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

$= 2,568E-4$

Creep coefficient at final time:

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

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: $E_{cd} = 37,434$ mm²
Eccentricity of the deck: $e_d = 372,459$ mm³

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

$$= \frac{2,6E-4*5,28E5*37,434*1000}{[1+0,7*0,6518]}\left[\frac{1}{9,53E5}\quad\frac{1163,86*372,459}{2,9E11}\right]$$

$$= -1,5466 \text{ MPa}$$

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

$$= \frac{197,0}{18,717}*-1,5466*0,88931\,*(1+0,7*0,65187)$$

$$= -21,083 \text{ MPa}$$
$$= \Delta f_{pss}$$

Total shrinkage gain of force $= \Delta f_{pss}\,.A_{ps}$
$= -21,083*2520,0$
$= -53,129$ kN

Corresponding gain of moment (about $y = 0$) $= -53,129*0,09005$
$= -4,7841$ kN.m

Moment about the centroid of the composite beam:
$M_{sh} = -4,7841-(-53,129*1,2539)$
$= 61,8345$ kN.m

Corresponding stresses:

top stress = -53,129/953132+61,8345/1,091E9
= -0,055741+0,0566662
= 0,000924744 MPa

bottom stress = -53,129/953132+61,8345/-2,316E8
= -0,055741+-0,267021
= -0,322763 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 = $-0,5 * \lambda * \sqrt{f'_c}$

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= -0,5 * 1,0 * √40
= -3,15469 MPa

PERMISSIBLE STRESSES FOR CAST IN PLACE CONCRETE

Compression

Article 5.9.4.2
char. strength of c.i.p. concrete = 40,0 MPa
stress limitation factor = 0,6
permissible compression stress = $0,6 * f_{cu}$
= 0,6 * 40
= 24,0 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.
However, no tensile stress is present at this section.

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	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
C.I.P. Stage 1A		971,954			6,05514	-6,5051
Time-Dependent Losses						
Time of Transfer to Time of Deck Placement						
Shrinkage	-60,669	42,2973			0,11841	-0,4281
Creep	-100,31	69,9308			0,19577	-0,7079
Relaxation	-18,043	12,5792			0,03522	-0,1273
Time of Deck Placement to Final Time						
Shrinkage						
Girder	-114,42	133,164	0,10288	0,00199	0,00199	-0,6950
Deck	55,8219	-64,969	-0,0502	-9,7E-4	-9,7E-4	0,33912
Creep	-302,13	351,639	0,27168	0,00526	0,00526	-1,8355
Relaxation	-18,043	20,9995	0,01622	3,14E-4	3,14E-4	-0,1096
TOTAL PERMANENT			0,34059	0,00659	4,93318	8,20622

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

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

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where $\psi(t,t_i)$ = 0,46174 (Article 5.14.2.3.6)]

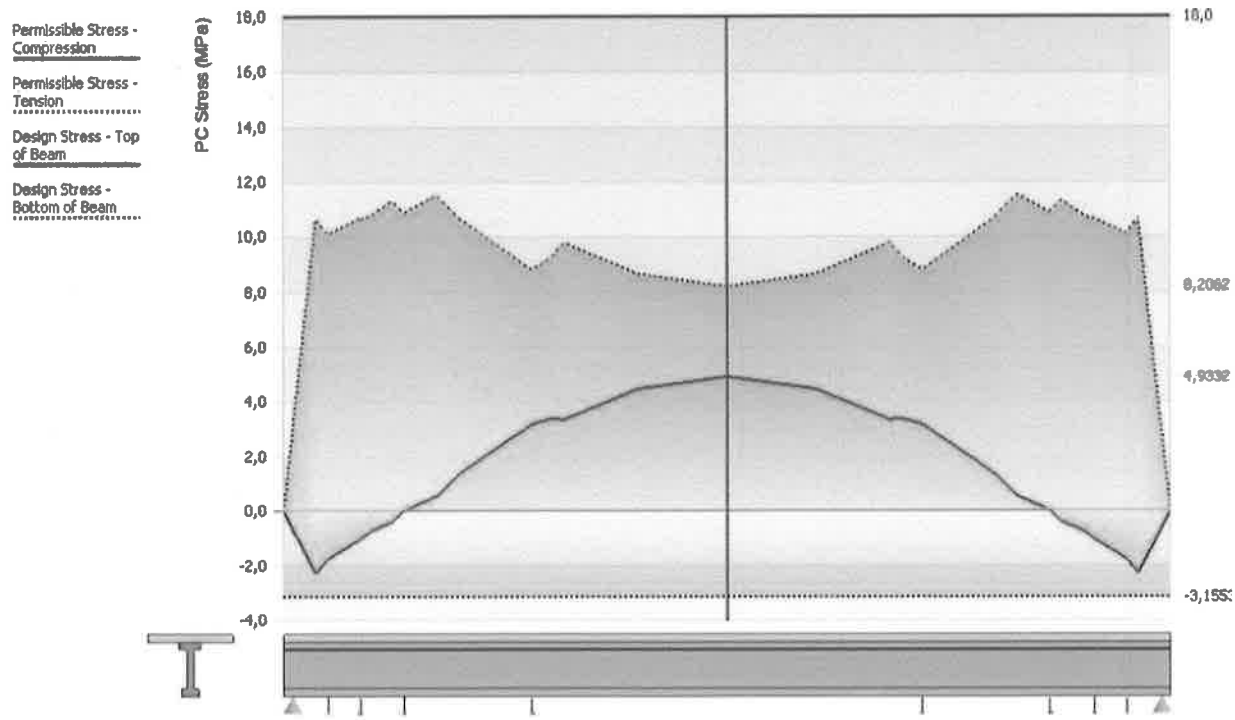
Short Term Elastic Modulus, E_B = 37,434 kN/mm²

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			
From in situ 1A	T	6,05514	E_S	161,755	220,743	-13,817	-13,817
	B	-6,5051		-173,77			

Stress limit at transfer (-1,38MPa) is exceeded. [Article 5.9.4.1.2]

[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 – cargas vivas

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:

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

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 5.9.5.4.4b

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)

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= 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:
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

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

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

$= 0,7661$

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

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

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

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

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: $I_g = 1,18E11 \text{ mm}^4$

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

Transformed section coefficient:

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

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

$= 0,77840$

Time factor for creep at deck placement:

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

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

$= 0,401$

Time factor for shrinkage at deck placement:

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

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

$= 0,411$

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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

=

1,009*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
= Δf_{pSR}

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

Corresponding gain of moment = 60,6692*0,09005
(about y = 0) = 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
= Δf_{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

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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 = 24,0day
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 MPa

= Δf_{pR1}

Total relaxation loss of force = Δf_{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

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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 = 9,46E5 mm²

Perimeter of section: p = 8893,9 mm

Volume to surface ratio: $V/S = A_c/p$ = 106,38 mm

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

Creep coefficient at final time:

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

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
= Δf_{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:

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

Creep coefficient at final time:

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

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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

Change in concrete stress at centroid of prestressing strands:
concrete stress change after permanent loads:
at top of precast beam = 0,18819 MPa
at bottom of precast beam = 14,7097 MPa
Δf_{cd} = 13,8494 MPa

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

$$= \frac{197,0}{37,434} * 18,5866 * (0,87842 - 0,46174) * 0,87741 + \frac{197,0}{18,717} * 13,8494 * 0,65781 * 0,87741$$

$$= 119,894 \text{ MPa} = \Delta f_{pCD}$$

Total creep loss of force = $\Delta f_{pCD} * A_{ps}$
= 119,894*2520,0
= 302,132 kN

Corresponding loss of moment = 302,132*0,09005
= 27,2059 kN.m
(about y = 0)

Moment about the centroid of the composite beam:
M_{cr} = 27,2059-(302,132*1,2539)
= -351,64 kN.m

Corresponding stresses:
top stress = 302,132/9,531E5+-351,64/1,09121E9
= 0,31698+-0,3222
= -0,0053 MPa

bottom stress = 302,132/9,531E5+-351,64/-2,3157E8
= 0,31698+1,51849
= 1,83548 MPa

Relaxation of Prestressing Strands - Article 5.9.5.4.3c

Loss of prestress = Δf_{pR1}
= 7,15995 MPa
= Δf_{pR2}

Total relaxation loss of force = $\Delta f_{pR2} . A_{ps}$
= 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*1,2539)
= -21,0 kN.m

Corresponding stresses:
top stress = 18,0431/953132+-20,9995/1,091E9
= 0,0189303+-0,019244
= -0,000314051 MPa

bottom stress = 18,0431/953132+-20,9995/-2,316E8

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= 0,0189303+0,0906829
= 0,109613 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: $E_{cd} = 37,434$ mm²
Eccentricity of the deck: $e_d = 372,459$ mm³

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

$$= \frac{2,6E-4*5,28E5*37,434*1000}{[1+0,7*0,6518]} \left[\frac{1}{9,53E5} \quad \frac{1163,86*372,459}{2,9E11} \right]$$

= -1,5466 MPa

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

$$= \frac{197,0}{18,717} * -1,5466*0,88931 * (1+0,7*0,65187)$$

= -21,083 MPa
$= \Delta f_{pss}$

Total shrinkage gain of force = $\Delta f_{pss} \cdot A_{ps}$
= -21,083*2520,0
= -53,129 kN

Corresponding gain of moment = -53,129*0,09005
(about $y = 0$) = -4,7841 kN.m

Moment about the centroid of the composite beam:
M_{sh} = -4,7841-(-53,129*1,2539)
= 61,8345 kN.m

Corresponding stresses:
top stress = -53,129/953132+61,8345/1,091E9
= -0,055741+0,0566662
= 0,000924744 MPa

bottom stress = -53,129/953132+61,8345/-2,316E8
= -0,055741+-0,267021
= -0,322763 MPa

SDL non-structural concrete etc. Loading

$M_{Applied}$ = 868,56283 kN.m
$P_{Applied}$ = -117,7665 kN

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Corresponding stresses:
top stress = -117,7665/953132,1 + 868,56283/1,0912E9
= -0,1235 + 0,79596
= 0,67240 MPa
bottom stress = -117,7665/953132,1 + 868,56283/-2,316E8
= -0,1235 + -3,751
= -3,8743 MPa

Transient Load BM 2 Loading

$M_{applied}$ = 1652,56 kN.m for Service-I
1322,05 kN.m for Service-III
$N_{applied}$ = -307,97 kN for Service-I
-246,37 kN for Service-III

Corresponding stresses:
top stress = -307,97/9,531E5 + 1652,56/1,0912E9 Service-I
= -0,3231 + 1,51443
= 1,19133 MPa
bottom stress = -246,37/9,531E5 + 1322,05/-2,316E8 Service-III
= -0,2584 + -5,709
= -5,9675 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 * λ * $\sqrt{f'_c}$ >= -4,137 MPa
= -0,498 * 1,0 * √40
= -3,15469 MPa

PERMISSIBLE STRESSES FOR CAST IN PLACE CONCRETE

Compression

Article 5.9.4.2
char. strength of c.i.p. concrete = 40,0 MPa
stress limitation factor = 0,6
permissible compression stress = 0,6*f_{cu}
= 0,6 * 40
= 24,0 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.
However, no tensile stress is present at this section.

