interest is beyond the
critical section for shear.

Determine β and θ - Article 5.8.3.4

Shear stress on the concrete - 5,8,2,9-1

Resistance factor for shear [A5,5,4,2] φ = 0,9
Vertical component of prestress at section V_p = 0,0 kN

$$v = \frac{V_u - \varphi.V_p}{\varphi.b_v.d_v} = \frac{638,473 - 0,9*0,0}{0,9*0,15*0,93975} = 5,03261 \text{ MPa}$$

Factored axial force N_u = 315,575 kN

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Area of prestress steel on tension side A_{ps} = 987,1 mm²
Area of concrete on flexural tension side A_c = 1,278E5 mm²
Area of non-prestress steel on tension side A_s = 1060,09 mm²
Modulus of elasticity of tendons E_p = 197,0 kN/mm²
Modulus of elasticity of non-prestress steel E_s = 200,0 kN/mm²
Modulus of elasticity of concrete E_c = 37,434 kN/mm²

$f_{po} = 0,7*f_{pu}$
$= 0,7*1900,0$
$= 1330,0$ MPa

$|V_u - V_p|*d_v$ = 600,009 kN.m
Factored moment (not less than $|V_u - V_p|d_v$) $|M_u|$ = 968,474 kN.m

General procedure: (Article 5.8.3.4.2)
Strain in reinforcement:

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0,5N_u + |V_u - V_p| - A_{ps} f_{po}}{E_s A_s + E_p A_{ps}}$$ (Eq. 5,8,3,4,2-4)

$$\varepsilon_s = \frac{\left[\frac{968,47}{0,9397} + 0,5*315,57 + |638,47 - 0,0| - 987,1* 1,33\right]}{200,0*1060,1 + 197,0*987,1}$$

$= 0,0013$

with at least the minimum amount of shear reinforcement:

$$\beta = \frac{4,8}{(1 + 750\varepsilon_s)} = \frac{4,8}{(1 + 750*0,0013)}$$ (Eq. 5,8,3,4,2-1)

$= 2,46363$

with no shear reinforcement:
s_x = 0,08292 m

maximum aggregate size: a_g = 25,4 mm = 1,0 IN

$$s_{xe} = s_x * \frac{1,38}{a_g + 0,63} = 0,083 * \frac{1,38}{1,0 + 0,63} = 0,0702 \text{ m}$$

$= 2,76382$ IN

$12,0\text{IN} <= s_{xe} <= 80,0\text{IN}$

s_{xe} = 12,0 IN

$$\beta = \frac{4,8}{(1 + 750\varepsilon_s)} * \frac{51}{(39 + s_{xe})}$$ (Eq. 5,8,3,4,2-2)

$$= \frac{4,8}{(1 + 750*0,0013)} * \frac{51}{(39 + 12,0)}$$

$= 2,46363$

$\theta = 29 + 3500\varepsilon_s$ (Eq. 5,8,3,4,2-3)

$= 29 + 3500* 0,0013$
$= 33,4256°$

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Nominal Shear Resistance
Article 5.8.3.3

inclination of the transverse reinforcement, α = 90,0°

nominal shear resistance $V_n = V_c + V_s + V_p$ (5,8,3,3-1)

with at least the minimum amount of shear reinforcement:

$V_c = 0,08297*\beta\lambda(\sqrt{f'_c}).b_v.d_v$ (5,8,3,3-3)
$= 0,08297*2,4636*1,0*(\sqrt{40,0})*150,0*939,76$
$= 182,24$ kN

with no shear reinforcement:

$V_c = 0,08297*\beta\lambda(\sqrt{f'_c}).b_v.d_v$ (5,8,3,3-3)
$= 0,08297*2,4636*1,0*(\sqrt{40,0})*150,0*939,76$
$= 182,24$ kN

$V_p = 0,0$ kN
V_s depends upon the reinforcement present.

Maximum Shear Force
Articles 5.8.2.4 and 5.8.3.3

Maximum value of shear resistance with no transverse reinforcement

$V_{r,no\ links} = 0,5*\varphi.(V_c + V_p)$ (5,8,2,4-1)
$= 0,5*0,9*(182,242+0,0)$
$= 82,0087$ kN

Maximum value of shear resistance with transverse reinforcement

$V_{n,max} = 0,25 * f'_c.b_v.d_v + V_p$ (5,8,3,3-2)
$= 0,25 * 40,0*150,0*939,757 + 0,0$
$= 1409,64$ kN

$V_{r,max} = \varphi.V_{n,max}$
$= 0,9*1409,64$
$= 1268,67$ kN

Transverse Reinforcement Requirements

inclination of the transverse reinforcement, α = 90,0°
minimum yield strength of transverse rft f_y = 500,0 MPa

Minimum Transverse Reinforcement (article 5.8.2.5):

$$\frac{A_v}{s} >= \frac{0,083*\lambda*\sqrt{f'_c}.b_v}{f_y}$$

$= 0,083*1,0*\sqrt{40,0}*150,0/500,0$
$= 0,1575$ mm²/mm

Shear resistance (article 5.8.3.3) with minimum reinforcement (article 5.8.3.3):

$V_n = V_c + V_s + V_p$

$V_s = A_v/s . f_y.d_v.(\cot(\theta)+\cot(\alpha)).\sin(\alpha)$ (5,8,3,3-4)

$V_{s,min} = 0,15754* 500,0*939,757* 1,5151$
$= 112,157$ kN

therefore,

$V_{n,min} = 182,246 + 112,157 + 0,0$
$= 294,404$ kN
$\varphi.V_{n,min} = 0,9*294,404$
$= 264,963$ kN

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Additional transverse reinforcement for shear (article 5.8.3.3):

V_u is greater than $\phi V_{n,min}$, therefore additional transverse reinforcement is required.

contribution to shear strength from shear rft,

$V_s = V_u/\phi - (V_c + V_p)$
$= 638,473/ 0,9-(182,246+ 0,0)$
$= 527,168$ kN

from equation 5,8,3,3-4

$A_v/s = V_s / (f_y . d_v . (\cot(\theta)+\cot(\alpha)) . \sin(\alpha))$

hence, $A_v/s = 527,168 / (500,0*939,757* 1,5151)$
$= 0,740492$ mm²/mm

Link arrangement:

size	diameter mm	Maximum spacing mm 2 legs	4 legs
10	11,3	270,867	541,734
15	16,0	543,049	1086,1
20	19,5	806,619	1613,24
25	25,2	1347,1	2694,2

Maximum spacing (article 5.8.2.7):

$0,125*f'_c = 0,125*40,0$
$= 5,0$ MPa

$$v_u = \frac{V_u - \phi . V_p}{\phi . b_v . d_v} \qquad \text{Eqn. 5,8,2,9-1}$$

$$= \frac{638,473-0,9*0,0}{0,9*150,0*939,757}$$

$= 5,03261$ MPa

$v_u >= 0,125*f'_c$
therefore, $s_{max} = 0,4*d_v <= 304,8$ mm
$= 0,4*939,757$
$= 375,903$ mm

hence maximum permissible spacing = 304,8 mm

Longitudinal Reinforcement - Article 5.8.3.5

Non-prestressed reinforcing steel:
Area on flexural tensile side $A_s = 1060,09$ mm²
yield stress $f_y = 500,0$ MPa
tensile capacity $= 1060,09*500,0$
$= 530,046$ kN

Prestressing steel:
Area on flexural tensile side $A_{ps} = 987,1$ mm²
tensile strength $f_{pu} = 1900,0$ MPa
allowance for development $= 1,0$
tensile capacity $= 1,0*987,1*1900,0$
$= 1875,49$ kN

Total tensile capacity = 530,046+1875,49
= 2405,54 kN

Equation 5,8,3,5-1 right hand side:

$M_u/(d_v . \phi) = 968,474/(0,93975*1,0)$
$= 1030,56$ kN
$0,5*N_u/\phi = 0,5*315,575/ 0,75$
$= 210,383$ kN
$V_u/\phi - 0,5*V_s - V_p = 638,473/0,9 - 0,5*527,168 - 0,0$
$= 445,83$
$* \cot(\theta) = 445,83*1,5151$
$= 675,479$ kN

where V_s is taken from equation 5,8,3,3-4 but not greater than V_u/ϕ

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V_u / φ = 638,473/0,9
= 709,414 kN
$A_v / s \ . \ f_y \ . d_v \ . \cot(\theta)$ = 7,4E-4*500,0*0,93975*1,5151
= 527,168 kN

1030,56+210,383+675,479 = 1916,42 kN

which is less than 2405,54, so Equation 1 is satisfied, therefore longitudinal
reinforcement is adequate

Summary of link requirements along the beam:
Preferred size of links: 11,3 mm Bar #: 10

dimension from left beam end m	A_v / s mm²/mm	Spacing in mm with preferred link size[1]
0,0	1,69282	118,485
0,25	0,45260	300,0
0,66044	0,43420	300,0
0,762	0,44317	300,0
1,06	0,46836	300,0
1,42244	0,49312	300,0
1,7	0,74049	270,867
1,84241	0,74427	269,491
2,56895	0,73936	271,281
2,60441	0,60745	300,0
3,15	0,57201	300,0
3,33095	0,54062	300,0
4,6	0,51659	300,0
5,10171	0,40989	300,0
5,86371	0,28387	300,0
6,05	0,26074	300,0
7,5	0,15743	300,0
8,95	0,16534	300,0
9,13629	0,16496	300,0
9,89829	0,18906	300,0
10,4	0,18626	300,0
11,6691	0,21224	300,0
11,85	0,25760	300,0
12,3956	0,21158	300,0
12,4311	0,30897	300,0
13,2576	0,22710	300,0
13,3	0,20336	300,0
13,5776	0,15743	300,0
13,94	0,15743	300,0
14,238	0,15743	300,0
14,3396	0,15743	300,0
14,75	0,15743	300,0
15,0	1,00463	199,65

[1] Based on 2 legs (double this value if using 4 legs)

