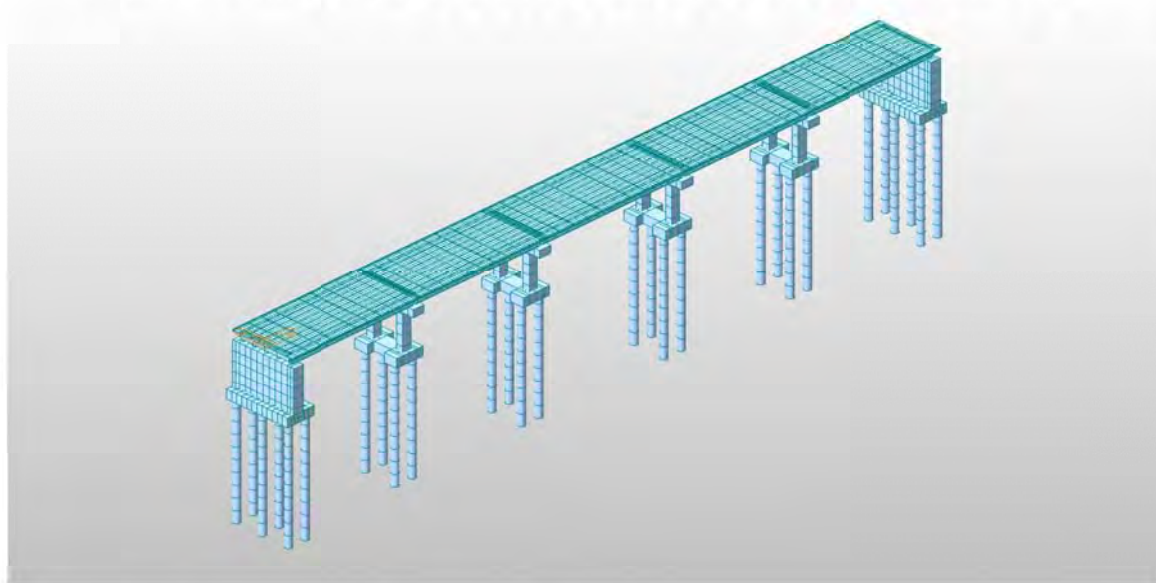


PASAJ KM 18+117 PESTE DJ 592 SI LINII DE TRAMVAI



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1 DESCRIERE SI BAZA DE PROIECTARE

1.1 Descrierea structurii

Varianta de ocolire Timisoara Sud va intersecta la km 18+117 drumul DJ 592. Conform avizului RATT/ 06.08.2010 se tine cont la proiectarea pasajului de viitoarele linii de tramvai. Pasajul este amplasat in plan pe o curba cu raza de 1200 m. Gabaritul de libera trecere pentru sinele de tramvai este de 6000 mm iar gabaritul de libera trecere peste drumuri (sens giratoriu) este de 5.50 m.

Suprastructura pasajului este alcatuita din 9 grinzi prefabricate precomprimate cu armatura preintinsa, avand lungimea de 23.50m , 26.50m si inaltimea de 1.05 m, solidarizate la partea superioara prin intermediul unei placi de suprabetonare din beton armat monolit cu grosimea minima de 15 cm. In zona reazemelor, grinzile sunt incastrate in antretoaze monolite de aprox.2 m latime.

Structura este continuata in dreptul pilelor prin intermediul suprabetonarii, grinzile fiind simplu rezemate pe banchete.

Placa de suprabetonare se va executa din beton armat, turnat direct peste grinzile joantive din componenta tablierului. Marginal, longrinele monolite in consola care sustin parapetele pietonale se vor executa cu cofraj fixat de grinzile marginale. Peste placa de suprabetonare se va aterne hidroizolatie de 1 cm grosime, protectia hidroizolatiei in grosime de 3 cm din BA8 si straturile rutiere in grosime de 2x4 cm din BAP 16 respectiv MAS 16 conform Proiect Ilustrativ.