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	bottom	Precast top	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

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Erection		0,0			0,0	0,0
C.I.P. Stage 1A		971,954			6,05514	-6,5051
Time-Dependent Losses						
Time of Transfer to Time of Deck Placement						
Shrinkage	-60,669	42,2973			0,11841	-0,4281
Creep	-100,31	69,9308			0,19577	-0,7079
Relaxation	-18,043	12,5792			0,03522	-0,1273
Time of Deck Placement to Final Time						
Shrinkage						
Girder	-114,42	133,164	0,10288	0,00199	0,00199	-0,6950
Deck	55,8219	-64,969	-0,0502	-9,7E-4	-9,7E-4	0,33912
Creep	-302,13	351,639	0,27168	0,00526	0,00526	-1,8355
Relaxation	-18,043	20,9995	0,01622	3,14E-4	3,14E-4	-0,1096
Utilities etc.		868,563	1,45404	0,79596	0,79596	-3,7507
	-117,77		-0,1235	-0,1235	-0,1235	-0,1235
(Diff. Shr. 1	-55,822	64,9687	0,0	0,0	9,72E-4	-0,3391)
(Diff. Shr. 2		0,0	0,0	0,0	0,0	0,0)
(Differential shrinkage is not adverse or not specified						)
TOTAL PERMANENT			1,67108	0,67900	5,60559	4,33193
(γ_{TG} = 0,5)						
Temp Grad P1	-1037,0	-418,83	2,81518	-0,7164	-0,7164	0,72062
Temp Grad R1	207,409	83,7667	-0,5630	0,14328	0,14328	-0,1441
Temp Grad R2		0,01687	2,82E-5	1,55E-5	1,55E-5	-7,3E-5
(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		1652,56	2,76651		1,51443	
	-307,97		-0,3231		-0,3231	
- III		1322,05		1,21155		-5,709
	-246,37			-0,2584		-0,2584
Transient Load BM 3- I		0,0	0,0		0,0	
- III		0,0		0,0		0,0
Transient Load BM 4- I		1652,56	2,76651		1,51443	
	-308,06		-0,3232		-0,3232	
- III		1322,05		1,21155		-5,709
	-246,45			-0,2585		-0,2585
Transient Load BM 5- I		0,0	0,0		0,0	
- III		0,0		0,0		0,0
Transient Load BM 6- I		0,0	0,0		0,0	
- III		0,0		0,0		0,0
TOTAL PERMANENT LOAD + TRANSIENT LOAD						
Transient Load BM 2			3,55148	0,91565	6,08051	-1,7798
Transient Load BM 4			3,55138	0,91557	6,0804	-1,7799

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 = 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)]

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Effective Elastic Modulus for time dependant effects, E_E = 30,1055 kN/mm²

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

ψ = creep factor = 0,87842

$\psi_{tc\%}$ = percentage of creep occurring before c.i.p. deck added = 0,46174/0,87842 = 52,5647%

χ = ageing factor, taken as 0,8

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

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	27,4587	Cracked
	B	18,2758		488,214			
At Erection	T	-1,1285	E_I	-44,068	-466,04	35,8941	35,8941
	B	17,0123		664,308			
Long-term Dead	T	5,60559					
	B	4,33193					
Dead Stress Increment	T	7,08354	E_E	235,291	459,512	-29,195	-29,195
	B	-13,944		-463,17			
Total Dead Load deflection					112,344	-1,7365	0,0
Transient Load BM 2-I	T	1,19133	E_S	31,8247	152,035	-9,6506	-9,6506
	B	-7,4594		-199,27			
Extreme in-service					264,379	-11,387	-11,387

Stress limit at transfer (-1,38MPa) is exceeded. [Article 5.9.4.1.2]

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

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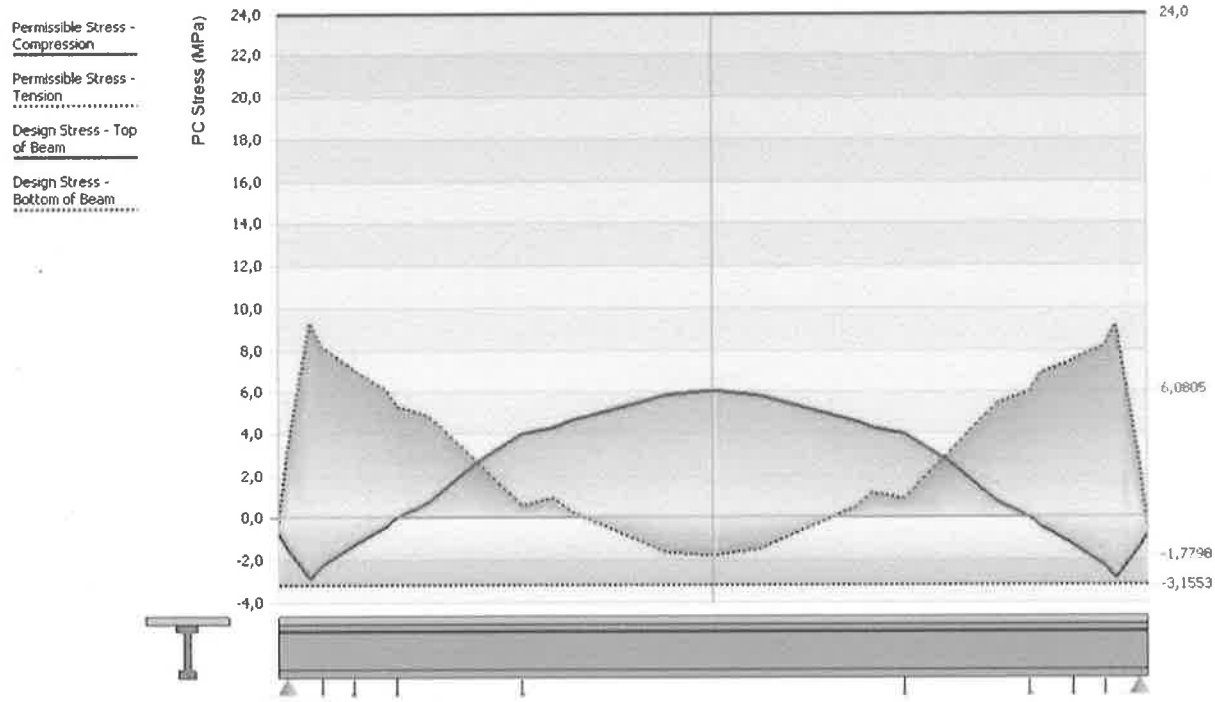

Momento en estado límite de resistencia I

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:

Live Load Bending Moments

Transient Load BM 1

Limit State - Strength I

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

---

height of neutral axis = 192,776 mm

depth in compression = 192,776 mm

Resistance Factor (ϕ) from Article 5.5.4.2.1

Type of force effect is:

Tension-controlled reinforced concrete:

ϕ = 0,90

Location		strain	stress	height
		MPa	mm	
Precast beam	- *top*	-0,020654	0,0	1520,0

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	- bottom	0,003	33,9999	0,0
C.I.P. stage 1	*- top*	-0,024078	0,0	1740,0
	- bottom	-0,020654	0,0	1520,0
Tendon	*- top*	-0,006131	-1207,9	150,0
	- bottom	-0,0047	-925,8	60,0
Reinforcement	*- top*	-0,020032	-500,0	1480,0
	- bottom	0,002362	400,0	41,0

Ultimate Strength capacity of section = 280,304*0,9
= 252,274 kN.m

In conjunction with axial load of -183,92 kN

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

---

height of neutral axis = 1657,98 mm
depth in compression = 82,0244 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 MPa	stress mm	height
Precast beam	*- top*	-0,005046	0,0	1520,0
	- bottom	-0,06064	0,0	0,0
C.I.P. stage 1	*- top*	0,003	33,9999	1740,0
	- bottom	-0,005046	0,0	1520,0
Tendon	*- top*	-0,061951	-1900,0	150,0
	- bottom	-0,065211	-1900,0	60,0
Reinforcement	*- top*	-0,006509	-500,0	1480,0
	- bottom	-0,05914	-500,0	41,0

Ultimate Strength capacity of section = 8202,7*1
= 8202,7 kN.m

In conjunction with axial load of -183,92 kN

SUMMARY OF APPLIED LOADINGS

	BENDING MOMENT kN.m	AXIAL kN
Erection loading before composite =	1020,8496	0,0
Erection loading to stage 1ª =	1214,9422	0,0
Utility loading etc. =	1085,7035	-147,2081
Transient loading =	2891,9795	-538,9406

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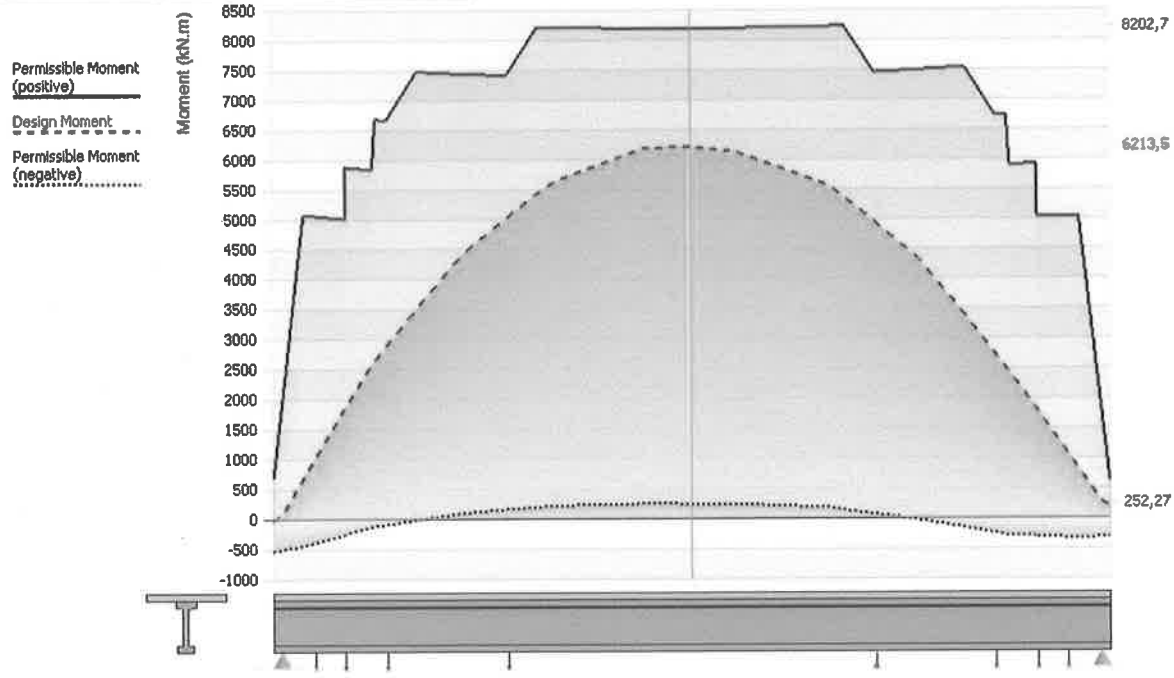
TOTAL	6213,4749	-686,1487

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



My_{total} = 6213,50 kN.m < 8202,70 kN.m

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Section details:
Ref 1 "Section 1"
at 0,99 x span = 0,99 x 25 = 24,75 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

SUMMARY OF APPLIED LOAD EFFECTS

	SHEAR FORCE kN	BENDING MOMENT kN.m	AXIAL LOAD kN
Erection loading before composite =	-134,1831	36,750586	0,0
Erection loading to stage 1A =	159,71011	43,740982	0,0
Wearing surface loading =	0,0	0,0	0,0
Utility loading etc =	197,9719	-187,5056	107,05388
Transient loading:			
Max SF + associated BM =	472,28211	1310,3591	-105,0957
Max BM + associated SF =	322,72268	398,96668	-344,8851
TOTAL - Max SF + associated BM:	695,78102	1203,3451	1,9581826
- Max BM + associated SF:	546,2216	291,95265	-237,8313

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

The current section is closer to the support than the critical section for shear (article 5.8.3.2). The shear forces in the table are therefore the values at the critical section for shear.