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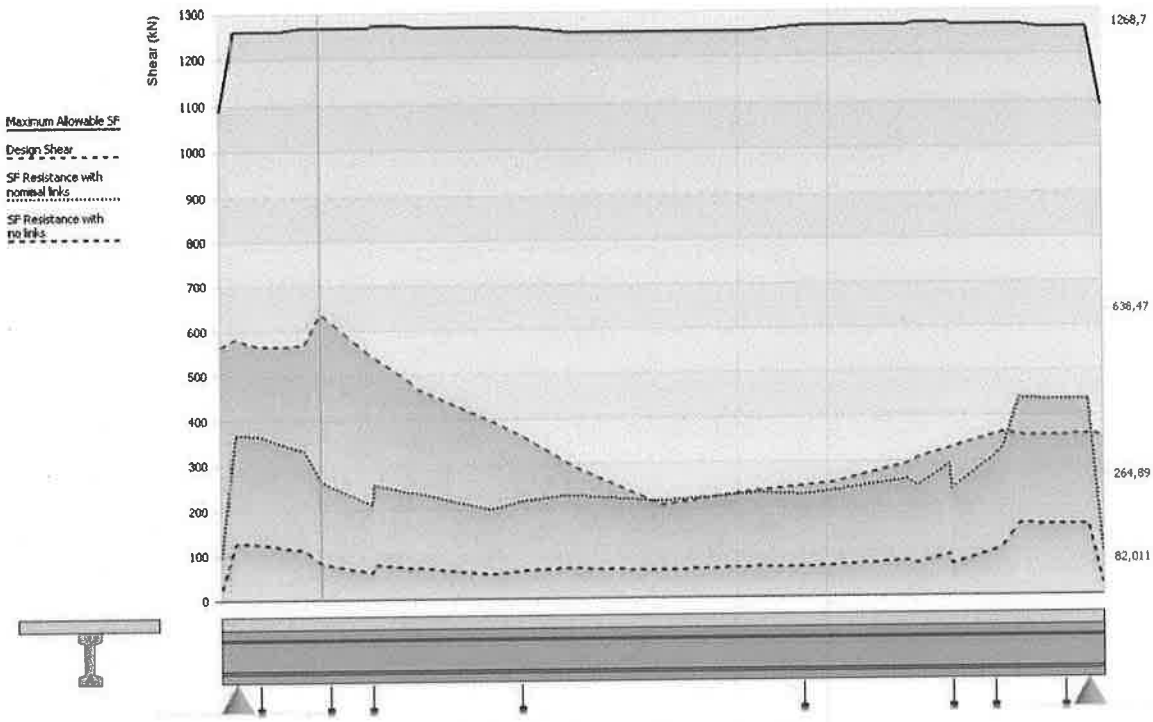

Fz_{total} = 638,47 kN.m < 1268,70 kN

Armadura de Pretensado

El pretensado estará compuesto por torones de diámetro 12,70 mm, 14 torones en el tramo central del vano y 6 torones en las extremidades escalonados dos a dos. En acuerdo con el artículo 5.9.3 de la AASHTO LRFD 7 será aplicada en los torones la tensión:

$$f_{po} = 0.75 f_{pu} = 0.75 \cdot 190 = 142.50 \ kN/cm^2$$

Tendons

x-y coordinates mm		area mm²	Property ref	K	d_b mm	type
-150,0	55,0	98,71	MP3	60	12,7	Full stress
-30,0	55,0	98,71	MP3	60	12,7	Full stress
30,0	55,0	98,71	MP3	60	12,7	Full stress
150,0	55,0	98,71	MP3	60	12,7	Full stress
-150,0	105,0	98,71	MP3	60	12,7	Full stress
-90,0	105,0	98,71	MP3	60	12,7	Debonded
-30,0	105,0	98,71	MP3	60	12,7	Debonded
30,0	105,0	98,71	MP3	60	12,7	Debonded
90,0	105,0	98,71	MP3	60	12,7	Debonded
150,0	105,0	98,71	MP3	60	12,7	Full stress
-30,0	155,0	98,71	MP3	60	12,7	Full stress
30,0	155,0	98,71	MP3	60	12,7	Full stress
0,0	205,0	98,71	MP3	60	12,7	Debonded
0,0	255,0	98,71	MP3	60	12,7	Debonded

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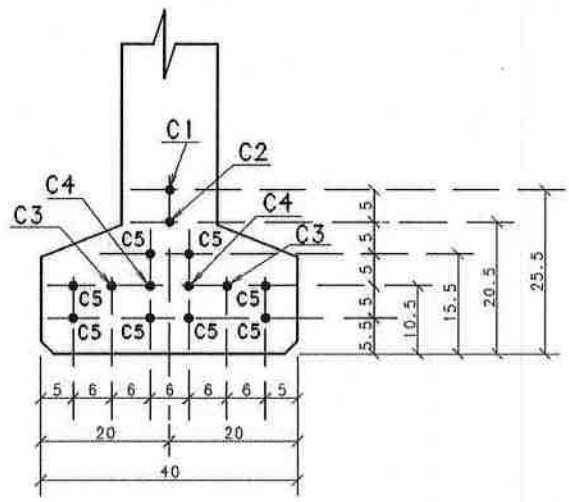
Debonded Tendons

x-y coordinates mm		distance from left end (m) start	end
-90,0	105,0	1,84241	13,1576
-30,0	105,0	0,66044	14,3396
30,0	105,0	0,66044	14,3396
90,0	105,0	1,84241	13,1576
0,0	205,0	2,56895	12,4311
0,0	255,0	5,10171	9,89829

PRETENSADO
ESC. 1:10



Armadura Pasiva

Según los esfuerzos solicitados, el software calculó el área de acero de armadura pasiva y su distribución y disposición en la viga, conforme la tabla abajo.

Reinforcement

x-y coordinates mm		diameter mm	Property ref	Start m	End m	Length m
-160,0	122,919	8,0	MP2	0,0	15,0	15,0
160,0	122,919	8,0	MP2	0,0	15,0	15,0
-35,0	172,919	8,0	MP2	0,0	15,0	15,0
35,0	172,919	8,0	MP2	0,0	15,0	15,0
-160,0	777,081	8,0	MP2	0,0	15,0	15,0
160,0	777,081	8,0	MP2	0,0	15,0	15,0
-35,0	727,081	8,0	MP2	0,0	15,0	15,0
35,0	727,081	8,0	MP2	0,0	15,0	15,0
-160,0	860,0	10,0	MP2	0,0	15,0	15,0
160,0	860,0	10,0	MP2	0,0	15,0	15,0
-35,0	860,0	10,0	MP2	0,0	15,0	15,0
35,0	860,0	10,0	MP2	0,0	15,0	15,0
-160,0	40,0	12,5	MP2	0,0	15,0	15,0
-106,67	40,0	12,5	MP2	0,0	15,0	15,0
-53,333	40,0	12,5	MP2	0,0	15,0	15,0
1,4E-14	40,0	12,5	MP2	0,0	15,0	15,0
53,3333	40,0	12,5	MP2	0,0	15,0	15,0
106,667	40,0	12,5	MP2	0,0	15,0	15,0
160,0	40,0	12,5	MP2	0,0	15,0	15,0

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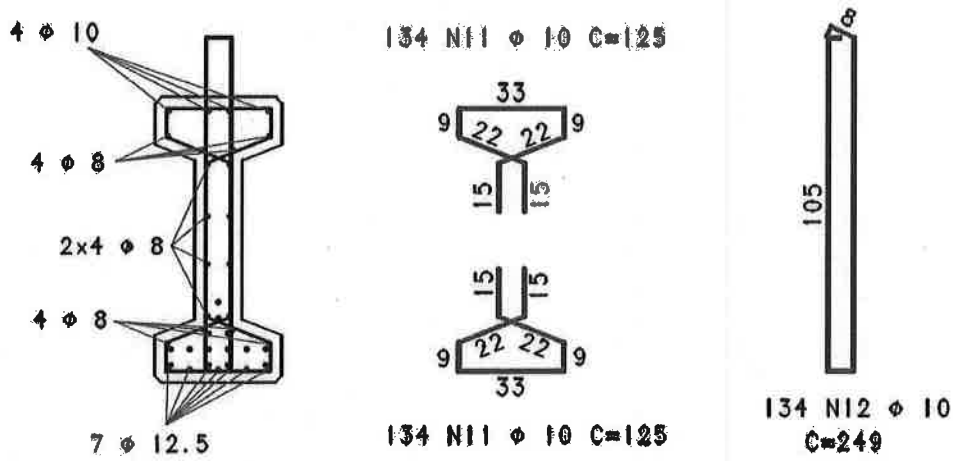
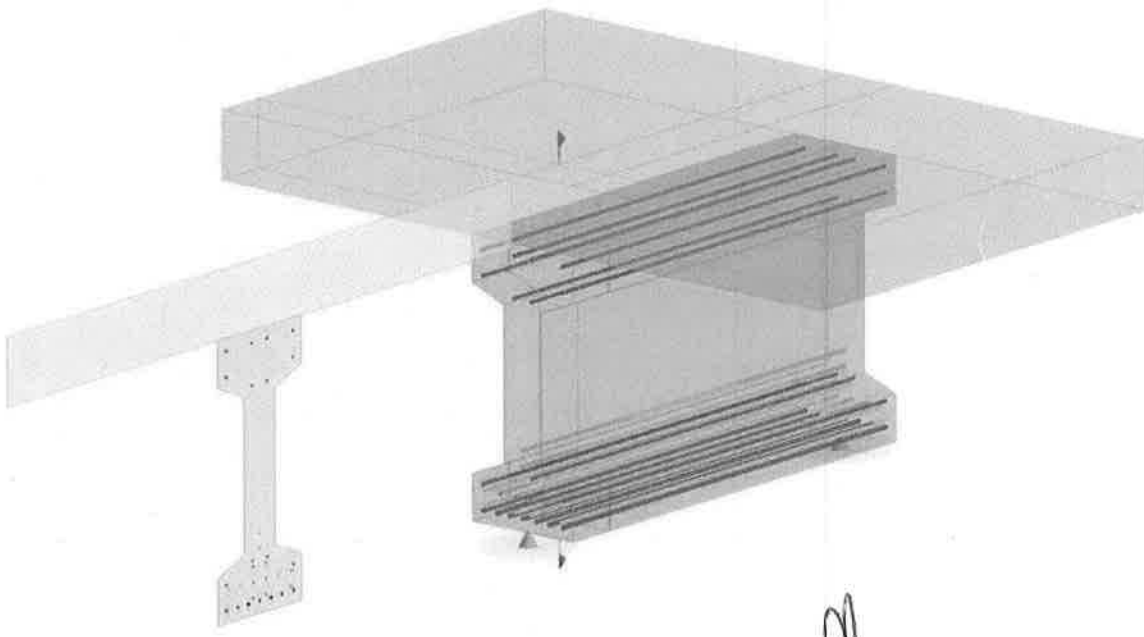
Las armaduras pasivas serán dispuestas de acuerdo con la figura siguiente mostrada en proyecto.



La figura abajo muestra la disposición de las armaduras pasivas y de pretensado.



Sin más, saludos cordiales,

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MEMORIA DE CÁLCULO DE VIGAS LONGITUDINALES. PUENTE L= 25,00 m

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MEMORIA DE CALCULO ESTRUCTURAL

Llamado MOPC Nº 188/2019

Lote Nº 3: San Vicente - Maracaná

Convenio Modificatorio Nº 1 - Sustitución de Vigas

Puente de Hº Aº L = 25M – PROG. 2 + 824,0

Localidad: SAN VICENTE - MARACANÁ

Cliente: SOTEC S.A.

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1) Introducción

Esta Memoria de Cálculo analiza los cambios introducidos en la Memoria de Cálculo original, resultante de sustitución de las vigas con postesado y cables curvos por nuevas vigas pretensadas y torones rectos. Para facilitar la ejecución también se eliminaron las vigas transversales, cambiando así las dimensiones de las prelosas previas, debido a la reducción en el apoyo de la viga.

Las diferencias básicas entre los dos proyectos son el peso proprio y la altura de las vigas. Los cambios en el tamaño de las prelosas no introdujeron ninguna alteración estructural, pues se mantuvieran todas las armaduras y espesor de hormigón del proyecto original.