Rosturile de dilatatie se vor amplasa doar la capete (in zona culeelor).

In sens transversal profilul suprastructurii va fi in "acoperis", cu pante transversale de 2.5% dinspre ax spre trotuare, iar pe zona trotuarelor la nivel panta transversala a acestora este de 1% inspre axul structurii.

In sectiune transversala suprastructura prezinta urmatoarele caracteristici: 7.80 m parte carosabila, incadrata cu parapete New Jersey 0.60m, doua trotuare la nivel de 1.00 m latime utila fiecare, si longrine cu latimea de 0.30 m, latimea totala a suprastructurii fiind de 11.60 m.

Conform proiectului ilustrativ, la marginea partii carosabile au fost prevazute parapete de siguranta de tip rigid (parapete New Jersey din beton armat cu grad de asigurare H4b) pentru protectia pietonilor iar pe partile exterioare ale trotuarelor se vor prevedea parapete pietonale metalice.

Infrastructurile sunt prevazute a fi executate din beton armat, fundate indirect, pe piloti forati de diametru mare $d=1200$ mm, avand capetele solidarizate la partea superioara cu radiere din beton armat .

Culeele vor avea elevatie de tip perete din beton armat.

Pilele vor avea elevatii "pline" (de tip perete din beton armat) cu evazaj sub forma de arc de cerc la partea superioara unde se va executa rigla din beton armat.

Pe banchetele infrastructurilor se vor executa cuzineti si dispozitive antiseismice din beton armat.

Racordarea cu terasamentele se va realiza cu ziduri de sprijin din pamant armat. In spatele culeelor, pentru drenarea apelor din infiltratii se vor executa drenuri din piatra bruta invelite in geotextil. Pe partea carosabila pentru tranzitia de la tablierul

podului la rambleul drumului s-au prevazut placi de racordare din beton armat cu lungimea de 6.00 m.

Urmatoarele faze de executie sunt considerate:

- Executarea piloților și a radierelor;
- Executarea elevațiilor pentru culee și pile;
- Poziționarea grinzilor;
- Poziționarea predalelor, armarea plăcii de suprabetonare și a antretoazelor;
- Betonarea plăcii de suprabetonare și a antretoazelor;
- Executarea terasamentelor;
- Executarea caili, parapetelor, a lisei și a bordurilor.

Clasa de incarcare: Eurocode: LM1

Clasa de importanta: "C"

Cerinte de calitate: A4, B2, D2.