Article 5.8.3
Transverse Shear Reinforcement

The *'maximum SF with associated BM'* loadcase is critical at this section.

factored shear force V_u = 695,781 kN
factored moment M_u = 1203,35 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 = 1740,0 mm
compression fiber to tension force centroid d_e = 1503,2 mm
resistance moment of section (no axial load) M_r = 5132,99 kN.m
total tension force T = 3465,03 kN
distance from resultant tension to compression = M_r / T
= 1,48137 m

$0,9*d_e$ = 0,9*1503,2 = 1352,88 mm
$0,72*h$ = 0,72*1740,0 = 1252,8 mm
hence, effective shear depth, d_v = 1481,37 mm

Critical section for shear - Article 5.8.3.2

angle of compressive stress (see below) θ = 29,0°

$0,5*d_v .\cot(\theta)$ = 0,5*1481,37*1,80405
= 1336,23 mm
critical section for shear is larger of d_v and 1336,23 mm
hence, critical section for shear:
from face of support = 1,48137 m
from end of the beam = 22,769 m

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

$$v = \frac{V_u - \varphi.V_p}{\varphi.b_v.d_v} = \frac{695,781 - 0,9*0,0}{0,9*0,174*1,48137} = 2,99929 \text{ MPa}$$

Factored axial force N_u = 237,831 kN
Area of prestress steel on tension side A_{ps} = 1400,0 mm²
Area of concrete on flexural tension side A_c = 2,198E5 mm²
Area of non-prestress steel on tension side A_s = 805,033 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
$|V_u - V_p|*d_v$ = 1030,71 kN.m
Factored moment (not less than $|V_u - V_p|d_v$) $|M_u|$ = 1203,35 kN.m

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General procedure: (Article 5.8.3.4.2)

Strain in reinforcement:

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

$$\varepsilon_s = \frac{\left[\frac{1203{,}3}{1{,}4814} + 0{,}5*237{,}83 + |695{,}78 - \quad 0{,}0| - 1400{,}0* \quad 1{,}33\right]}{200{,}0*805{,}03 + 197{,}0*1400{,}0}$$

$$= -5{,}E{-}4$$

which is less than 0, therefore $\varepsilon_s = 0{,}0$

with at least the minimum amount of shear reinforcement:

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

$$= 4{,}8$$

with no shear reinforcement:

$s_x = 0{,}12775$ m

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

$$s_{xe} = s_x * \frac{1{,}38}{a_g + 0{,}63} = 0{,}127 * \frac{1{,}38}{1{,}0 + 0{,}63} = 0{,}10815 \text{ m}$$

$$= 4{,}25813 \text{ IN}$$

12,0IN <= s_{xe} <= 80,0IN

$s_{xe} =$ 12,0 IN

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

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

$$= 4{,}8$$

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

$$= 29 + 3500* \ 0{,}0$$

$$= 29{,}0°$$

Nominal Shear Resistance
Article 5.8.3.3

---

inclination of the transverse reinforcement, $\alpha = 90{,}0°$

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)

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$= 0,08297*4,8*1,0*(\sqrt{40,0})*174,0*1481,4$
$= 649,26$ kN

with no shear reinforcement:

$V_c = 0,08297*\beta\lambda(\sqrt{f'_c}).b_v.d_v$ (5,8,3,3-3)
$= 0,08297*4,8*1,0*(\sqrt{40,0})*174,0*1481,4$
$= 649,26$ 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)
$= 0,5*0,9*(649,261+0,0)$
$= 292,167$ 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)
$= 0,25 * 40,0*174,0*1481,37 + 0,0$
$= 2577,58$ kN

$V_{r,max} = \varphi.V_{n,max}$
$= 0,9*2577,58$
$= 2319,82$ kN

Transverse Reinforcement Requirements

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

Minimum Transverse Reinforcement (article 5.8.2.5):

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

$= 0,083*1,0*\sqrt{40,0}*174,0/500,0$
$= 0,1827$ 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)

$V_{s,min} = 0,18275* \ 500,0*1481,37*1,80405$

$= 244,196$ kN

therefore,

$V_{n,min} = 649,277 + 244,196 + 0,0$
$= 893,473$ kN
$\varphi.V_{n,min} = 0,9*893,473$
$= 804,126$ kN

Additional transverse reinforcement for shear (article 5.8.3.3):

V_u is less than $\varphi V_{n,min}$, therefore minimum reinforcement only is required

Link arrangement:

size	diameter mm	Maximum spacing mm 2 legs	4 legs
10	11,3	1098,3	2196,6
15	16,0	2201,93	4403,87
20	19,5	3270,64	6541,29

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25 25,2 5462,17 10924,3

Maximum spacing (article 5.8.2.7):
$0,125*f'_c$ = 0,125*40,0
= 5,0 MPa

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

$$= \frac{695,781-0,9*0,0}{0,9*174,0*1481,37}$$

= 2,99929 MPa

v_u < $0,125*f'_c$
therefore, s_{max} = $0,8*d_v$ <= 609,6 mm
= 0,8*1481,37
= 1185,09 mm

hence maximum permissible spacing = 609,6 mm

Longitudinal Reinforcement - Article 5.8.3.5

Non-prestressed reinforcing steel:
Area on flexural tensile side A_s = 805,033 mm²
yield stress f_y = 500,0 MPa
tensile capacity = 805,033*500,0
= 402,517 kN

Prestressing steel:
Area on flexural tensile side A_{ps} = 1400,0 mm²
tensile strength f_{pu} = 1900,0 MPa
allowance for development = 0,27340
tensile capacity = 0,27340*1400,0*1900,0
= 727,253 kN

Total tensile capacity = 402,517+727,253
= 1129,77 kN

Equation 5,8,3,5-1 right hand side:
$M_u / (d_v . \varphi)$ = 1203,35/(1,48137*1,0)
= 812,32 kN
$0,5*N_u / \varphi$ = 0,5*237,831/ 0,75
= 158,554 kN
$V_u / \varphi - 0,5*V_s - V_p$ = 695,781/0,9 - 0,5*244,026 - 0,0
= 651,077
* cot(θ) = 651,077*1,80405
= 1174,57 kN
where V_s is taken from equation 5,8,3,3-4 but not greater than V_u / φ
V_u / φ = 695,781/0,9
= 773,09 kN
$A_v / s . f_y . d_v . \cot(\theta)$ = 1,83E-4*500,0*1,48137*1,80405
= 244,026 kN

812,32+158,554+1174,57 = 2145,45 kN

which is greater than 1129,77, so Equation 1 is not satisfied, therefore additional longitudinal reinforcement is required.
Area of fully developed non-prestressed reinforcing steel required on flexural tension side of member is given by:
$A_s . f_y$ = 2145,45-1129,77 = 1015,68 kN
hence additional area required,
= 1015,68/500,0
= 2031,36 mm²

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]

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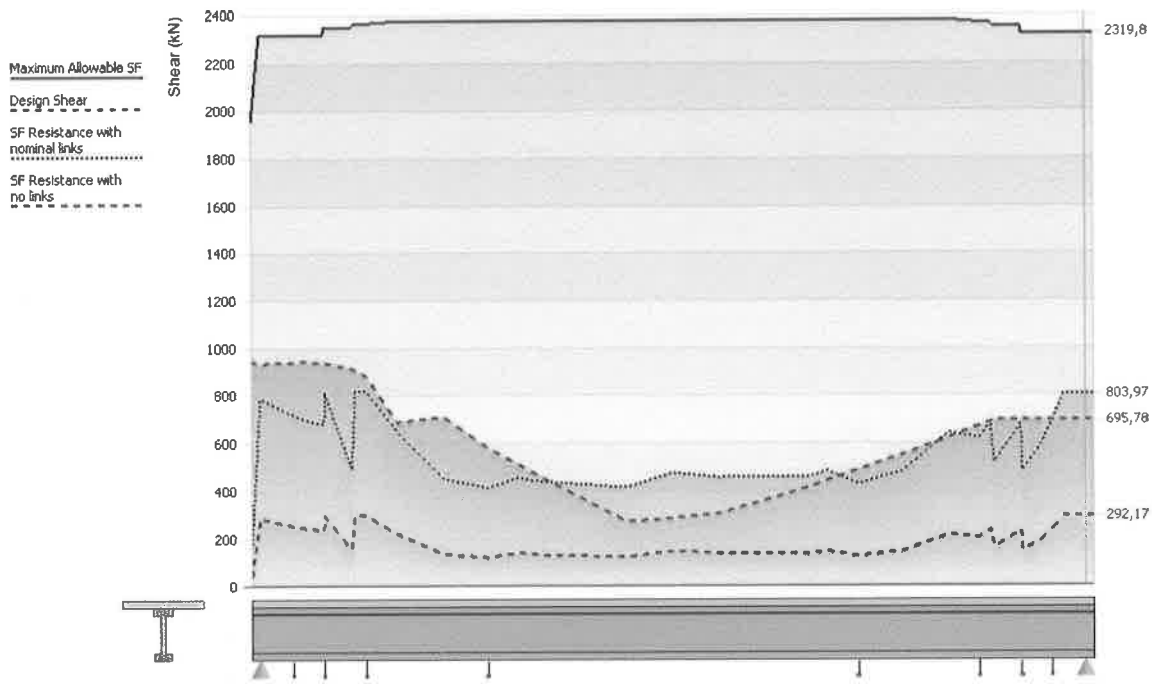
0,0	1,80382	111,195
0,25	0,30586	600,0
0,9144	0,35090	571,597
1,25	0,37175	539,543
1,618	0,39334	509,919
2,15	0,40581	494,247
2,1644	0,28324	600,0
2,97222	0,59392	337,71
3,0644	0,25404	600,0
3,4	0,23361	600,0
4,3144	0,21479	600,0
5,69444	0,44275	453,014
7,0	0,36287	552,734
7,9144	0,23531	600,0
8,41667	0,21349	600,0
11,1389	0,18262	600,0
12,5	0,18262	600,0
13,8611	0,18262	600,0
16,5833	0,18262	600,0
17,0856	0,18262	600,0
18,0	0,24389	600,0
19,3056	0,24946	600,0
20,6856	0,18262	600,0
21,6	0,22776	600,0
21,9356	0,18302	600,0
22,0278	0,35010	572,9
22,8356	0,19604	600,0
22,85	0,39532	507,364
23,382	0,28692	600,0
23,75	0,18788	600,0
24,0856	0,18262	600,0
24,75	0,18262	600,0
25,0	0,18262	600,0

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



Fz_{total} = 695,78 kN.m < 2319,80 kN

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Armadura de Pretensado

El pretensado estará compuesto por torones de diámetro 15,20 mm, 18 torones en el tramo central del vano y 10 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
-157,5	60,0	140,0	MP3	60	15,24	Full stress
-112,5	60,0	140,0	MP3	60	15,24	Debonded
-67,5	60,0	140,0	MP3	60	15,24	Full stress
-22,5	60,0	140,0	MP3	60	15,24	Full stress
22,5	60,0	140,0	MP3	60	15,24	Full stress
67,5	60,0	140,0	MP3	60	15,24	Full stress
112,5	60,0	140,0	MP3	60	15,24	Debonded
157,5	60,0	140,0	MP3	60	15,24	Full stress
-157,5	105,0	140,0	MP3	60	15,24	Full stress
-112,5	105,0	140,0	MP3	60	15,24	Debonded
-67,5	105,0	140,0	MP3	60	15,24	Full stress
-22,5	105,0	140,0	MP3	60	15,24	Debonded
22,5	105,0	140,0	MP3	60	15,24	Debonded
67,5	105,0	140,0	MP3	60	15,24	Full stress
112,5	105,0	140,0	MP3	60	15,24	Debonded
157,5	105,0	140,0	MP3	60	15,24	Full stress
-22,5	150,0	140,0	MP3	60	15,24	Debonded
22,5	150,0	140,0	MP3	60	15,24	Debonded

Debonded Tendons

x-y coordinates mm		distance from left end (m) start	end
-112,5	60,0	1,25	23,75
112,5	60,0	1,25	23,75
-112,5	105,0	2,15	22,85
-22,5	105,0	3,4	21,6
22,5	105,0	3,4	21,6
112,5	105,0	2,15	22,85
-22,5	150,0	7,0	18,0
22,5	150,0	7,0	18,0

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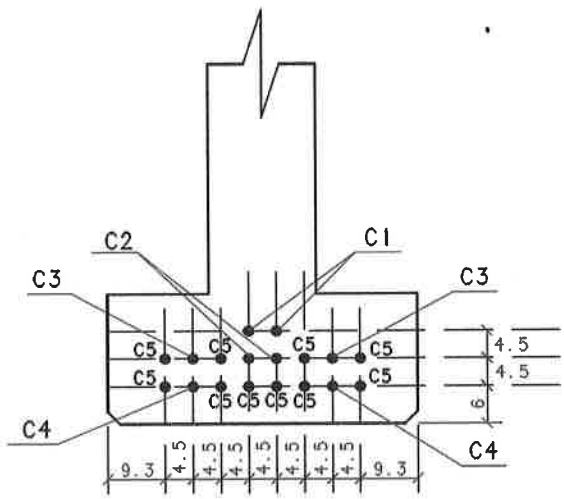
PRETENSADO