Además, se considera, según las informaciones recibidas, que el proyecto original ya estaba aprobado por los órganos viales del Paraguay. Por esto, intentando simplificar las tramitaciones futuras, se buscó cambiar lo mínimo posible en los elementos estructurales del proyecto original. Solo fue cambiado las vigas pretensadas y la largura de las prelosas para conseguir el apoyo necesario.

Se mantuvo el tablero, los estribos, las pilas, las barreras, las barandas, las juntas de dilatación y las fundaciones.

2) Especificaciones de los materiales

Hormigón

Vigas - C40:

f'_c = 40 MPa;

Módulo de elasticidad - Ec = 37,434 kN/mm²

Coeficiente de Poisson - v = 0,2

Módulo de cizalla – G = 15,597 kN/mm2

Módulo de ruptura = -3,9857 MPa

Coeficiente de dilatación térmica = 1,08 x 10^{-5}/°C

Densidad = 25 kN/m³

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Losas - C30:

f'_c = 30 MPa;

Módulo de elasticidad - Ec = 34,044 kN/mm²

Coeficiente de Poisson - v = 0,2

Módulo de cizalla – G = 14,185 kN/mm2

Módulo de ruptura = -3,4517 MPa

Coeficiente de dilatación térmica = 1,08 x 10^{-5}/°C

Densidad = 25 kN/m³

Acero reforzado

CA50

f_y = 500 MPa;

Módulo de elasticidad - Es = 200 kN/mm²

Densidad = 77 kN/m³

Acero de pretensado

CP 190 RB

f_{pu} = 1900 MPa

Módulo de elasticidad - Ep = 197 kN/mm²

Límite elástico - f_{py} (% f_{pu}) = 90%

Fuerza de pretensado inicial (% f_{pu}) = 75%

Densidad = 77 kN/m³

3) Análisis estructural

El procesamiento y modelado de la estructura se elaboraron con el software *Autodesk® Structural Bridge Design 2019*, utilizando métodos de análisis de elementos finitos.

4) Geometría de las vigas pretensadas

La nueva viga pretensada tiene la sección abajo cuando será prefabricada:

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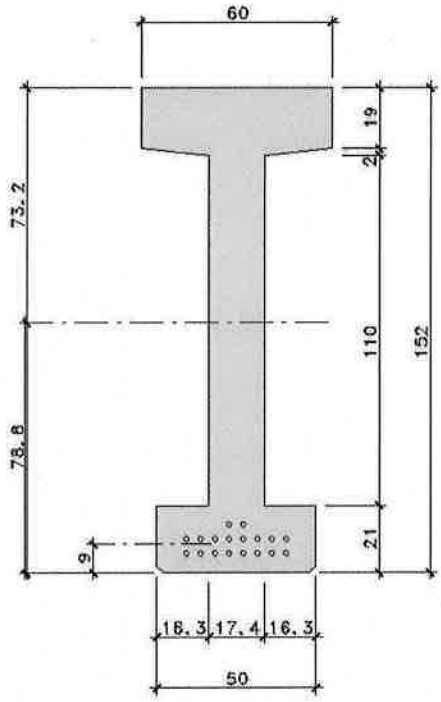


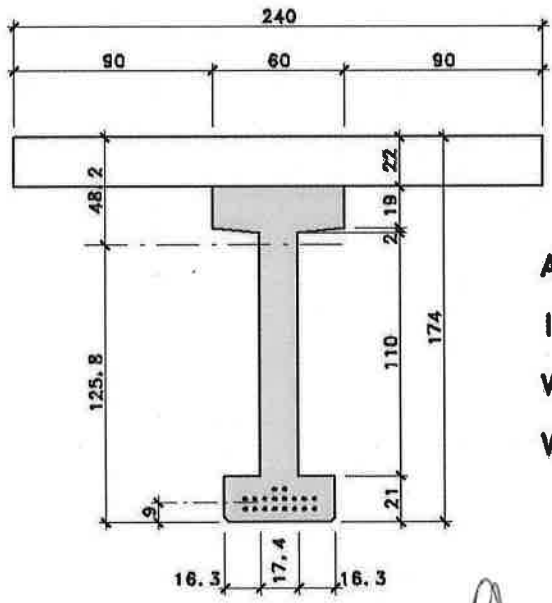
Ac = 4177 cm2

Ic = 11737920 cm4

Ws = 160354 cm3

Wi = 148958 cm3

Cuando pasa a hacer parte del puente la sección tiene las propiedades abajo:



Ac = 9457 cm2

Ic = 28486580 cm4

Ws = 591008 cm3

Wi = 226443 cm3

Su peso total prefabricada será

0,41774 x 2,5 x 24,95 = 26,06tf

El puente consta de cuatro vigas pretensadas, de acuerdo con la siguiente figura:

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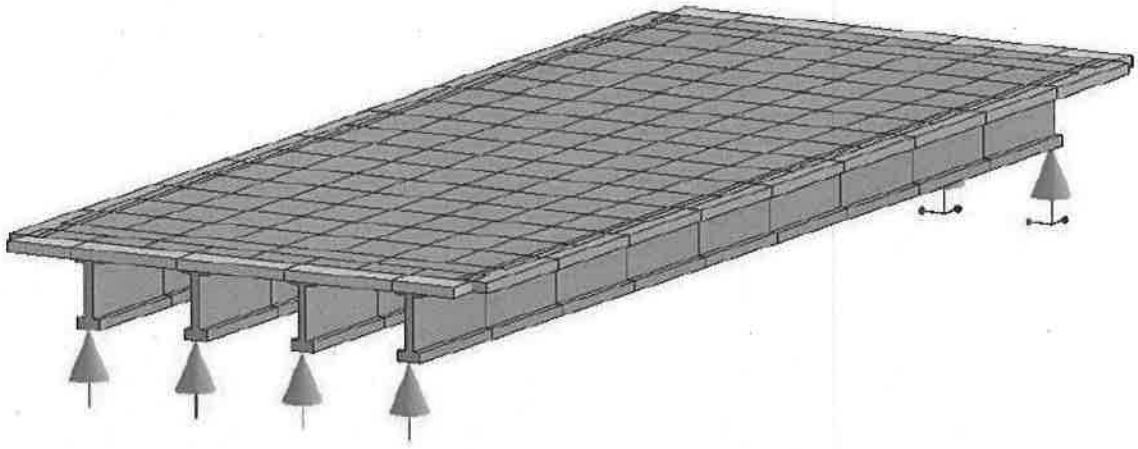

5) Normas utilizadas

La norma utilizada para el proyecto de las vigas cambiadas es la misma del diseño original o sea la AASHTO LRFD 7 siendo que para los vehículos tipo se mantuvo la consideración del HL-93 y Carga Especial Bitren del Ministerio de Obras Públicas y Comunicaciones.

6) Solicitaciones de carga

Cargas muertas:

Cargas muertas consideradas como el peso propio de la estructura, y cargas posteriores como el peso del pavimento, new jersey y riostras.

Peso proprio de la viga: 0,41774 x 2,5 x 24,95 = 26,06 tf

Barandas: 0,08 tf/m

Veredas: 0,25x2,4 = 0,6 tf/m²

Capa de rodadura

Asfalto con 7 cm: 0,07 x 2,4 = 0,17 tf/m²

Eventual repavimentación: 0,2 tf/m²

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Carga viva peatonal:

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Se aplica una carga peatonal de 3,59 kN/m² a todas las veredas de más de 600 mm de ancho considerada simultáneamente con la carga vehicular de diseño en el carril (design lane load) para vehículos.

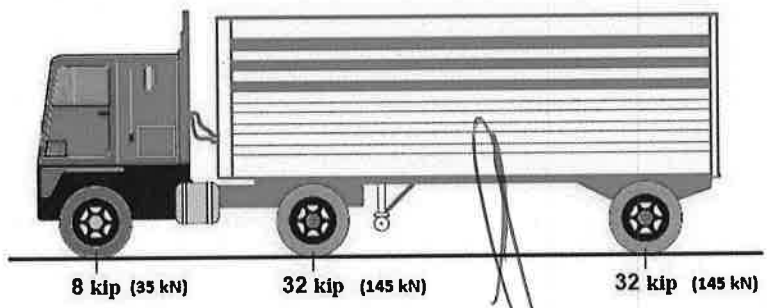
Carga viva vehicular:

Los valores de las solicitaciones debidas a las cargas móviles vehiculares sobre las vigas fueron calculados de acuerdo con el código AASHTO LRFD 7 para la colocación de carga HL-93.

HL-93 es un tipo de carga vehicular teórico propuesto por la AASHTO em 1993. Se utiliza como la carga de diseño de estructuras viales y es una combinación de tres diferentes cargas.

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).



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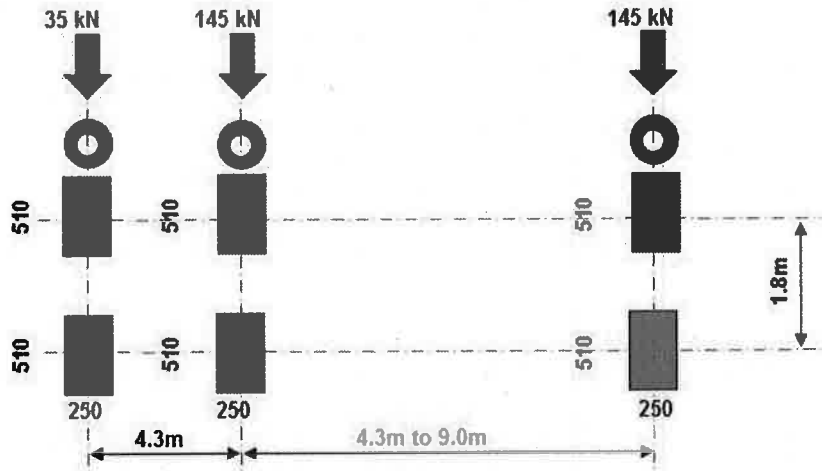
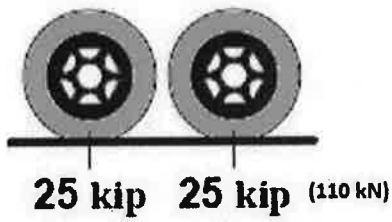
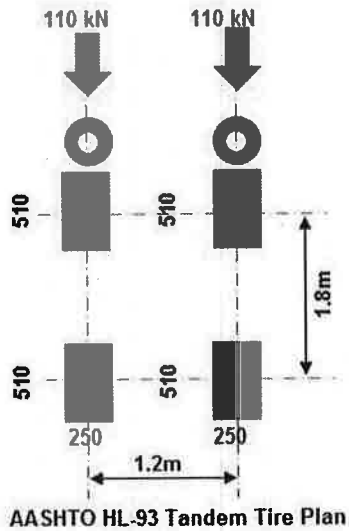


AASHTO HL-93 Truck Tire Plan

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).





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).