Proiectare seismică: $a_g = 0.20g$, $T=0.7s$ conform P100/2013

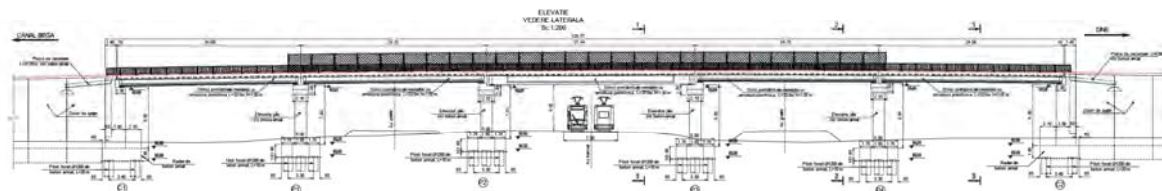


Fig. 1 Elevatie

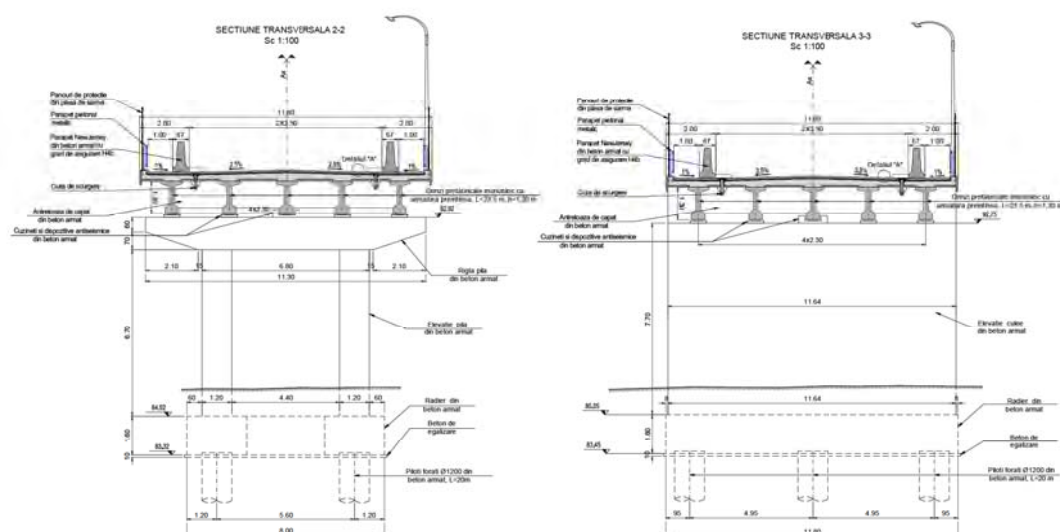


Fig. 2 Sectiuni transversale

1.2 Baza de proiectare si materiale

Instrucțiuni și standarde considerate în proiectarea structurii:

- Eurocode 1: Acțiuni asupra structurilor
- Eurocode 2: Proiectarea structurilor de beton
- Eurocode 7: Proiectarea geotehnica
- Eurocode 8: Proiectarea pentru rezistența la cutremur a structurilor
- P100/2013: Cod de proiectare seismică

Caracteristicile minime ale materialelor:

- Otel pentru armatura: BST 500 C
- Otel pentru precomprimare: Y 1860
- Beton in coloane: C25/30
- Beton in radier culee si pile: C25/30
- Beton in elevatii culee si pile: C30/37
- Beton pentru turnarea placii in-situ, grinzile de parapet: C35/45;
- Beton pentru grinzi prefabricate: C50/60

2 ACTIUNI

2.1 Actiuni considerate

In concordanta cu Eurocode 1, actiunile considerate in proiectarea structurii sunt:

- Actiuni permanente (G):
 - Greutatea proprie a grinzilor, placilor, elevatiei, radierelor si a pilotilor;
 - Greutatea caii, parapetelor, trotuar;
 - Impingerea pamantului;
 - Impingerea pamantului din seism;
- Actiuni variabile (Q):
 - Actiuni din trafic la poduri (SR EN 1991-2:2005) (convoiul LM1, incarcari din oameni pe trotuare, franarea si forta centrifuga);
 - Actiuni termice, temperatura (T-) si T(+);
- Actiuni accidentale (A):
 - Actiuni seismice, Seism X (longitudinal), Seism Y (transversal) , Seism Z (vertical).

2.2 Valori caracteristice

2.2.1 Actiuni permanente (G).

2.2.1.1 Greutatea proprie a elementelor structurale

Element	buc	Arie m ²	Densitate kN/m ³	Gn kN/m	γ	Gc kN/m
Grinzi prefabricate H=1.30m	5	0.467	25	58.38	1.35	78.81
Placa grinzi marginale	2	0.578	25	28.90	1.35	39.02
Placa grinzi centrale	3	0.731	25	54.83	1.35	74.01
Cale	1	1.3464	24	32.31	1.35	43.62
Lisa prefabricata	2	0.043	25	2.15	1.35	2.90
Parapet directiona	2	-	-	11.00	1.35	14.85
Parapet pietonal	2	-	-	1.50	1.35	2.03
Grinda parapet	2	0.042	25	6.50	1.35	8.78
TOTAL / ml				195.56		264.01

2.2.1.2 Impingerea pamantului

$h=$	8	m	Inaltime elevatie
$h_s=$	1.3	m	inaltimea echivalenta suprasarcina
$H_T=$	9.3	m	Inaltime totala
$\phi_{df}=\phi_{kf}=$	33	grade	
$g_{kd}=g_{kf}=$	19	kN/m ³	
$K_{ad}=\tan(\pi/4-\phi_{df}/2)^2=$	0.333	coef. presiune activa	