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
-208,75	41,0	12,5	MP2	0,0	25,0	25,0
-62,5	41,0	12,5	MP2	0,0	25,0	25,0
62,5	41,0	12,5	MP2	0,0	25,0	25,0
208,75	41,0	12,5	MP2	0,0	25,0	25,0
-208,75	168,75	10,0	MP2	0,0	25,0	25,0
210,0	170,0	10,0	MP2	0,0	25,0	25,0
-47,0	170,0	10,0	MP2	0,0	25,0	25,0
47,0	170,0	10,0	MP2	0,0	25,0	25,0
-260,0	1366,42	10,0	MP2	0,0	25,0	25,0
-47,0	1346,42	10,0	MP2	0,0	25,0	25,0
47,0	1346,42	10,0	MP2	0,0	25,0	25,0
260,0	1366,42	10,0	MP2	0,0	25,0	25,0
-260,0	1480,0	12,5	MP2	0,0	25,0	25,0
260,0	1480,0	12,5	MP2	0,0	25,0	25,0
47,0	1480,0	12,5	MP2	0,0	25,0	25,0
-47,0	1480,0	12,5	MP2	0,0	25,0	25,0

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

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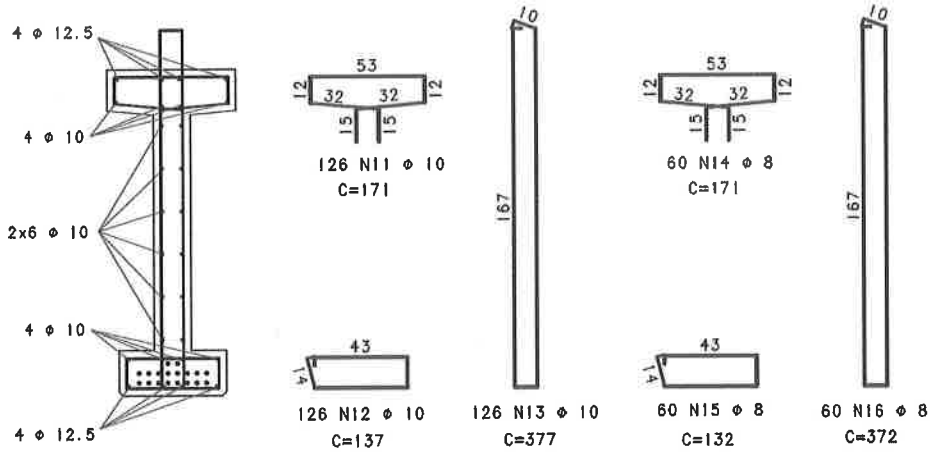
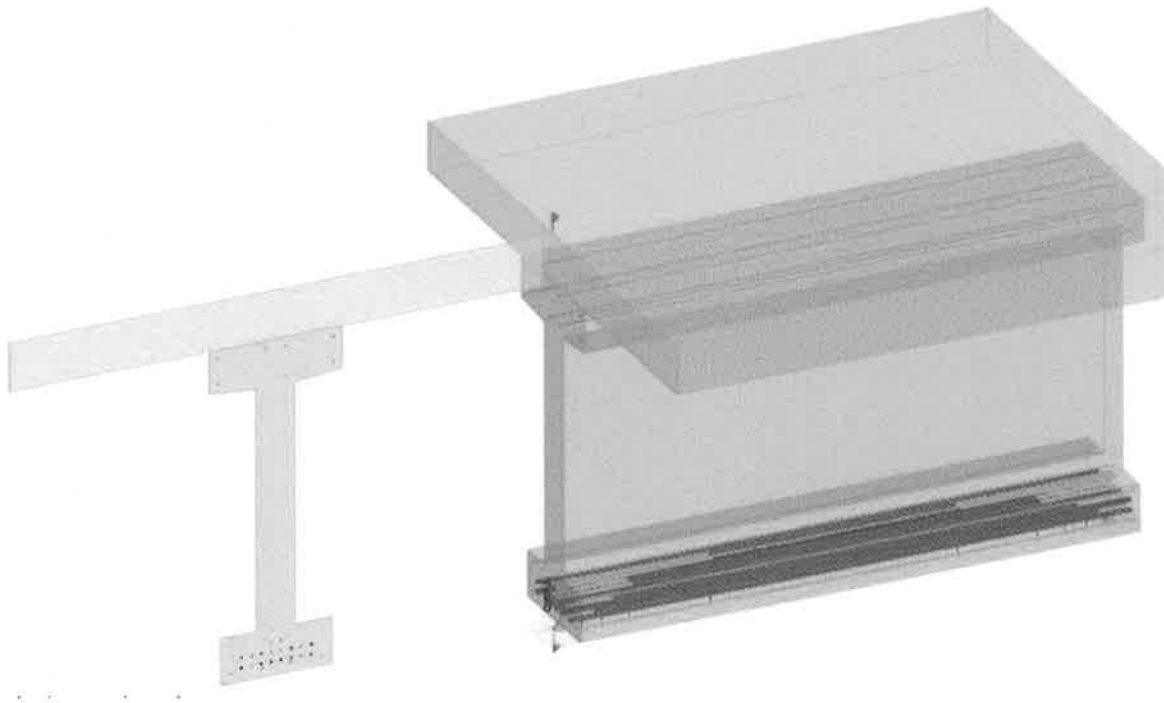
CORTE A-A

ESC. 1:25



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



Sin más, saludos cordiales,

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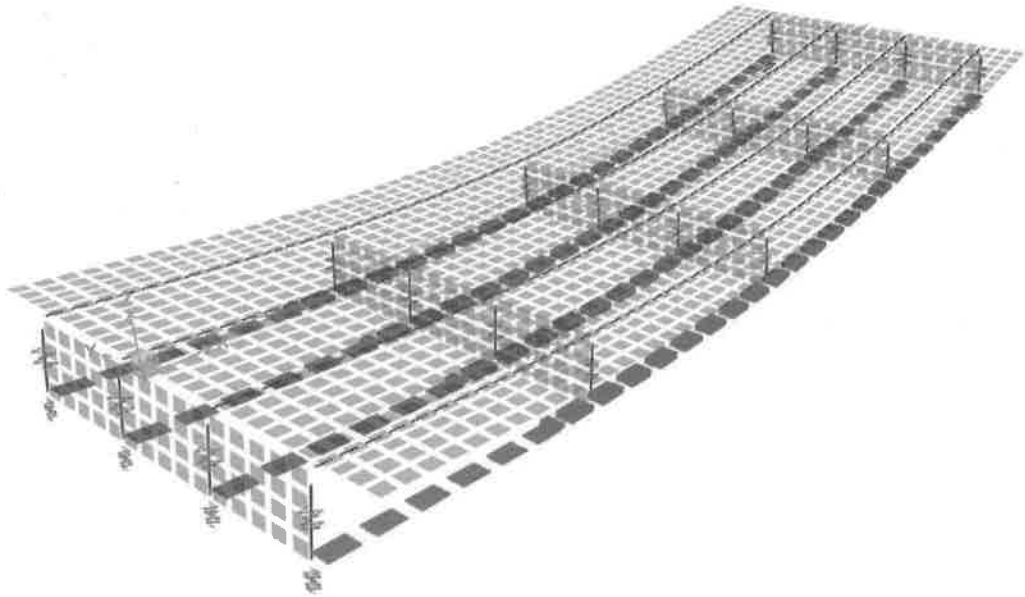
ANEXO 6

JUSTIFICACION DE REVISION DE PROYECTO

.- • Memoria de Cálculo de los TABLEROS, L=25m - PUENTE ORDEN 01 y 02



MEMORIA DE CÁLCULO

TABLEROS L=25 m. ÑACUNDAY & YATYTAY

Descripción breve

Consideraciones generale de diseño – Armados y modelado de TABLEROS PARA LUCES VANO DE 25 m.

PUENTES YATYTAY 1X25 m & ÑACUNDAY 3X25m

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MODELO DESCRIPCIÓN

Se ha modelado el tablero empleando elementos finitos (FEM Model), empleando elementos de área o shells (Losa Deck Tablero, Vigas Diafragmas transversales c/ 6.25m máx.), elementos frame o vigas lineales (I-152 Rotesma), discretizado c/ 1.00m máximo.

Los cabos de pretensado se han modelado como elementos, no como cargas. A los elementos se les aplica una pretensión inicial F0 = 14.250 kg/cm2. Se evalúan pérdidas por deformaciones elásticas, fluencia lenta, retracción del Hormigón y relajación del cabo (Diferidas + Instantáneas).

Elementos Links para simular apoyo de viga en base o ala inferior y aparatos de apoyos con sus alturas (Rotulas fijas y deslizantes longitudinales).

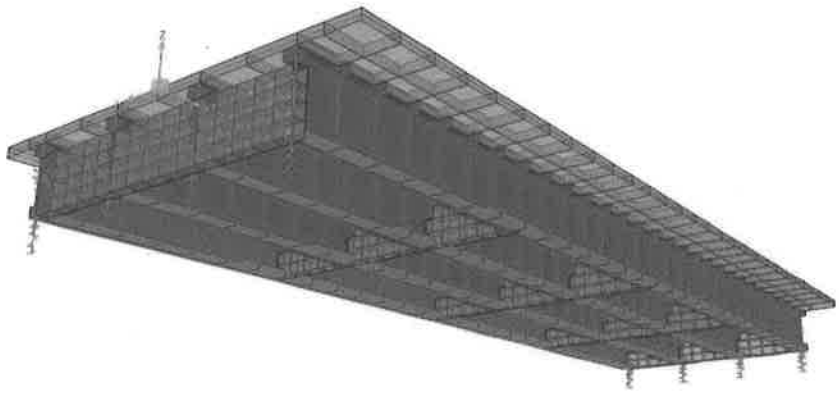

Ilustración 1: VISTA 3D - MODELO DE ANÁLISIS EXTRUIDO & DISCRETIZADO. –

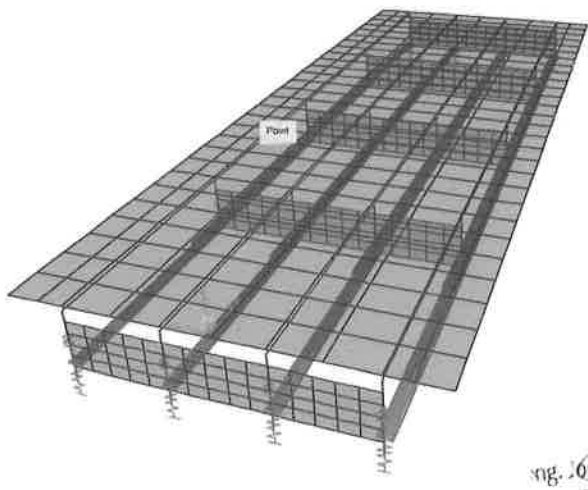

Ilustración 2: Vista 3D. Fill Object.

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ELEMENTOS DE MODELADO

1. **VIGAS LONGITUDINALES:**
   a. **VIGAS ROTESMA I-152 HA-40.**

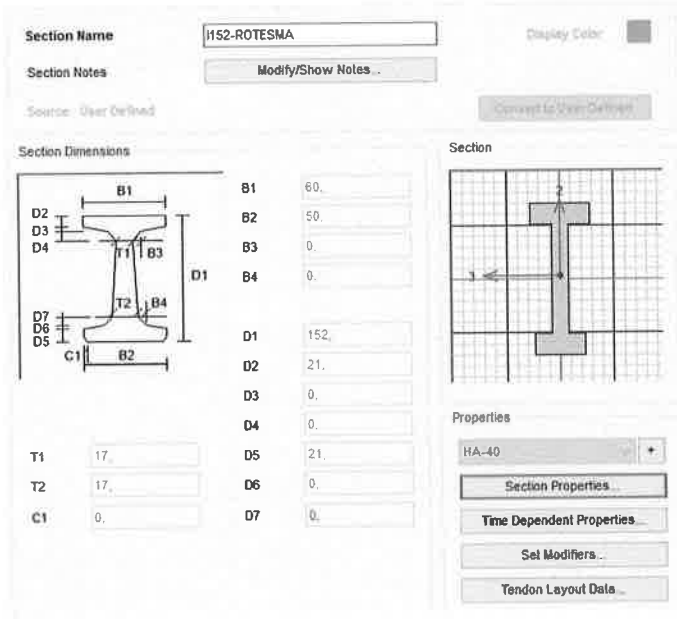

Ilustración 3: Definición de Sección I152-Rotesma. Geometría. Material HA-40.