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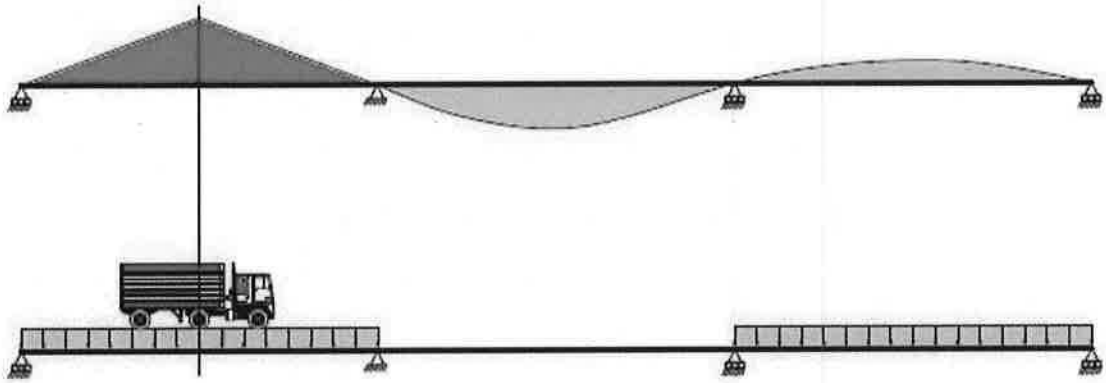
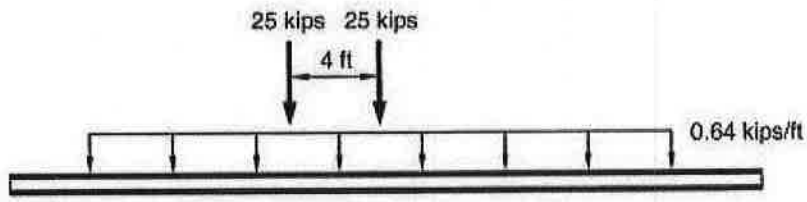
El diseño de los vehículos y cargas de LL deben aplicarse de tal manera que se obtiene un efecto de fuerza extrema para el cálculo. En caso de múltiples carriles, se considerarán factores múltiples carriles descritos en el código.

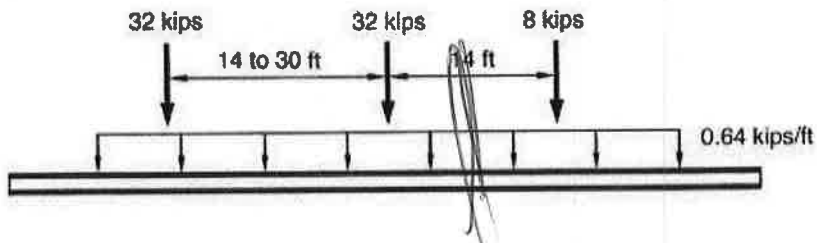


Factor de impacto o de la asignación de carga dinámica se aplica sólo para los camiones y tándems, no para la carga de carril. La fuerza máxima de cálculo se toma como el mayor de los siguientes:

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



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



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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.

- **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

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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	

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.

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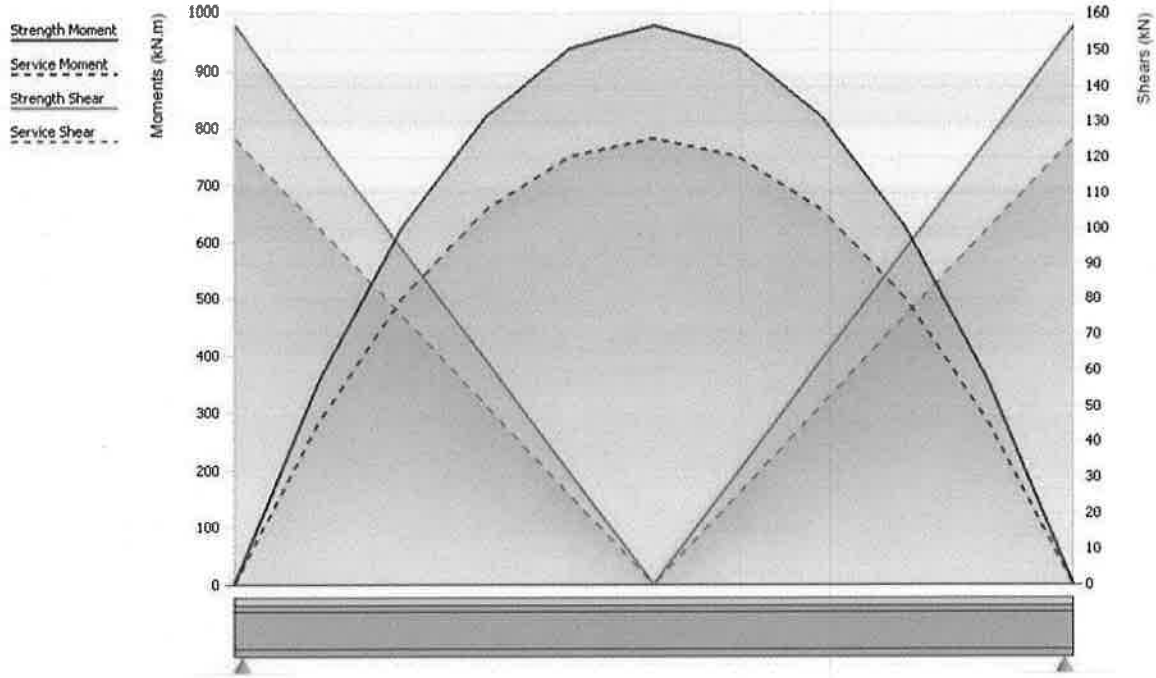
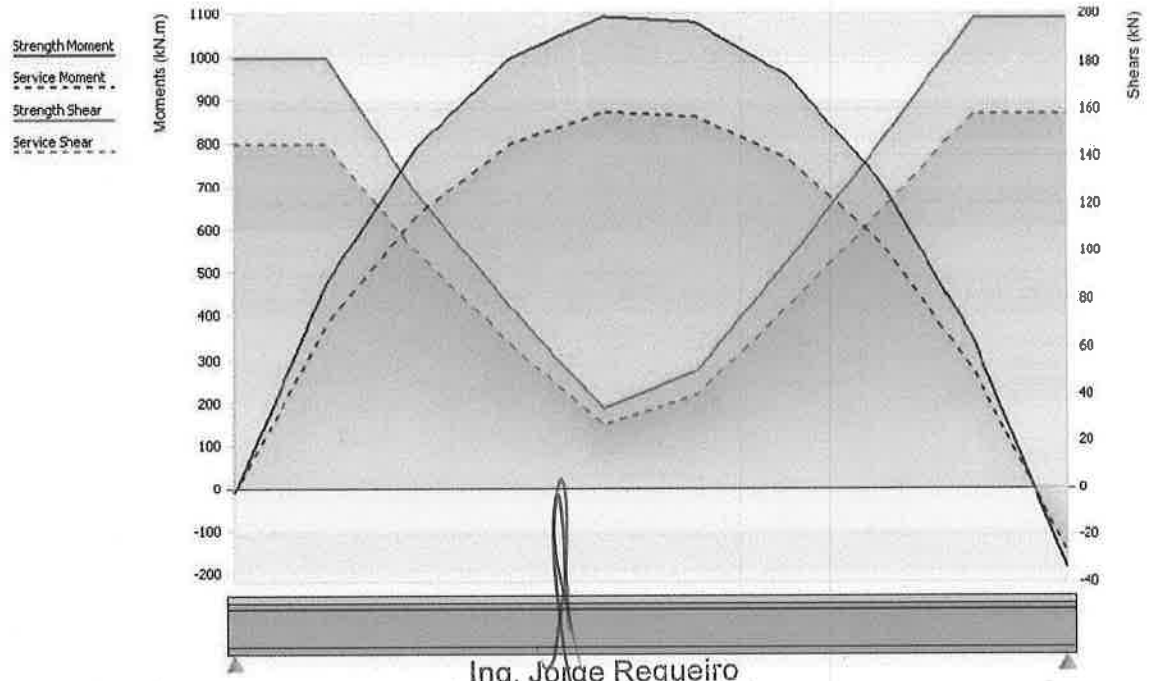
Esfuerzos en las vigas pretensadas