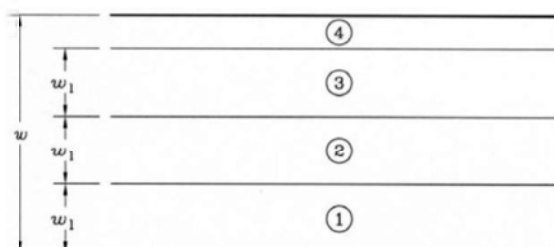
2.2.2 Actiuni variabile (Q)

2.2.2.1 Acțiuni din trafic la poduri (SR EN 1991-2:2005);

Pentru determinarea efectelor zonelor de incarcarea a partii carosabile a unui pod aceasta este divizata in benzi teoretice.

Lățimea părții carosabile w	Numărul benzilor teoretice de circulație	Lățimea unei benzi teoretice de circulație	Lățimea zonei rămase
$w < 5,4 \text{ m}$	$n_l = 1$	3 m	$w - 3 \text{ m}$
$5,4 \text{ m} \leq w < 6 \text{ m}$	$n_l = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_l = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTĂ – De exemplu, pentru o parte carosabilă cu lățimea de 11 m, $n_l = \text{Int}\left(\frac{w}{3}\right) = 3$, și lățimea zonei rămase este: $11 - 3 \times 3 = 2 \text{ m}$.			

Numărul, poziția și numerotarea benzilor teoretice încărcate, se realizează astfel incat efectele convoaielor de calcul să fie cele mai defavorabile in scopul unei verificări individuale.



w =Lățimea părții carosabile

w_l =Lățimea unei benzi teoretice de circulație

1=Banda teoretică nr.1

2=Banda teoretică nr.2

3=Banda teoretică nr.3

4=Zona rămasă

2.2.2.1.1 Acțiuni verticale

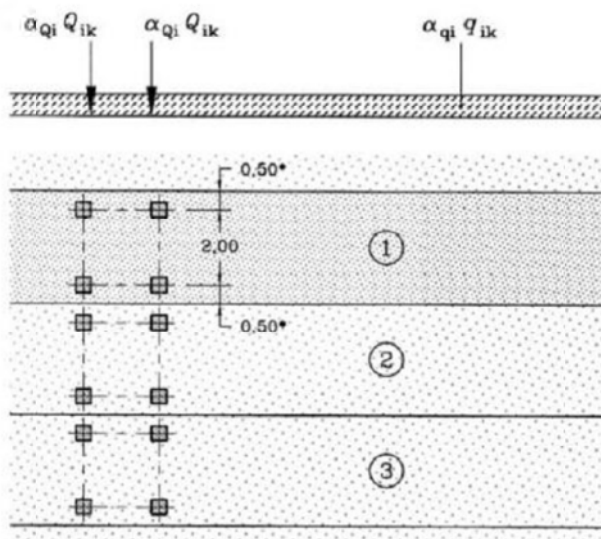
Convoiul de calcul 1 (LM1) :

Acest convoi este alcătuit din încărcări concentrate și încărcări uniform distribuite ce acoperă cele mai multe efecte din traficul alcătuit din camioane și automobile, fiind utilizat atât pentru verificări locale cât și globale.

Încărcările concentrate sunt reprezentate de sistemul tandem TS alcătuit din 2 osii cu greutatea " $\alpha_Q Q_k$ " pe fiecare osie.

Notă: atât pentru verificări locale cât și generale se va aplica un singur sistem tandem pe fiecare bandă de circulație cu precizarea că poziționarea acestuia se va face centrat pe axul fiecărei benzi, în cazul verificărilor globale, și în poziția cea mai defavorabilă în cazul celor locale, fără însă a depăși limita de 0.5 m între două sisteme de pe două benzi adiacente.