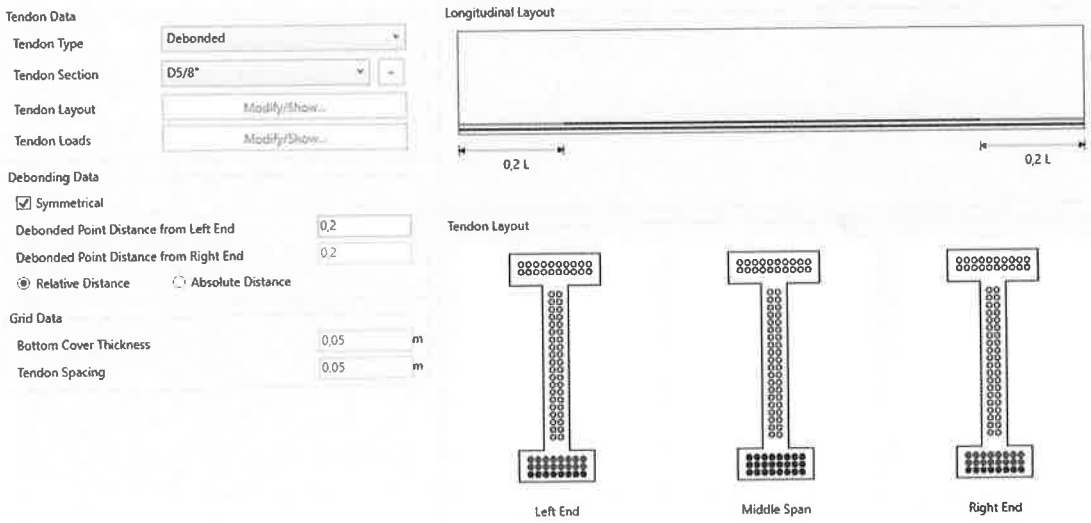

Ilustración 4: Trazado de Tendones de pretensado en pista. En verde zona envainada para alivio de momentos flectores en extremos.

- ✓ **Tensión de tesado inicial: 14200 kg/cm2**
- ✓ **Cantidad de cabos: 24 x D15.8mm – As = 1.50 cm2 por tendón.**

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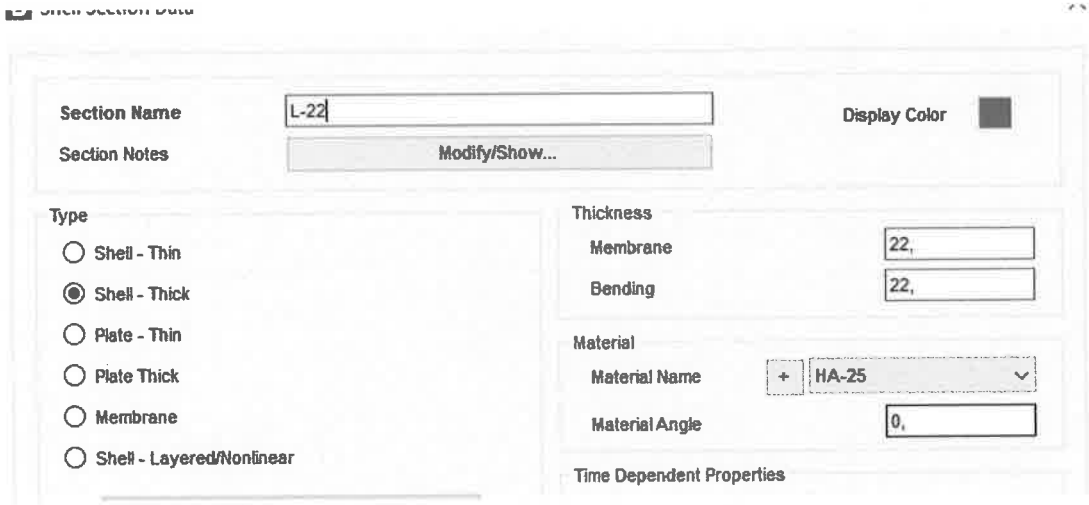
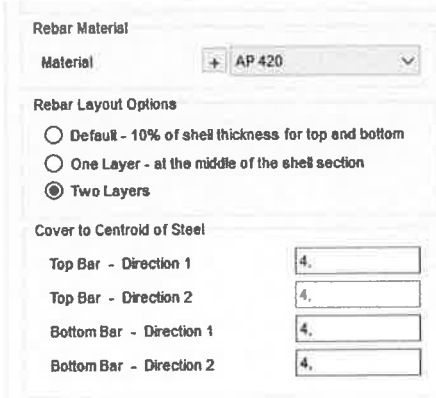
2. ELEMENTOS SHELL:

Se definieron para el Tablero elementos shells de espesor 22cm, material HA-25. Elemento tipo shell-Thick, es decir se considerará la deformación por corte transversal. (Formulación de placa GRUESA).

La deformación por corte tiende a ser importante cuando el espesor de la lámina es mayor que aproximadamente 1/5 a 1/10 del tramo de la curvatura de flexión de la placa. El corte también puede volverse significativo en lugares donde se concentran esfuerzos de flexión, que ocurren cerca de cambios repentinos en el espesor o en las condiciones de soporte, y cerca de aberturas o esquinas entrantes.





Los recubrimientos mínimos desde el centroide de las áreas al borde de los paramentos serán de al menos 4cm.

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Las vigas riostras fueron discretizadas como elementos área, elementos shell tipo thick espesor igual al de la viga, 30cm.

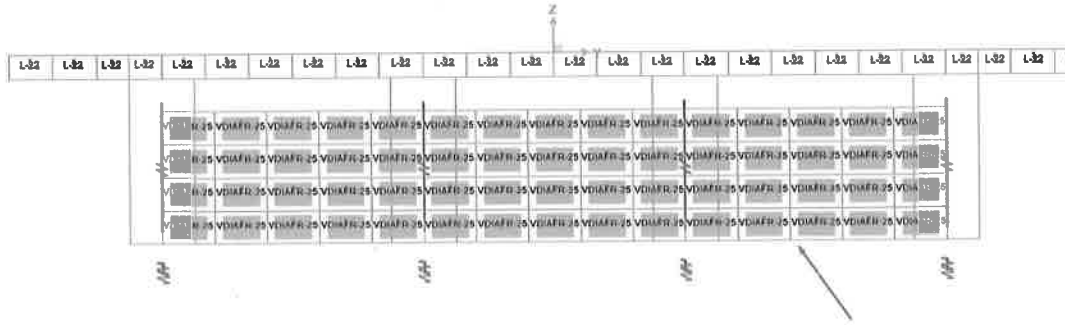
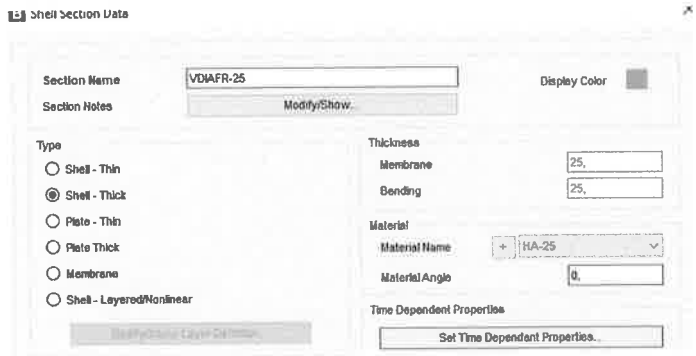


Ilustración 5: VISTA 2D - ALZADO VIGA RIOSTRA INTERMEDIA



Los recubrimientos mínimos desde el centroide de las áreas al borde de los paramentos serán de al menos 3cm.

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NORMATIVAS DE DISEÑO

1. **AASHTO LRFD 2020**
2. **MANUAL DE CARRETERAS MOPC**

MATERIALES

1. **CONCRETO**
   a. **TABLERO DECK & Vigas RIOSTRAS: Fck-250**
      E = 320000 Modulo elástico
      U = 0.2 COEFICIENTE POISON
      G = 1361120 Modulo Transversal
      Fck = 250 Kg/cm2 Resistencia característica 28 días.

   b. **VIGAS LONGARINAS I-152 ROTESMA PREMOLDADO: HA-40 (40 MPa).**

2. **ACERO PASIVO: AP 420**
3. **ACERO ACTIVO DE PRETENSADO: ASTM A416 Gr.270**
4. **ACERO PLACAS DE NEOPRENO: ASTM A36**
5. **NEOPRENO APARATOS DE APOYOS: Dureza SHORE 60.**

CARGAS DE DISEÑO

PESO PROPIO DEAD "DC" – Cálculo automático por peso específico de la sección definida.

PESO BARRERA "DC" – Accesorio NO ESTRUCTURAL.

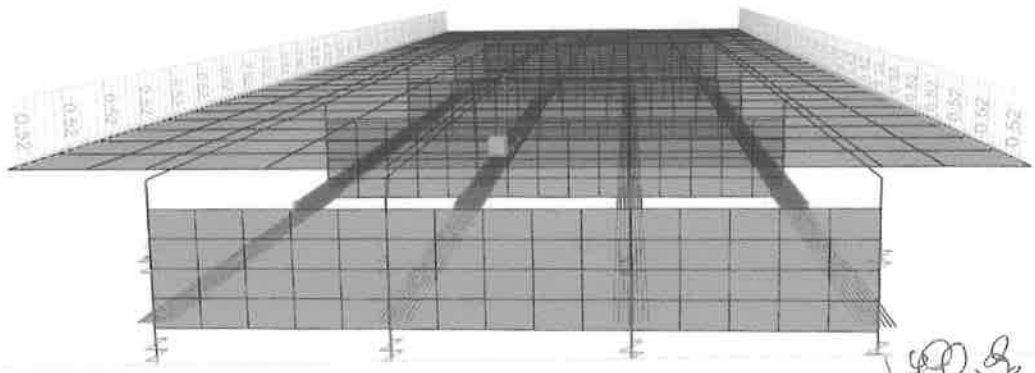

Ilustración 6: BARRIER: Valor de 0.52 Tonf/m. Aplicado en los 25 m de tramo.

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PESO CARPETA ASFÁLTICA "DW": **Peso específico del concreto asfáltico 2.24 Tonf/m3. Distribuidos a 1,00m de cada extremo.**

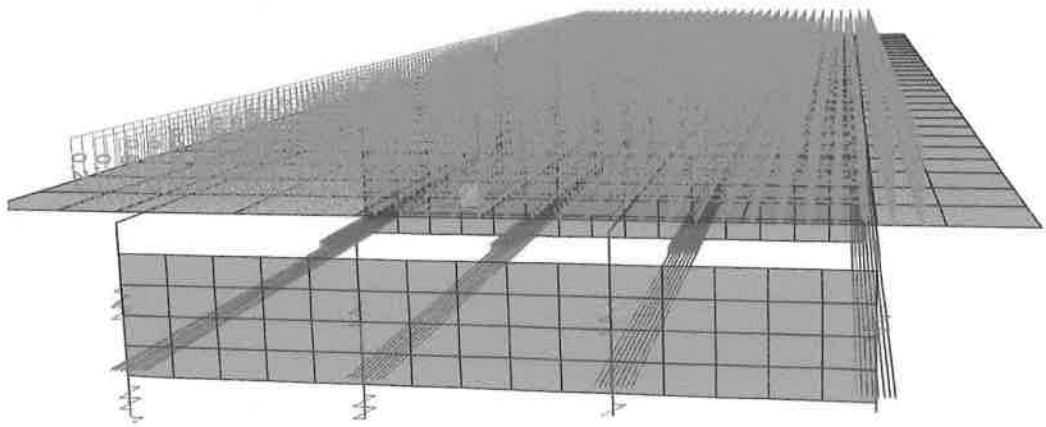

Ilustración 7: Carga DW - Valor 0,22 Tonf/m2 - Espesor medio de 10cm de carpeta asfáltica.