Esfuerzos debidos al izaje de la viga



Esfuerzos debidos a las cargas muertas



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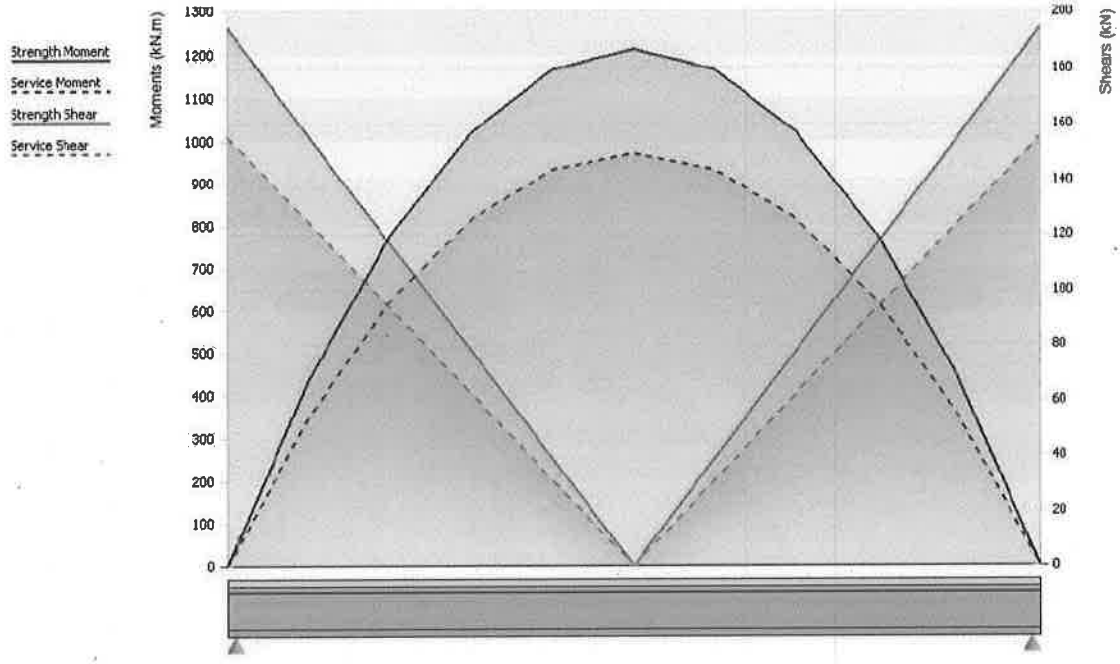
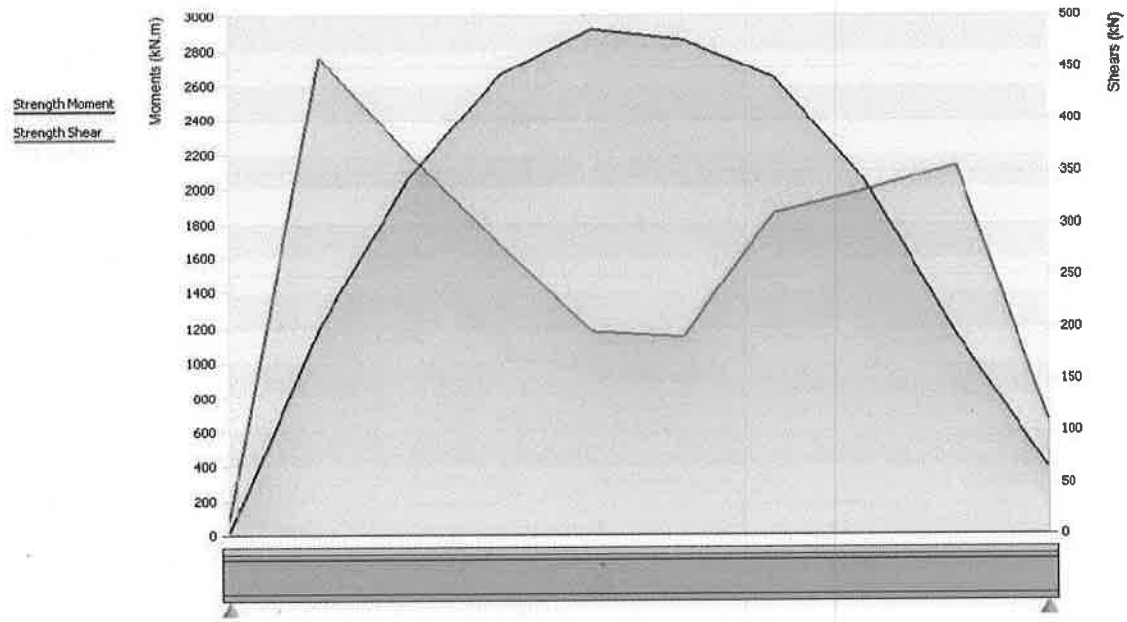
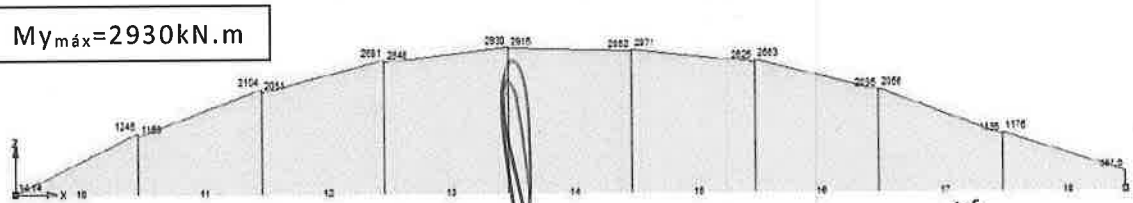
Esfuerzos debidos a etapa de construcción 1A



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



$My_{máx}$=2930kN.m



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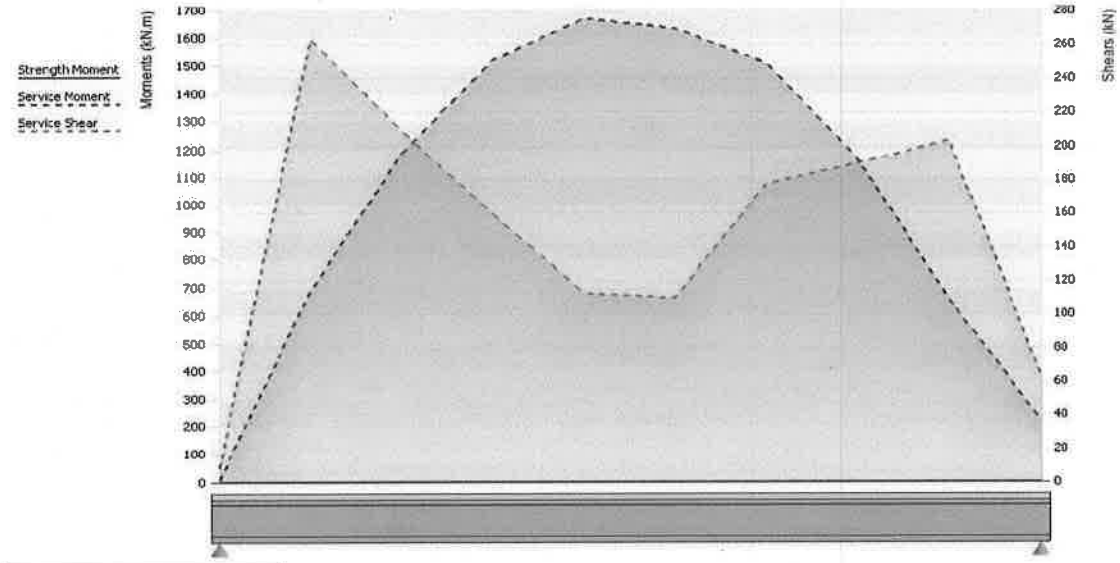
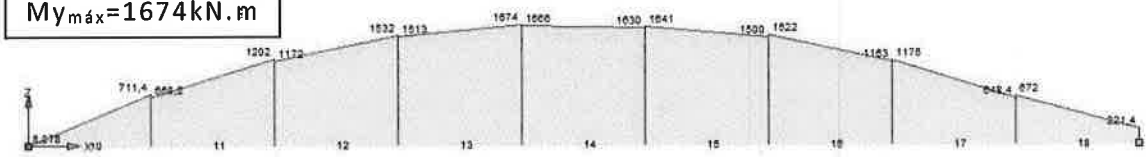
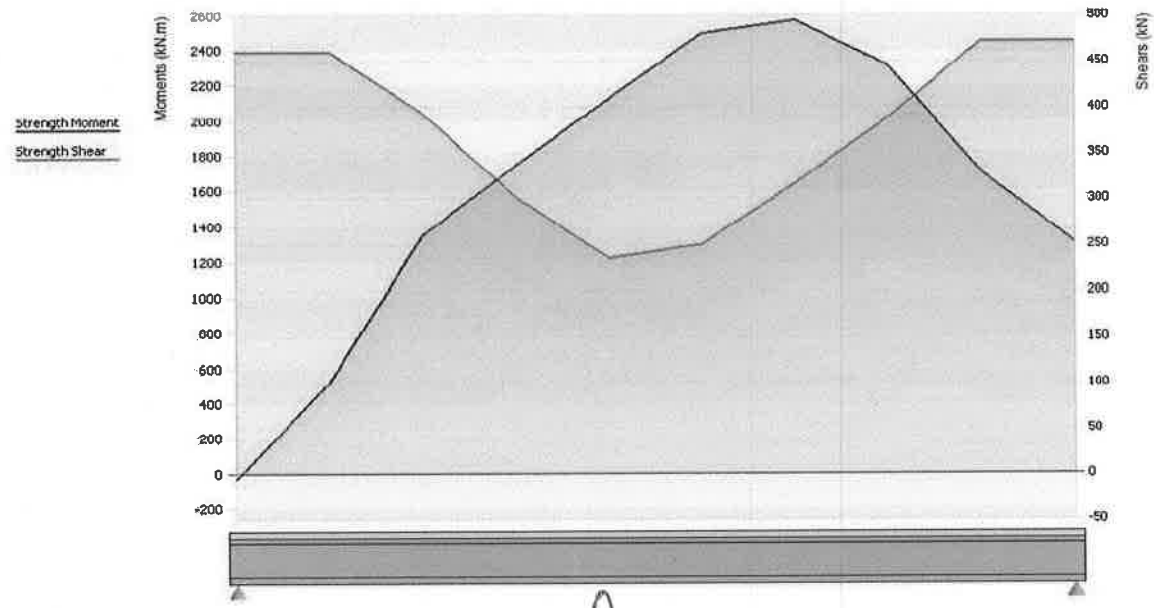
Envolvente de Momentos My+ en estado de límite de servicio I (kN.m)

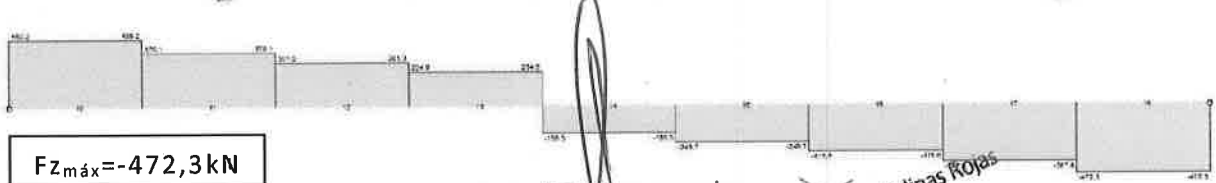


$My_{máx}$=1674kN.m



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





$Fz_{máx}$=-472,3kN

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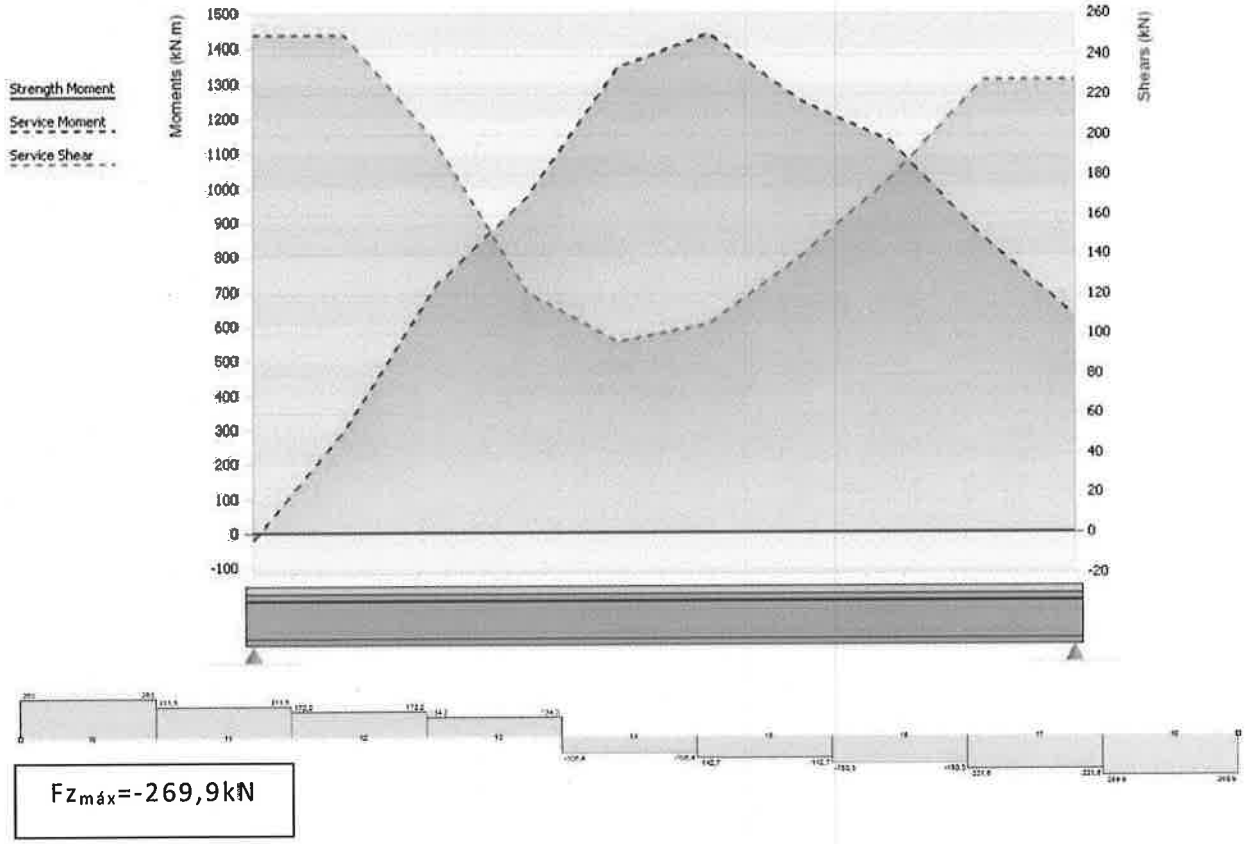
Envolvente de Cortantes Fz+ en estado de límite de servicio I (kN)