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65 (SESENTA Y CINCO)

PESO ACERA "SIDEWALK"

Peso de la acera de Hormigón Armado – Espesor 15cm. Valor: 380 kgf/m2.

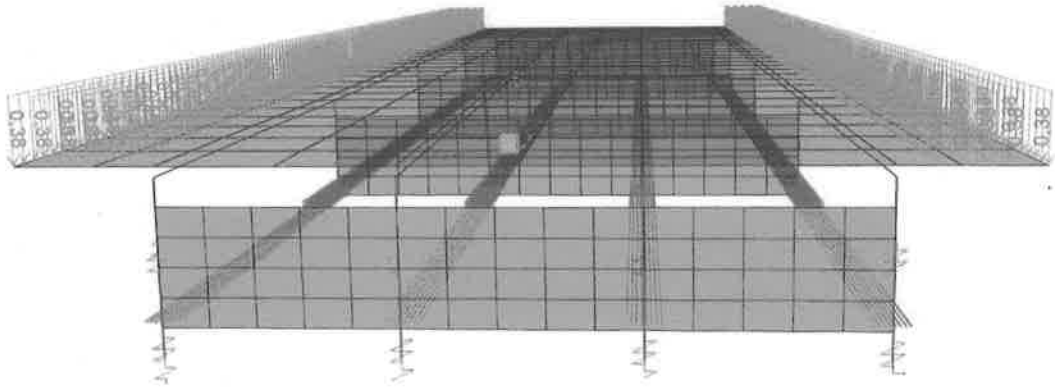

Ilustración 8: Valor de carga de Acera Sidewalk.

SOBRECARGA PEATONAL "PL": **Se considera un valor de 366 kgf/m2 distribuidos en el ancho de acera Art. 3.6.1.6 AASHTO LRFD.**

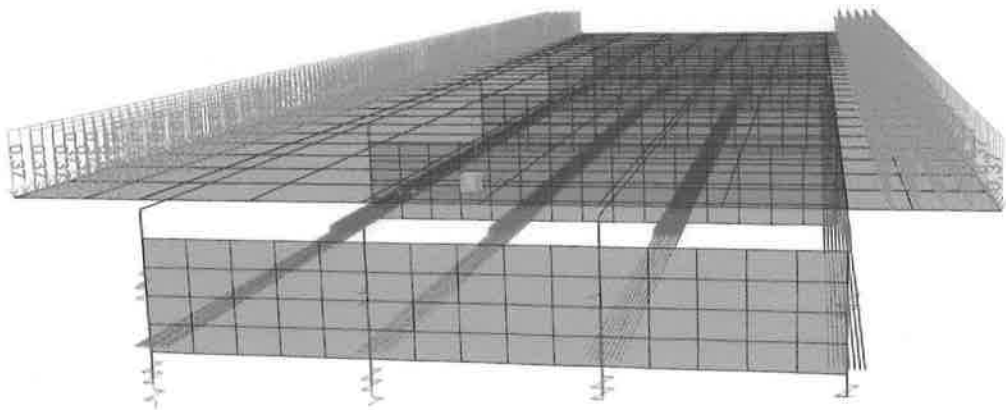

Ilustración 9: Sobrecarga peatonal sobre Aceras. PL. Valor: 366 kg/m2

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SOBRECARGA VEHICULAR "LL"

Se definieron dos carriles de 4,00m de ancho (magenta y verde) para recorrido de las cargas móviles siguientes de la HL-93.

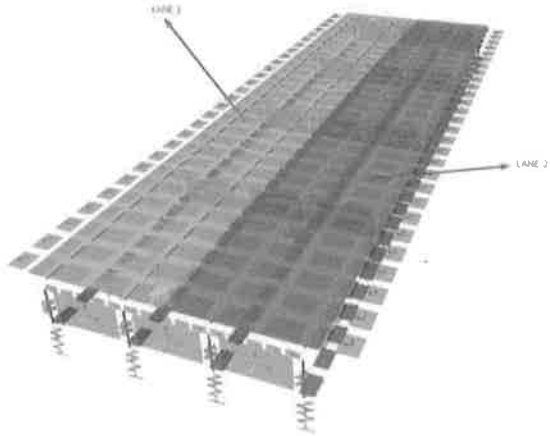

Ilustración 10: VISTA ANCHO DE CARRILES.

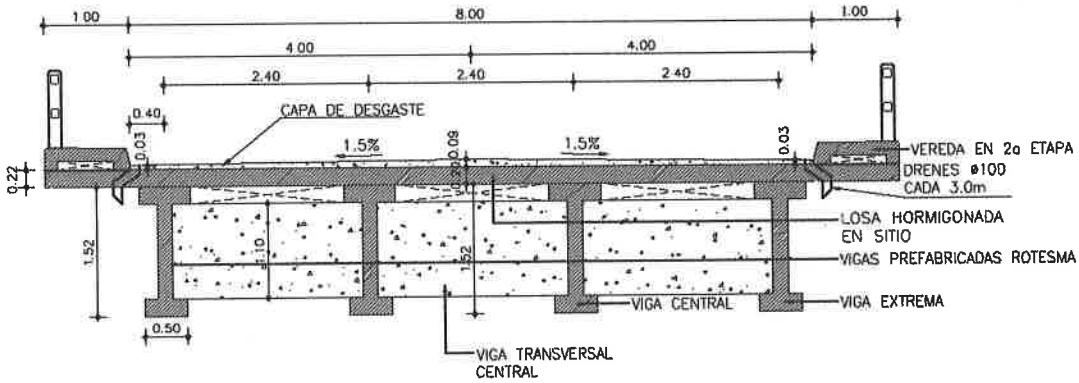

Ilustración 11: SECCIÓN DE TABLERO.-

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VEHICULO HL-93M

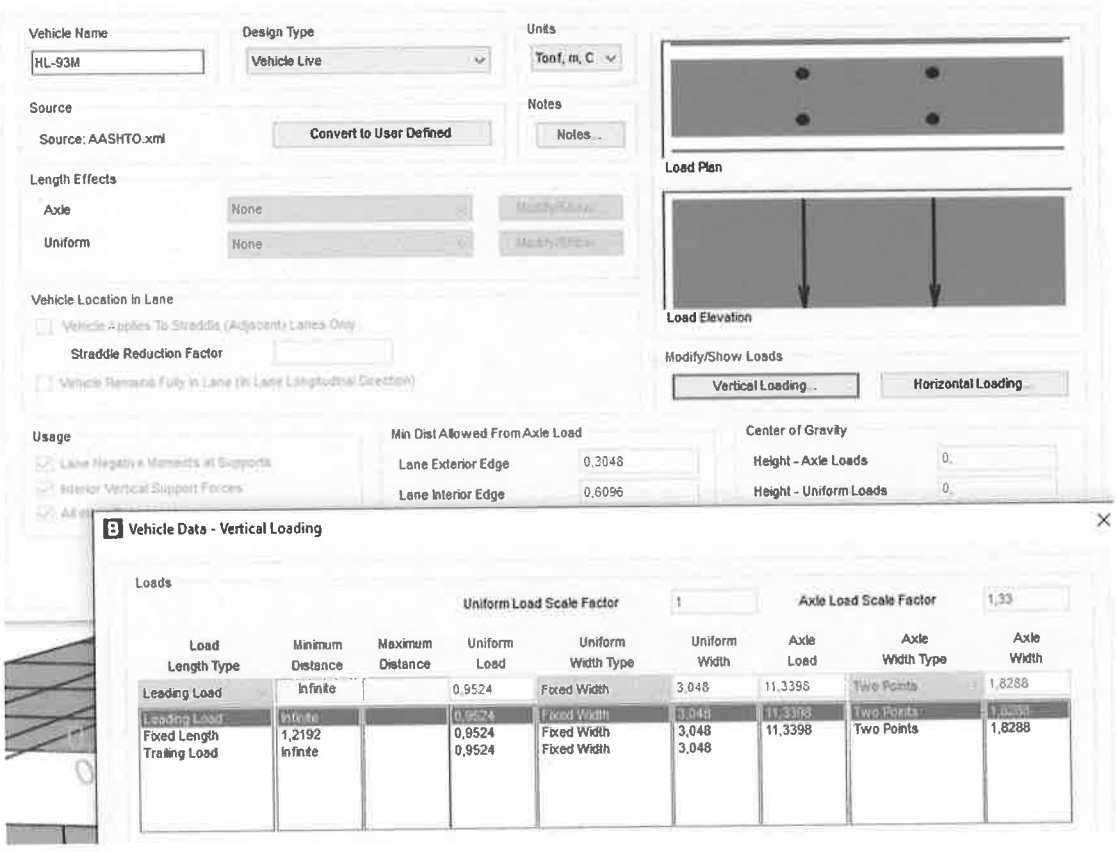

Ilustración 12: DEFINICIÓN HL-93M

Se aplicará con min. distancias:

1. **Borde del lane o carril exterior: 0,30 m.**
2. **Borde del lane o carril interior: 0,60 m.**
3. **Separación entre ejes de ruedas transv.: 1,82 m.**

Valores axiles: 11,3 Tonf

Carga distribuida en el ancho del carril 3,05m: 0,95 Ton/m

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VEHICULO HL-93K:

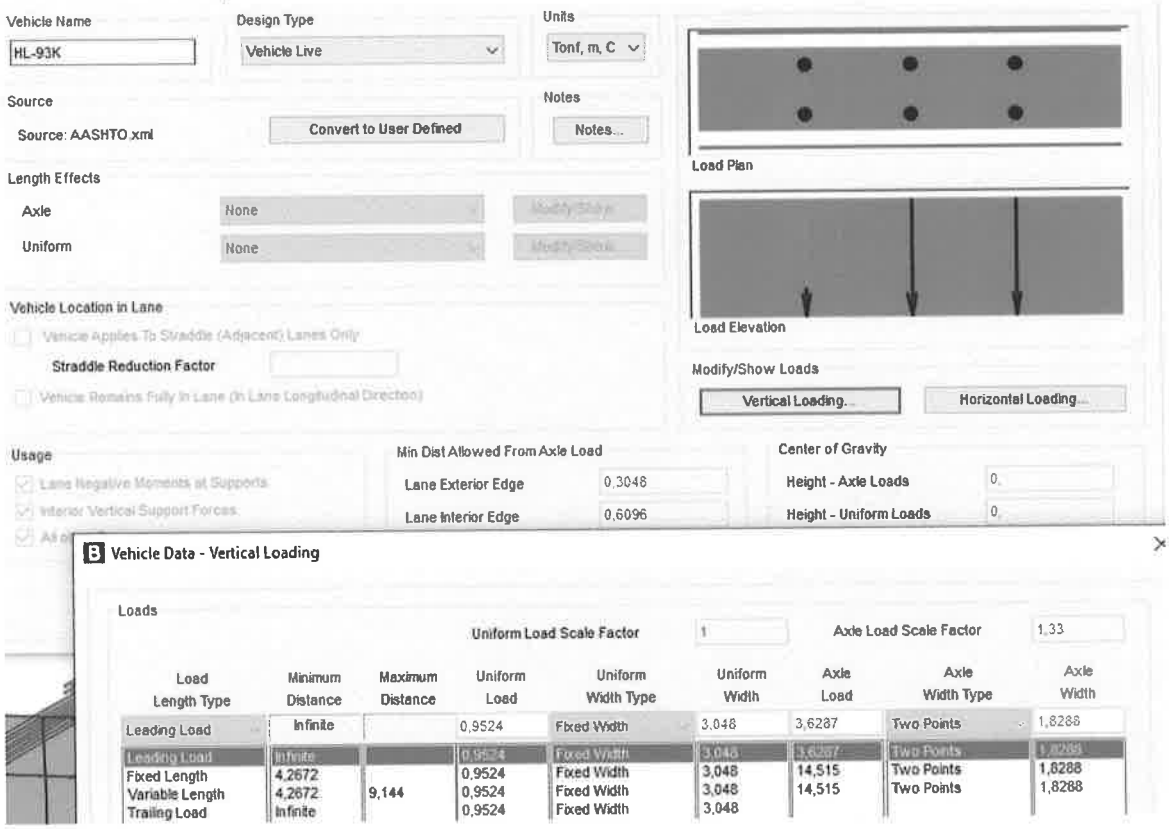

Ilustración 13: Definición carga HL-93K

Se aplicará con min. distancias:

1. **Borde del lane o carril exterior: 0,30 m.**
2. **Borde del lane o carril interior: 0,60 m.**
3. **Separación entre ejes de ruedas transv.: 1,82 m.**

Valores axiles: 14,52 & 3,63 Tonf separados según esquema anterior.