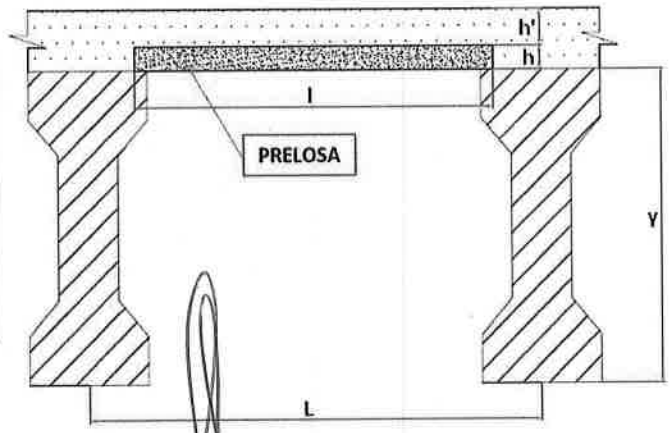


$Fz_{máx}$=-269,9kN

9) Dimensionamiento de las estructuras

Prelosas

Entrada de dados	
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	2500,00 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

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- 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

Verificación prelosas premoldeadas – Montaje y hormigonado

Momento solicitante:

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	2500,00 cm

Dimensionamiento de Prelosa										
Cargas			Verificación de Flexión			Verificación de Cortante				
g1	g2	p - acidental	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	

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

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

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

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

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

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

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
```

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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)

$$\varepsilon_{cl} = -0,0026$$

Tension-controlled strain limit in the extreme tension steel (5,7,2,1)

$$\varepsilon_{tl} = -0,005$$

Net tensile strain in extreme steel at nominal resistance

$$\varepsilon_t = -0,00421$$

nonprestressed section:

$$0,75 <= \varphi = 0,75 + 0,15\left[\frac{eps_t - eps_{cl}}{eps_{tl} - eps_{cl}}\right] <= 0,9 \quad \text{(Eq.: 5,5,4,2,1-2)}$$

$$0,75 + 0,15\left[\frac{-0,0042 - -0,0026}{-0,005 - -0,0026}\right]$$

$$= 0,85013$$

$$\varphi = 0,85013$$

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
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.

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

For Monolithic sections with no prestress,

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

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

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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!

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

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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

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.

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

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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

```
      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.

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

For Monolithic sections with no prestress,

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

$\gamma_1 = 1,6$

$\gamma_3 = 0,67$ for A615, Grade 60 reinforcement
$0,75$ for A706, Grade 60 reinforcement

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

$S_c = 8516136,2$ 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 $1,33M_u$

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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.

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

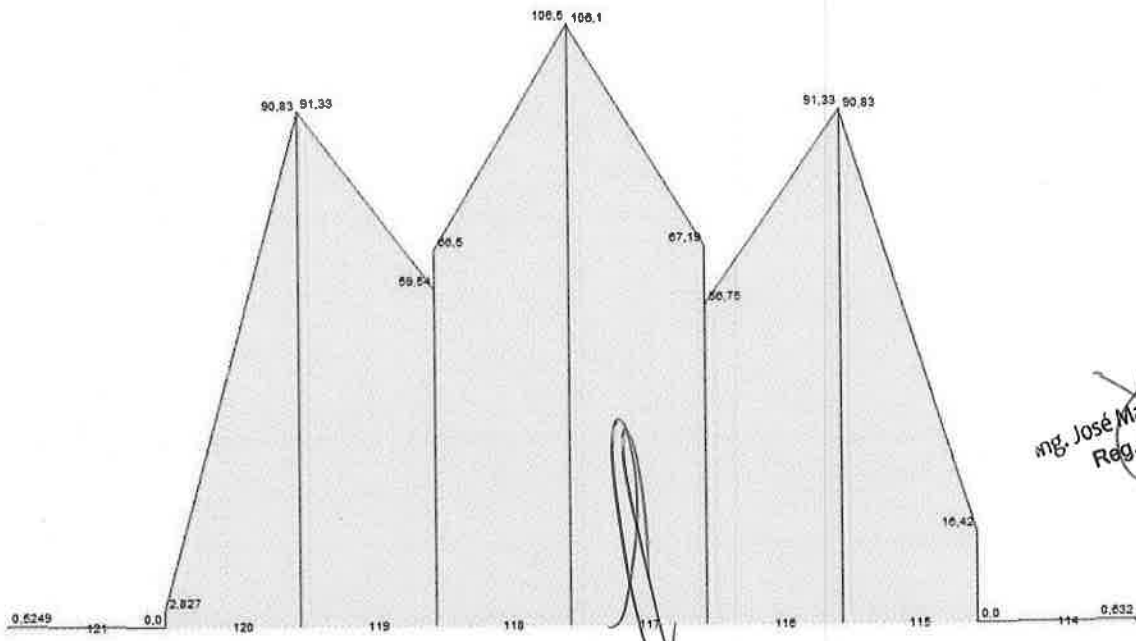
Momento solicitante positivo

Momento debido al peso proprio:

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

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

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



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$$M_{max\ total\ (pos)} = 106,50\ kN.m$$

Momento en una losa:

$$M_{max\ (pos)} = \frac{106,50}{25/10} = 42,60\ kN.m/m$$

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

$$M_{k\ (pos)} = 3,30 \times 1,25 + 42,60 = 46,73\ kN.m$$

$$\therefore M_{r\ (pos)} = 81,0352\ kN.m > M_{k\ (pos)} = 46,73\ kN.m$$

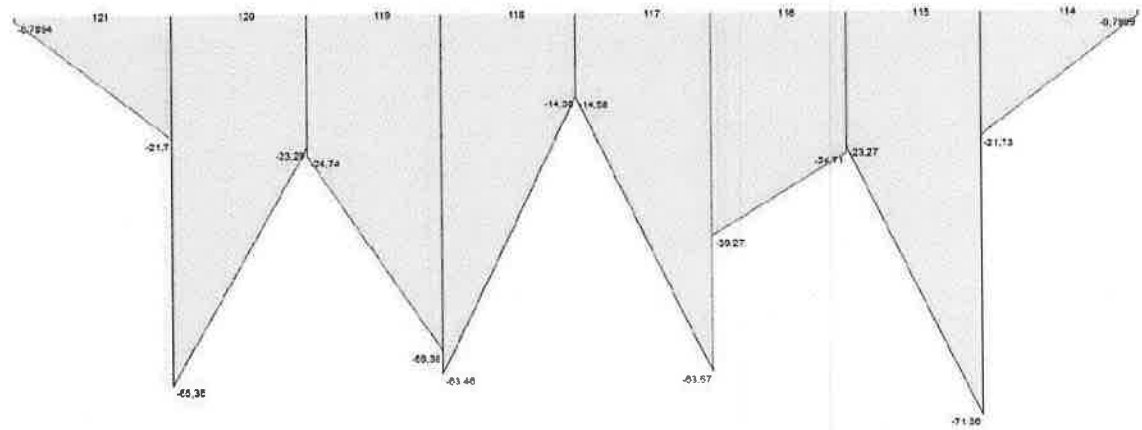
Verificación OK!

Momento solicitante negativo

Momento debido al peso proprio para losa en voladizo:

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

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



$$M_{max\ total\ (neg)} = 21,73\ kN.m$$

Momento en una losa:

$$M_{max\ (neg)} = \frac{21,73}{25/10} = 8,69\ kN.m/m$$

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

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$$M_{k\,(neg)} = 5{,}39 \times 1{,}25 + 8{,}69 = \mathbf{15{,}43\ kN.m}$$

$$\therefore M_{r\,(neg)} = \mathbf{62{,}78\ kN.m} > M_{k\,(neg)} = \mathbf{15{,}43\ kN.m}$$

Verificación OK!

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

$$M_{max\ total\,(neg)} = \mathbf{71{,}36\ kN.m}$$

Momento en una losa:

$$M_{max\,(neg)} = \frac{71{,}36}{25/10} = \mathbf{28{,}54\ kN.m/m}$$

$$\therefore M_{r\,(neg)} = \mathbf{62{,}78\ kN.m} > M_{max\,(neg)} = \mathbf{28{,}54\ kN.m}$$

Verificación OK!

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 25 = 12,5 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

18 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
60,0	8	1900,0	140,0	75,0	1596,0
105,0	8	1900,0	140,0	75,0	1596,0
150,0	2	1900,0	140,0	75,0	399,0
TOTAL	18				3591,0

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
60,0	8	1425,0	1710,0	0,0	140,0	1596,0	95,76
105,0	8	1425,0	1710,0	0,0	140,0	1596,0	167,58
150,0	2	1425,0	1710,0	0,0	140,0	399,0	59,85
TOTAL	18					3591,0	323,19

Moment about the centroid of the precast beam:

M_r = 323,19-(3591,0*0,787224)
= -2503,733 kN.m

Corresponding stresses:

top stress = 3591/418,14+-2503,73/1,6052E8
= 8,5880327+-15,59791
= -7,00987 MPa

bottom stress = 3591/418,14+-2503,73/-1,494E8
= 8,5880327+16,756906
= 25,3449 MPa

Self weight moment:

c.s.a. = 4,181E5 mm²
density = 25,0 kN/m³
self weight = 4,181E5*25,0
= 10,4535 kN/m
beam length = 25,0 m
distance = 12,5 m
M_{sw} = 0,5*10,4535*12,5*(25,0-12,5)
= 816,68 kN.m

Corresponding stresses:

top stress = 816,68/1,6052E8
= 5,0878 MPa

bottom stress = 816,68/-1,494E8
= -5,4658 MPa

ELASTIC SHORTENING LOSSES - Article 5.9.5.2.3a

concrete stresses at mid-span:
stress at top of precast beam = -1,9221 MPa
stress at bottom of precast beam = 19,8791 MPa
depth of precast beam = 1520,0 mm

height mm	No of tendons	conc stress MPa	E_p / E_{ci}	tendon force kN	tendon moment kN.m
60,0	8	19,01852	5,2626	112,09727	6,7258363
105,0	8	18,37309	5,2626	108,29304	11,370769
150,0	2	17,72766	5,2626	26,122202	3,9183303
TOTAL	18			246,51251	22,014936