Carga distribuida en el ancho del carril 3,05m: 0,95 Ton/m

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VEHICULO HL-93S

Ilustración 14: Camión de diseño HL-93S.

Se aplicará con min. distancias:

1. **Borde del lane o carril exterior: 0,30 m.**
2. **Borde del lane o carril interior: 0,60 m.**
3. **Separación entre ejes de ruedas transv.: 1,82 m.**

Valores axiles: 14,52 & 3,63 Tonf separados según esquema anterior.

Carga distribuida en el ancho del carril 3,05m: 0,95 Ton/m

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CAMIÓN TIPO HL-93LB: TIPO BI-TREN.

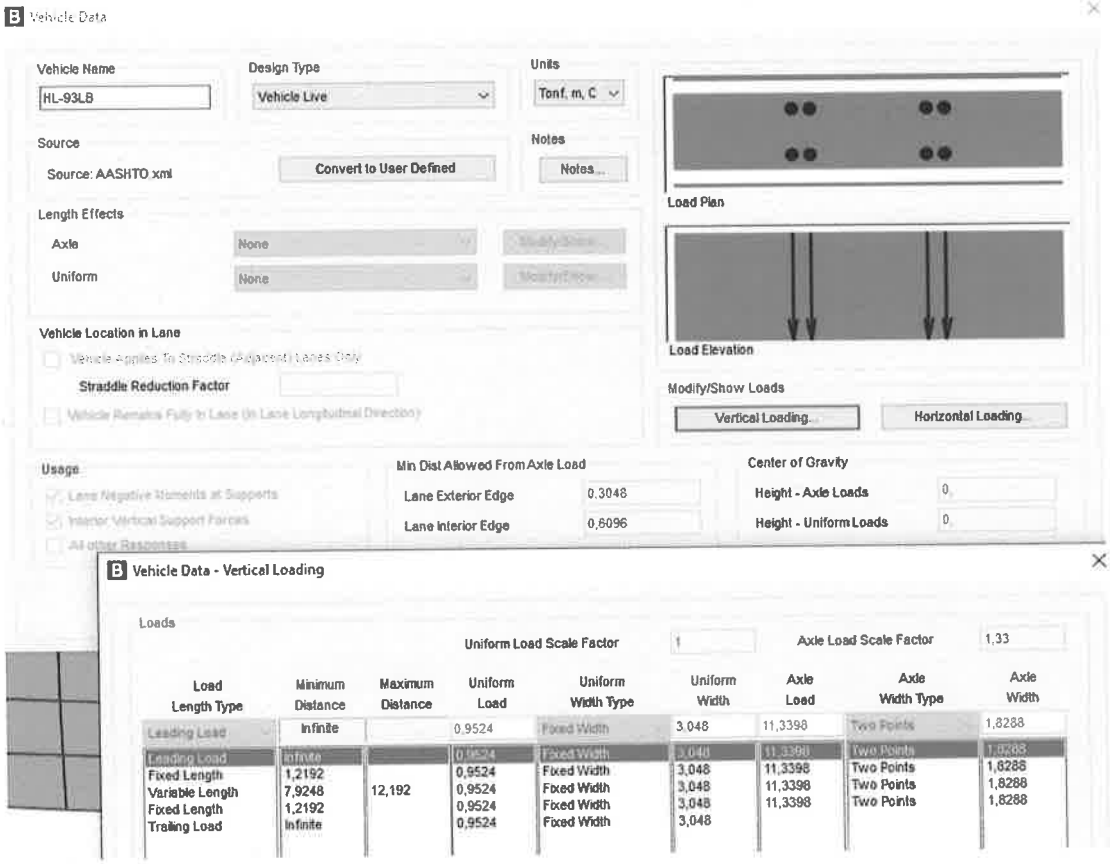

Ilustración 15: Definición del camión Tipo HL-93LB

Se aplicará con min. distancias:

1. **Borde del lane o carril exterior: 0,30 m.**
2. **Borde del lane o carril interior: 0,60 m.**
3. **Separación entre ejes de ruedas transv.: 1,82 m.**

Valores axiles: 14,52 & 3,63 Tonf separados según esquema anterior.

Carga distribuida en el ancho del carril 3,05m: 0,95 Ton/m

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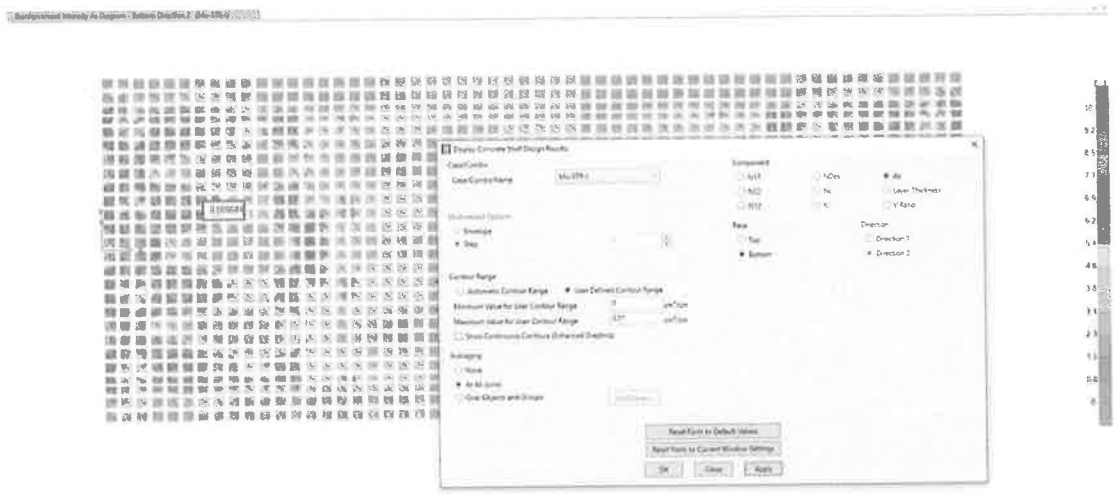
RESULTADOS OBTENIDOS

Combinaciones para estudiar para los armados de LOSAS DECK & VIGAS RIOSTRAS.

I. **COMBO 1 (C1): Mu-SRT1 = 1.25 DC (PESO PROPIO, BARRERA, ACERA) + 1.50 DW + 1,75 (1+%IM) LL + 1.25 PRET.**
El factor de presencia múltiple está incluido: m=1 para ambos carriles cargados, y m=1,20 para un solo carril cargado.
Efecto de la carga viva vehicular. Máximos positivos.



Valor Extremo: As_C1 = 0,010 cm2/cm == 10,0cm2/m
Cubierto por: 3 varillas de 12mm c/10 (3 varillas por viguetas).
As dispuesto: 1,13cm2 x 3 x 100/10 = 11,3 cm2

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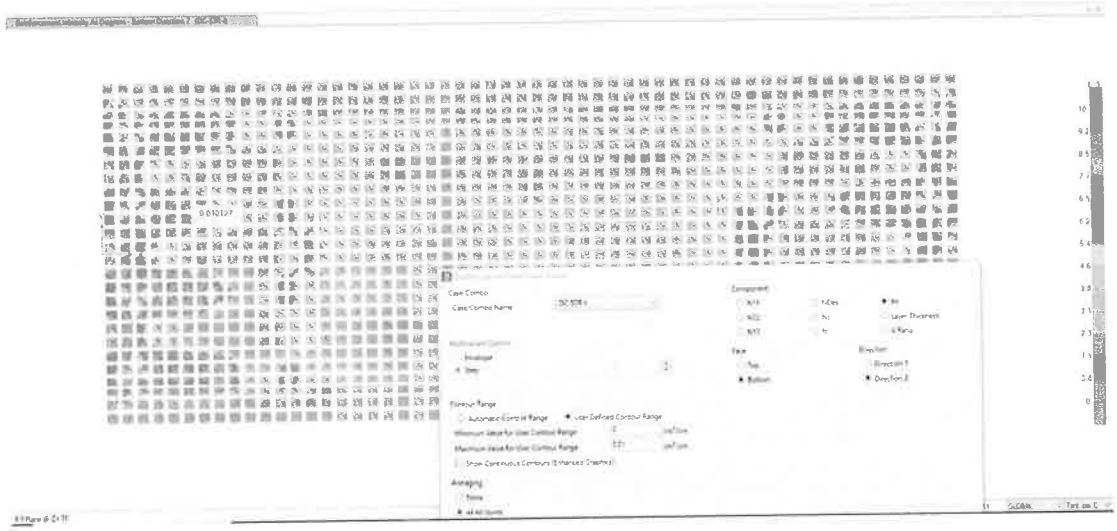
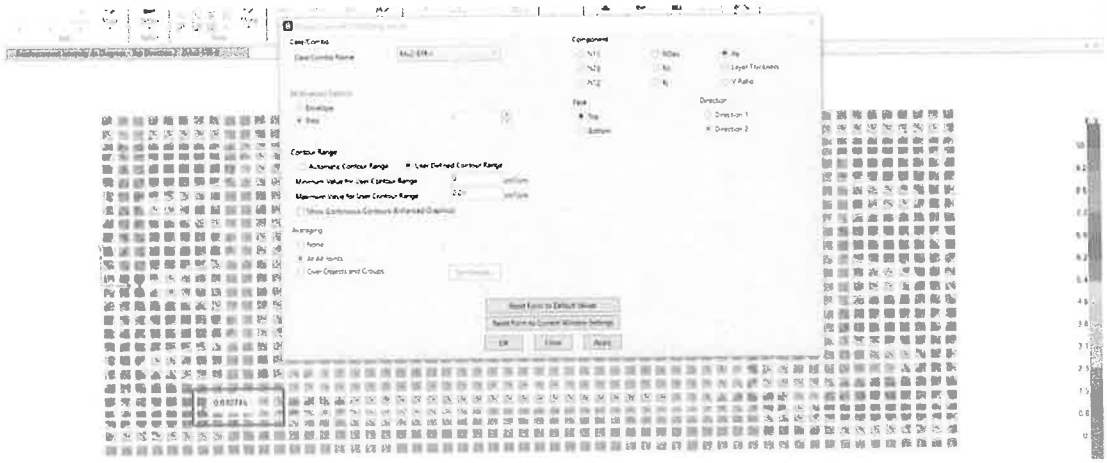
II. **COMBO 2 (C2): Valor extremo de pesos propios: DC-SRT (C2): 1.25 DC + 1.25 Barrera + 1.25 Acera + PRET.**



Valor Extremo: As_C2 = 0,09== 9cm2/m

COMBO 3: Negativos para Armados de Voladizos y armadura superior

C3: 1.25 (DC Barrera Acera) + 1,50 DW + 1.75 PL



Valor extremo: 1,10 cm2 /m -→ Cubierto por D12mm c/10cm.

LIMITES DE Armaduras de refuerzo

Refuerzos mínimos normativos son los siguientes:

- **Armadura de distribución: Art. 9.7.3.2 AASHTO LRFD**
  Armadura principal perpendicular al tráfico como porcentaje de la principal:

$$A_{s1} = \frac{121}{\sqrt{S}} \leq 67\%$$

S = min (180, 131) = 131 cm

$$A_{s1} = \frac{121}{\sqrt{131}} = 11\% \leq 67\%$$

Se deberá disponer de al menos una armadura de reparto igual a 11% para los negativos y positivos:
Negativos: D12 c/10 cm
Distribución: 11,3 cm2/m x 0,11 = 1,1 cm2/m cubierto por la armadura dispuesta igual a D10mm c/15 (Longitudinal reparto para la ppal.).