Moment about the centroid of the precast beam:

M_{ed} = 22,014936-(246,51251*0,787224)
= -172,0457 kN.m

hence top stress = -1,922-246,513/418,14--172,046/1,6052E8
= -1,922-0,5895454--1,071821
= -1,4398 MPa

bottom stress = 19,879-246,513/418,14--172,046/-1,494E8
= 19,879-0,5895454-1,1514623
= 18,1381 MPa

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

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top stress = -1,47795 MPa
bottom stress = 18,2758 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} = 2520,0 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 = 4,0 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: $k_{tdc,f,i} = \frac{t_f - t_i}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_i}$

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

$= 0,7645$

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 = 4,18E5 mm²
Perimeter of section: p = 4853,9 mm
Volume to surface ratio: $V/S = A_g/p$ = 86,146 mm
Factor for the effect of the volume: k_s = 1,45-0,0051*(V/S) = 1,0107
Creep coefficient at final time:

$\psi_b(t_f, t_i) = 1,9*k_s . k_{hc} . k_f . k_{tdc,f,i} . t_i^{-0,118}$
$= 1,9*1,0107*0,96*2,7341*0,7645*4,0^{-0,118}$
$= 3,272$

Inertia of the gross concrete section: I_g = 1,18E11 mm⁴
Eccentricity of prestressing force: e_{pg} = -697,18 mm
Transformed section coefficient:

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

$$= \frac{1}{197,0 \; 2520,0 \left[4,18E5*-697,2^2\right]}$$

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$$1 + \frac{}{37,434}*\frac{}{4,18E5}\left[1+\frac{}{1,2E11}\right]*(1+0,7*3,272)$$

$$= 0,77840$$

Time factor for creep at deck placement:

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

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

$$= 0,401$$

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: $\varepsilon_{bid} = k_s \cdot k_{hs} \cdot k_f \cdot k_{tds,d,i}$

*0,00048

1,009*0,95*0,73*0,41*0,00048

$= 1,393E-4$

Loss of prestress $= \varepsilon_{bid} \cdot E_p \cdot K_{id}$
$= 1,39E-4*197,0*0,87741$
$= 24,075$ MPa
$= \Delta f_{pSR}$

Total shrinkage loss of force $= \Delta f_{pSR} \cdot A_{ps}$
$= 24,0751*2520,0$
$= 60,6692$ kN

Corresponding gain of moment (about $y = 0$) $= 60,6692*0,09005$
$= 5,46303$ kN.m

Moment about the centroid of the composite beam:

$M_{sh} = 5,46303-$

(60,6692*0,78722)

$= -42,297$ kN.m

Corresponding stresses:

top stress $= 60,6692/418140+-42,2973/1,605E8$
$= 0,1450931+-0,263506$
$= -0,118413$ MPa

bottom stress $= 60,6692/418140+-42,2973/-1,494E8$
$= 0,1450931+0,2830858$
$= 0,428179$ MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:

$\psi_b(t_d, t_i) = 1,9 k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,d,i} \cdot t_i^{-0,118}$

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$= 1,9*1,0*0,96*0,7351*0,4019*4,0^{-0,118}$
$= 0,4575$

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,921 MPa
at bottom of precast beam = 19,878 MPa
at center of gravity of prestress: f_{cgp} = 18,5866 MPa

$$\text{Loss of prestress} = \frac{E_p}{E_{ci}}.f_{cgp}.\psi_b(t_d, t_i).K_{id}$$
$$= \frac{197,0}{37,434}*18,587*0,4575*0,8893$$
$$= 39,8037 \text{ MPa} = \Delta f_{pCR}$$

Total creep loss of force $= \Delta f_{pCR}.A_{ps}$
$= 39,8037*2520,0$
$= 100,305$ kN

Corresponding loss of moment $= 100,305*0,09005$
$= 9,03212$ kN.m
(about $y = 0$)

Moment about the centroid of the composite beam:
$M_{cr} = 9,03212-(100,305*0,78722)$
$= -69,931$ kN.m

Corresponding stresses:
top stress $= 100,305/4,181E5+-69,931/1,60517E8$
$= 0,23988+-0,4356$
$= -0,1957$ MPa

bottom stress $= 100,305/4,181E5+-69,931/-1,4941E8$
$= 0,23988+0,46803$
$= 0,70791$ MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.2c

Stress in tendons after transfer: $f_{pt} = 1334,91$ MPa
Yield strength of prestressing steel: $f_{py} = 1710,0$ MPa
Time between tensioning and deck placement: $t = t_s - t_i = 28,0- 4,0 = 24,0$day
Factor for relaxation steel type: $K'_L = 45,0$ (low relaxation)

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

$$= \frac{1334,9}{45,0}*\frac{\log(24*24,0)}{\log(24*4,0)}*\left[\frac{1334,9}{1710,0}-0,55\right]\left[1- \frac{3*(24,075+39,804)}{1334,9}\right]*0,87$$

$= 7,15995$ MPa

$= \Delta f_{pR1}$

Total relaxation loss of force $= \Delta f_{pR1}.A_{ps}$
$= 7,1599*2520,0$
$= 18,043074$ kN

Corresponding loss of moment $= 18,043*0,09$
$= 1,6247093$ kN.m
(about $y = 0$)

Moment about the centroid of the composite beam:

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M_{rx} = 1,62471-(18,0431*0,7872)
= -12,579 kN.m

Corresponding stresses:
top stress = 18,0431/418140+-12,5792/1,605E8
= 0,0431508+-0,078367
= -0,0352161 MPa

bottom stress = 18,0431/418140+-12,5792/-1,494E8
= 0,0431508+0,0841899
= 0,127341 MPa

Erection of beam Loading

Bending moment from erection loadcase at current span location:
$M_{Applied}$ = 816,67969 kN.m
Corresponding stresses:
top stress = 816,67969/1,6052E8
= 5,0878 MPa
bottom stress = 816,67969/-1,494E8
= -5,4658 MPa

Remove the dead load applied for transfer calculations
M_{sw} = -816,68 kN.m
Corresponding stresses:
top stress = -816,68/1,6052E8
= -5,0878 MPa
bottom stress = -816,68/-1,494E8
= 5,46585 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 * λ * $\sqrt{f'_c}$
= -0,5 * 1,0 * √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	Cast In Place bottom	Precast top	Precast bottom
Prestress	3591,0	-2503,7			-7,0099	25,3449
Self Weight		816,68			5,0878	-5,4658
Prestress + Self Wt.					-1,9221	19,8791
Elastic Def	-227,02	158,437			0,44412	-1,6033
TRANSFER	3363,98	-1528,6			-1,478	18,2758
Erection		0,0			0,0	0,0
TOTAL PERMANENT					-1,478	18,2758

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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 = 25,6091 kN/mm²

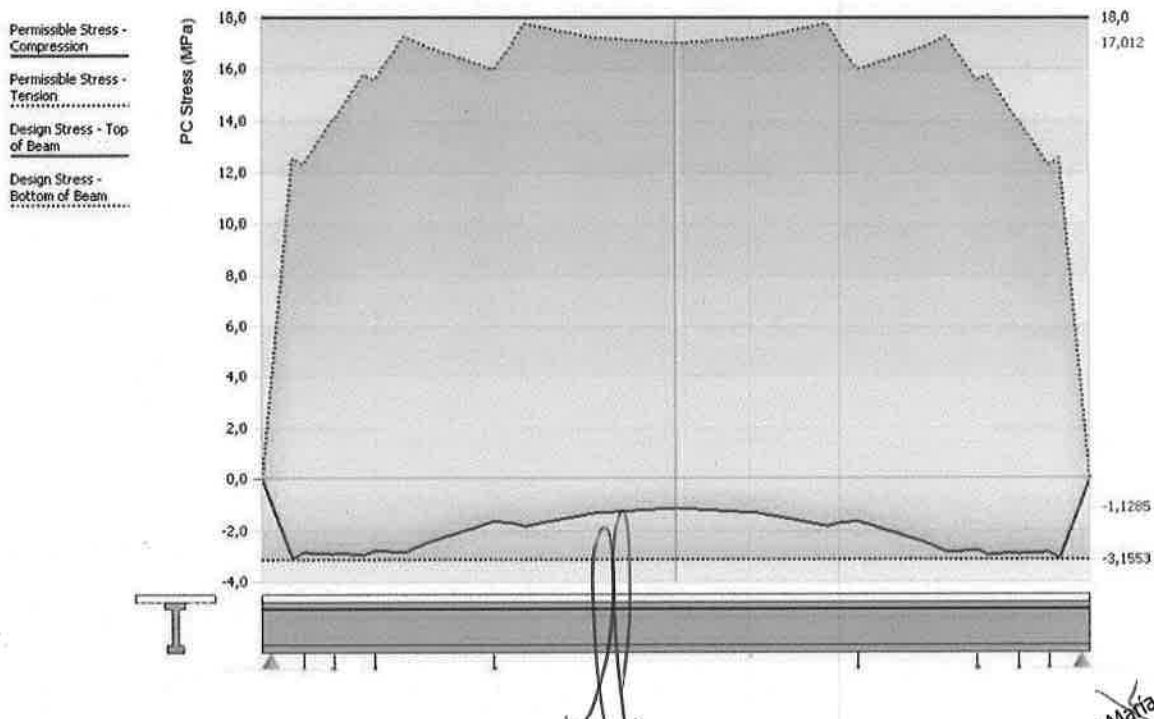
$E_I = E_c / (\psi(t,t_i)+1)$
where $\psi(t,t_i)$ = 0,46174 (Article 5.14.2.3.6)]

Precast section height = 1520,0 mm

		Precast Stress (MPa)	E	Strain (x10⁻⁶)	Curvature (x10⁻⁶) (rad/m)	Deflection[1] (mm) Here	Max.
At Transfer	T	-1,478	E_T	-39,482	-347,17	27,4657	Cracked
	B	18,2758		488,214			
At Erection	T	-1,1285	E_I	-44,068	-466,04	35,7992	35,7992
	B	17,0123		664,308			

Stress limit at transfer (-1,38MPa) is exceeded. (Article 8.3.4.1.2)

[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 25 = 12,5 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

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18 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
60,0	8	1900,0	140,0	75,0	1596,0
105,0	8	1900,0	140,0	75,0	1596,0
150,0	2	1900,0	140,0	75,0	399,0
TOTAL	18				3591,0

Transverse Eccentricity

Tendon layout is symmetrical about the vertical centroidal axis.