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- Armadura de retracción y temperatura: El área de refuerzo en cm2 por metro, en cada cara y en cada dirección será al menos;

$$A_{s2} \leq \frac{758.4\ bh}{2(b+h)f_y}$$

Para nuestro material especificado, AP 420, con fy = 4200 kg/cm2 tenemos:

As2 > 1.62 cm2. OK.

Donde:

  - As2 = área de refuerzo en cada cara y dirección en cm2/m.
  - b = ancho de la sección.
  - h = altura de la sección.
  - fy = tensión de fluencia del acero especificado.

- Disposiciones constructivas de los armados según normativa:
  - La separación de armaduras no excederá:
    - 3 veces el espesor de componente: 3x22 = 66cm OK.
    - no mayor a 45 cm. OK.

VERIFICACIÓN DISEÑO DE VIGAS RIOSTRAS:

Para los armados de vigas riostras, se analizó el COMBO 1: Incluye LL efecto.

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Las armaduras por caras obtenidas fueron las siguientes:

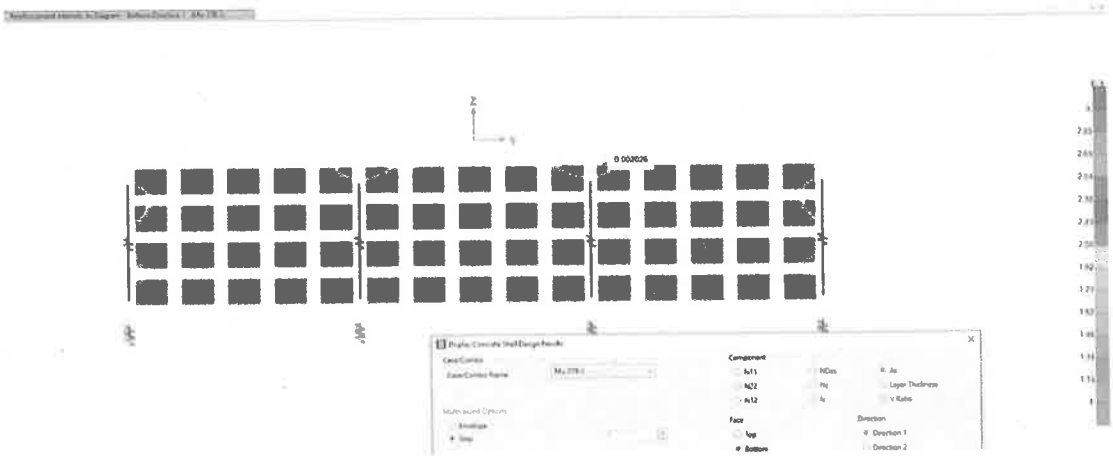

Ilustración 16: Armadura longitudinal - Acero Long. Max. 3cm2/m por cara. Total: 6 cm2 superior: Cubierto por 3d20mm dispuesto.

Para las armaduras de estribos:

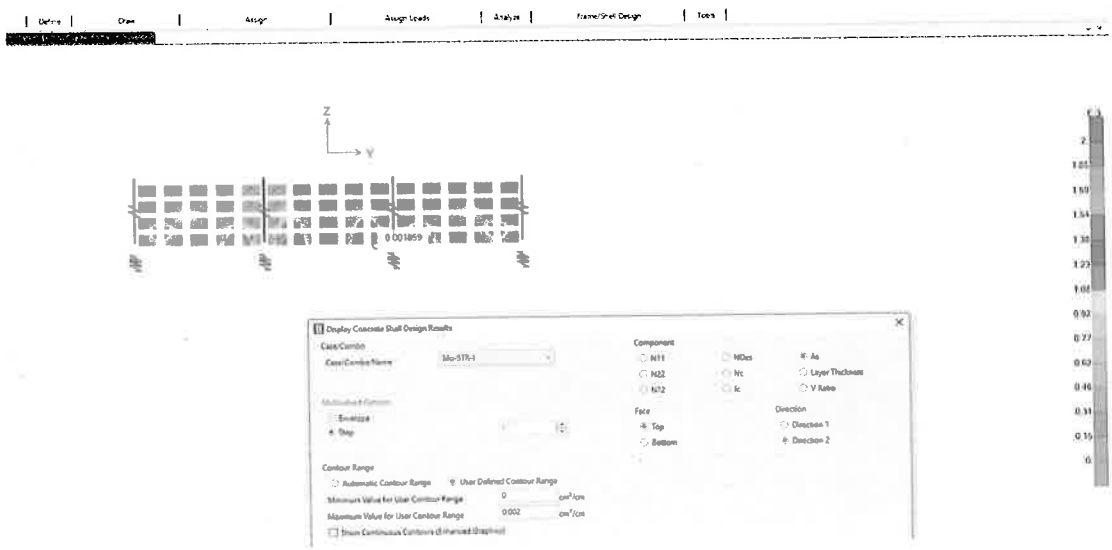

Ilustración 17: Armaduras de Estribos - Cubiertos por ramas simples de estribos d10 c/10cm. Valor por cara Max.: 2 cm2/m.

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55 (cincuenta y cinco)

VERIFICACIÓN DE APARATOS DE APOYOS

En la figura siguiente se observa el aparato de apoyo propuesto:

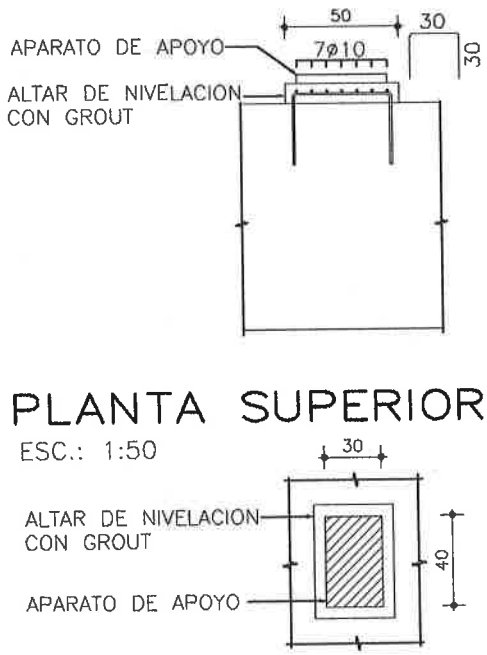

Ilustración 18: Aparato de Apoyo propuesto.

Método de diseño MÉTODO A – Art. 14.7.6 AASHTO LRFD. Aplicable cuando la rotación principal ocurre en un eje paralelo al transversal del puente.

Material especificado para el aparato de apoyo: <u>NEOPRENO DUREZA SHORE A 60.</u>

I. **Módulo de Corte G: 14.06 kg/cm2.**
II. **Deflexión por creep a 25 años dividida por la deflexión inicial: 0.35.**

En estado limite de servicio el esfuerzo de compresión en los aparatos de apoyo rectangulares reforzados con placas de acero se verificará:

$$\sigma_s \leq 1.25\, GS_i \quad \text{Art. 14.7.6.3.2-7}$$

$$\sigma_s \leq 87.9\ \frac{kg}{cm^2} \quad \text{Art. 14.7.6.3.2-8}$$

Siendo:

Si = Factor de forma de una capa interior.

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CARGAS EN ESTADO DE SERVICIO I – DISEÑO APARATOS DE APOYO:

VERTICALES MÁXIMAS:

- **VDC = 34.0 Tonf.**
- **VBarrera = 5.0 Tonf.**
- **VAcera = 3.5 Tonf.**
- **VDW = 4.3 Tonf.**
- **VPL = 3.3 Tonf.**
- **VLL = 40.0 Tonf.**

Total, Vertical: V = 90.1 Tonf.

Tensión máxima de trabajo para el neopreno de diseño: 90100 kg/1200cm2 = 75.1 kg/cm2 < 87.9 kg/cm2 → OK.

Siendo:

Area de neopreno: A = 30x40cm2 = 1200 cm2

MÁXIMA ELONGACIÓN POR CORTE (DEFORMACIÓN LONGITUDINAL DEL TABLERO)

BR: Fuerza de frenado

Máximo entre:

1. **5% peso camión pesado + Carga carril: 2.75 Tonf**
2. **25% peso por eje del camión de diseño: 3.60 Tonf**

BR = 3.60 Tonf.

máxima deformación horizontal del tablero:

$$\Delta = \gamma_{TU}(\Delta Tem\ (\text{Se tomará pos. } 15°C) + \Delta BR)$$

$$\Delta = 1.20\ x\ (0.40 + 0.05) \cong 1.0\ cm.$$

$$\Delta = 1.0\ cm.$$

Espesor requerido del elastómero:

$$h_{rt} \geq 2\Delta = 2x\ 1.0\ cm. = 2.0\ cm.$$

Valor adoptado: 5cm. (50mm).

Factor de forma mínimo Si:

σ = sigma trabajo = 75.1 kg/cm2

$$Si \geq \frac{\sigma}{1.25\ G} = \frac{75.1}{1.25*9.14} = 6.57$$

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Grosor de una capa interior del elastómero:

Del Art. 14.7.5.1-1 de la AASHTO:

$$h_{ri} \leq \frac{\text{Largo x Ancho}}{2*Si*(\text{Largo} + \text{ancho})} = \frac{30*40}{2*6.57*(30+40)} = 1.30 \text{ cm.}$$

Grosor de la capa interior: 13 mm.

Grosor de las capas exteriores:

Se tomará el 70% del valor interior: 0.70 hri = 0.70 x 13mm = 9mm

Grosor de la capa exterior: 9mm

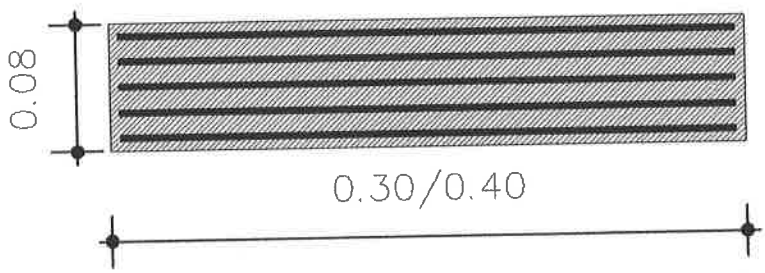
Espesor de las placas de refuerzo de Acero:

$$e \geq \frac{3h_{max.}\sigma}{f_y \text{ (Para acero A-36: Fy} = 2530 \frac{kg}{cm^2})} = \frac{3 \times 1.3cm \times 75.1 \frac{kg}{cm^2}}{2530 \frac{kg}{cm^2}} = 0.11 \text{ cm.}$$

Se adopta de forma satisfactoria un espesor de chapa de acero A-36 igual de al menos 2mm. (1/16").

Numero de capas:

I. **Capas interiores: 4**
II. **Capas exteriores: 2**



CAPAS EXTERIORES: 2 x e = 9 mm
CAPAS INTERIORES: 4 x e = 13 mm
CHAPAS ACERO: 5 x e = 2 mm

Sección en planta: 30x40 cm2.

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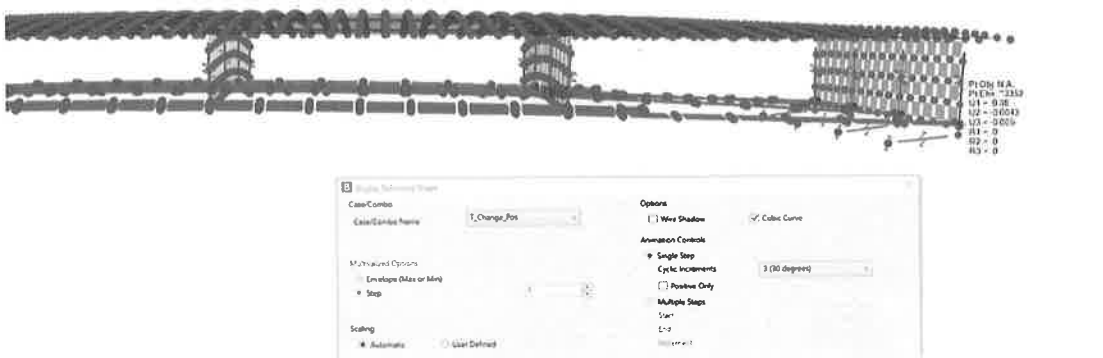

Ilustración 19: Deformación por carga térmica: + 15°C = 0.4 cm.

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