INITIAL RELAXATION - Article 5.9.5.4b

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
60,0	8	1425,0	1710,0	0,0	140,0	1596,0	95,76
105,0	8	1425,0	1710,0	0,0	140,0	1596,0	167,58
150,0	2	1425,0	1710,0	0,0	140,0	399,0	59,85
TOTAL	18					3591,0	323,19

Moment about the centroid of the precast beam:

M_r = 323,19-(3591,0*0,787224)
= -2503,733 kN.m

Corresponding stresses:

top stress = 3591/418,14+-2503,73/1,6052E8
= 8,5880327+-15,59791
= -7,00987 MPa

bottom stress = 3591/418,14+-2503,73/-1,494E8
= 8,5880327+16,756906
= 25,3449 MPa

Self weight moment:

c.s.a. = 4,181E5 mm²
density = 25,0 kN/m³
self weight = 4,181E5*25,0
= 10,4535 kN/m
beam length = 25,0 m
distance = 12,5 m
M_{sw} = 0,5*10,4535*12,5*(25,0-12,5)
= 816,68 kN.m

Corresponding stresses:

top stress = 816,68/1,6052E8

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= 5,0878 MPa
bottom stress = 816,68/-1,494E8
= -5,4658 MPa

ELASTIC SHORTENING LOSSES - Article 5.9.5.2.3a

concrete stresses at mid-span:
stress at top of precast beam = -1,9221 MPa
stress at bottom of precast beam = 19,8791 MPa
depth of precast beam = 1520,0 mm

height mm	No of tendons	conc stress MPa	E_p / E_{ci}	tendon force kN	tendon moment kN.m
60,0	8	19,01852	5,2626	112,09727	6,7258363
105,0	8	18,37309	5,2626	108,29304	11,370769
150,0	2	17,72766	5,2626	26,122202	3,9183303
TOTAL	18			246,51251	22,014936

Moment about the centroid of the precast beam:
M_{ed} = 22,014936-(246,51251*0,787224)
= -172,0457 kN.m

hence top stress = -1,922-246,513/418,14--172,046/1,6052E8
= -1,922-0,5895454--1,071821
= -1,4398 MPa
bottom stress = 19,879-246,513/418,14--172,046/-1,494E8
= 19,879-0,5895454-1,1514623
= 18,1381 MPa

After a further 2 iterations of the above process, the top and bottom stresses are as follows:
top stress = -1,47795 MPa
bottom stress = 18,2758 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} = 2520,0 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 = 4,0 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: $k_{tdc,f,i} = \frac{t_f - t_i}{12*\left[\frac{100-4*f'_{ci}}{f'_{ci}+20}\right] + t_f - t_i}$

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

= 0,7645

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

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$$\left\lfloor f'_{ci}+20 \right\rfloor$$

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

$$= 0,7661$$

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

Gross area of section: $A_g = 4,18E5\ mm^2$

Perimeter of section: $p = 4853,9\ mm$

Volume to surface ratio: $V/S = A_g/p = 86,146\ mm$

Factor for the effect of the volume: $k_s = 1,45-0,0051*(V/S) = 1,0107$

Creep coefficient at final time:

$$\psi_b(t_f, t_i) = 1,9*k_s.k_{hc}.k_f.k_{tdc,f,i}.t_i^{-0,118}$$
$$= 1,9*1,0107*0,96*2,7341*0,7645*4,0^{-0,118}$$
$$= 3,272$$

Inertia of the gross concrete section: $I_g = 1,18E11\ mm^4$

Eccentricity of prestressing force: $e_{pg} = -697,18\ mm$

Transformed section coefficient:

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

$$= \frac{1}{1+\frac{197,0}{37,434}*\frac{2520,0}{4,18E5}\left[1+\frac{4,18E5*-697,2^2}{1,2E11}\right]*(1+0,7*3,272)}$$

$$= 0,77840$$

Time factor for creep at deck placement:

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

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

$$= 0,401$$

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

$$= 1,009*0,95*0,73*0,41*0,00048$$

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$= 1,393E-4$

Loss of prestress $= \varepsilon_{bid} \cdot E_p \cdot K_{id}$
$= 1,39E-4*197,0*0,87741$
$= 24,075$ MPa
$= \Delta f_{pSR}$

Total shrinkage loss of force $= \Delta f_{pSR} \cdot A_{ps}$
$= 24,0751*2520,0$
$= 60,6692$ kN

Corresponding gain of moment (about $y = 0$) $= 60,6692*0,09005$
$= 5,46303$ kN.m

Moment about the centroid of the composite beam:
$M_{sh} = 5,46303-(60,6692*0,78722)$
$= -42,297$ kN.m

Corresponding stresses:
top stress $= 60,6692/418140+-42,2973/1,605E8$
$= 0,1450931+-0,263506$
$= -0,118413$ MPa

bottom stress $= 60,6692/418140+-42,2973/-1,494E8$
$= 0,1450931+0,2830858$
$= 0,428179$ MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:
$\psi_b (t_d, t_i) = 1,9 k_s \cdot k_{hc} \cdot k_f \cdot k_{tdc,d,i} \cdot t_i^{-0,118}$
$= 1,9*1,0*0,96*0,7351*0,4019*4,0^{-0,118}$
$= 0,4575$

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,921 MPa
at bottom of precast beam = 19,878 MPa
at center of gravity of prestress: f_{cgp} = 18,5866 MPa

Loss of prestress $= \frac{E_p}{E_{ci}} \cdot f_{cgp} \cdot \psi_b (t_d, t_i) \cdot K_{id}$
$= \frac{197,0}{37,434}*18,587*0,4575*0,8893$
$= 39,8037$ MPa
$= \Delta f_{pCR}$

Total creep loss of force $= \Delta f_{pCR} \cdot A_{ps}$
$= 39,8037*2520,0$
$= 100,305$ kN

Corresponding loss of moment $= 100,305*0,09005$
$= 9,03212$ kN.m (about $y = 0$)

Moment about the centroid of the composite beam:
$M_{cr} = 9,03212-(100,305*0,78722)$
$= -69,931$ kN.m

Corresponding stresses:
top stress $= 100,305/4,181E5+-69,931/1,60517E8$
$= 0,23988+-0,4356$
$= -0,1957$ MPa

bottom stress $= 100,305/4,181E5+-69,931/-1,4941E8$
$= 0,23988+0,46803$
$= 0,70791$ MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.2c

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Stress in tendons after transfer: f_{pt} = 1334,91 MPa
Yield strength of prestressing steel: f_{py} = 1710,0 MPa
Time between tensioning and deck placement: $t = t_s - t_i$ = 28,0- 4,0 = 24,0day
Factor for relaxation steel type: K'_L = 45,0 (low relaxation)

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

$$= \frac{1334{,}9}{45{,}0} * \frac{\log(24*24{,}0)}{\log(24*4{,}0)} * \left[\frac{1334{,}9}{1710{,}0} - 0{,}55\right]\left[1 - \frac{3*(24{,}075+39{,}804)}{1334{,}9}\right] * 0{,}87$$

$$= 7{,}15995 \text{ MPa}$$

$$= \Delta f_{pR1}$$

Total relaxation loss of force = $\Delta f_{pR1} . A_{ps}$
= 7,1599*2520,0
= 18,043074 kN

Corresponding loss of moment = 18,043* 0,09
= 1,6247093 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{rx} = 1,62471-(18,0431*0,7872)
= -12,579 kN.m

Corresponding stresses:
top stress = 18,0431/418140+-12,5792/1,605E8
= 0,0431508+-0,078367
= -0,0352161 MPa

bottom stress = 18,0431/418140+-12,5792/-1,494E8
= 0,0431508+0,0841899
= 0,127341 MPa

Erection of beam Loading

Bending moment from erection loadcase at current span location:
$M_{Applied}$ = 816,67969 kN.m
Corresponding stresses:
top stress = 816,67969/1,6052E8
= 5,0878 MPa
bottom stress = 816,67969/-1,494E8
= -5,4658 MPa

Remove the dead load applied for transfer calculations
M_{sw} = -816,68 kN.m
Corresponding stresses:
top stress = -816,68/1,6052E8
= -5,0878 MPa
bottom stress = -816,68/-1,494E8
= 5,46585 MPa

Construction stage 1A Loading

$M_{Applied}$ = 971,95374 kN.m
Corresponding stresses:
top stress = 971,95374/1,6052E8
= 6,05514 MPa
bottom stress = 971,95374/-1,494E8
= -6,5051 MPa

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

Gross composite section area: A_c = 9,146E5 mm
Perimeter of section: p = 8893,9 mm
Volume to surface ratio: $V/S = A_c / p$ = 102,8 mm

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

Creep coefficient at final time:

$$\psi_b(t_f, t_i) = 1,9*k_s.k_{hc}.k_f.k_{tdc,f,i}.t_i^{-0,118}$$
$$= 1,9*1,0*0,96*2,734*0,764*4,0^{-0,118}$$
$$= 3,2375$$

Inertia of the net concrete section: $I_c = 2,9E11$ mm⁴

Eccentricity of prestressing force: $e_{pc} = 1167,5$ mm

Transformed section coefficient:

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

$$= \frac{1}{1 + \frac{197,0}{37,434}*\frac{2520,0}{9,46E5}\left[1+\frac{9,46E5*1167,5^2}{2,9E11}\right]*(1+0,7*3,2375)}$$

$$= 0,79834$$

Shrinkage of Girder Concrete - Article 5.9.5.4.3a

Concrete shrinkage strain of girder: $\varepsilon_{bdf} = k_s.k_{hs}.k_f.k_{tds,f,i}$

$$*0,00048$$

$$=$$

$$1,009*0,95*0,73*0,76*0,00048$$

$$= 2,592E-4$$

Loss of prestress $= \varepsilon_{bdf}.E_p.K_{df}$
$= 2,59E-4*1,97E8*0,88931$
$= 45,4033$ MPa
$= \Delta f_{psD}$

Total shrinkage loss of force $= \Delta f_{psD}.A_{ps}$
$= 45,4033*2520,0$
$= 114,416$ kN

Corresponding gain of moment (about $y = 0$) $= 114,416*0,09005$
$= 10,3027$ kN.m

Moment about the centroid of the composite beam:
$M_{sh} = 10,3027-(114,416*1,2539)$
$= -133,16$ kN.m

Corresponding stresses:
top stress $= 114,416/953132+-133,164/1,091E9$
$= 0,1200423+-0,122033$
$= -0,00199149$ MPa

bottom stress $= 114,416/953132+-133,164/-2,316E8$
$= 0,1200423+0,5750462$
$= 0,695089$ MPa

Creep of Girder Concrete - Article 5.9.5.4.2b

Creep coefficient at deck placement:

$$\psi_b(t_d, t_i) = 1,9k_s.k_{hc}.k_f.k_{tdc,d,i}.t_i^{-0,118}$$
$$= 1,9*1,0091*0,96*0,7351*0,4019*4,0^{-0,118}$$
$$= 0,4617$$

Creep coefficient at final time:

$$\psi_b(t_f, t_d) = 1,9k_s.k_{hc}.k_f.k_{tdc,f,i}.t_d^{-0,118}$$
$$= 1,9*1,0091*0,96*0,7351*0,7203*28,0^{-0,118}$$
$$= 0,6578$$

Concrete stress at the center of gravity of prestressing tendons:
concrete stresses at transfer:
at top of precast beam = -1,921 MPa

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