Încărcările uniform distribuite sunt reprezentate de sistemul UDL cu greutatea " $\alpha_q q_k$ " pe metru pătrat;



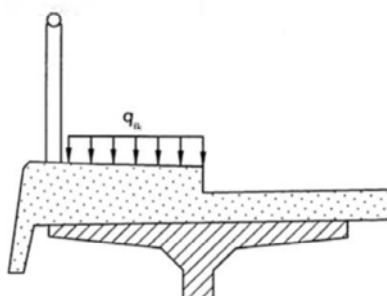
Coeficienții de corecție pentru încărcările uniform distribuite și concentrate " α_q " și " α_Q " se iau egali cu 1 (conform secțiunea 4-4.3.2(3)-SR EN 1991-2:2004/NB:2006).

Poziția	Sistemul tandem TS	Sistemul UDL
	Încărcarea pe osie Q_{ik} (kN)	q_{ik} (sau q_{rk}) (kN/m ²)
Banda numărul 1	300	9,0
Banda numărul 2	200	2,5
Banda numărul 3	100	2,5
Alte benzi	0	2,5
Zona rămasă (q_{rk})	0	2,5

Nota: Valorile încărcărilor prezentate în tabel includ amplificarea dinamică.

Încărcarea uniform distribuită

Pentru podurile de șosea care susțin trotuare pentru pietoni sau piste pentru cicliști, se definește o forță uniform distribuită q_{fk} .



Valoarea $q_{fk}=3\text{kN/mp}$ pentru gruparea gr1a.

2.2.2.1.2 Forțe orizontale

Forța de frânare (SR EN 1991-2:2005)

Forța de frânare " Q_{lk} " este o forță ce acționează în sens longitudinal podului la nivelul suprafeței de rulare a părții carosabile.

Valoarea caracteristică Q_{lk} este limitată la 900 kN pentru întreaga lățime a podului, și trebuie calculată ca fracțiune din totalul sarcinilor verticale maxime corespunzătoare convoiului LM1, aplicabile benzii nr.1 după cum urmează:

$$Q_{lk}=0.60\alpha_{Q1} (2Q_{1k})+0.10 \alpha_{q1}q_{1k}w_1L \leq 900(\text{kN})$$

L =lungimea tablierului sau a părții considerate in calcul.

Pentru cazurile curente cand lungimea de încărcare $L > 1,2m$ forța de franare este egală cu $Q_{lk} = 360 + 2,7L \leq 900$ (kN).

Limita inferioară a forței de franare la podurile de șosea este de 300kN.

$L_{cele} =$	123.6	m	$w_l =$	3	m
$Q_{lk} =$	300	kN	$\alpha_{Q1} =$	1	
$q_{lk} =$	9	kN/m ²	$\alpha_{q1} =$	1	
$Q_{lk} =$	693.72	kN	< 900	kN	
$Q_{lk} =$	693.72	kN			
Nr Grinzi	6				
$Q_{lk} / grinda =$	115.62	kN - forta orizontala			

2.2.2.2 Actiuni termice (SR EN 1991-1-5:2005), temperatura (T-) si T(+)

Valoarea caracteristică a componentei de temperatură uniformă este asociată fie contracției, fie dilatării fiind notată cu $\Delta_{TN,contractie}$ sau cu $\Delta_{TN,dilatare}$ pentru fiecare caz in parte.

$$\Delta_{TN,contractie} = T_0 - T_{e,min} \quad \text{- temperatura, } T(-)$$

$$\Delta_{TN,dilatare} = T_{e,max} - T_0 \quad \text{- temperatura, } T(+)$$

T_0 - reprezintă temperatura inițială, se utilizeaza valoarea recomandata 10°C (SR EN 1991-1-5:2004/NA:2008).

$T_{e,min}$ și $T_{e,max}$ reprezintă componentele de temperatură uniformă minimă și maximă.

$T_{e,min} = -32^\circ C$ si $T_{e,max} = 42^\circ C$ – conf. SR EN 1991-1-5:2004/NA:2008, tabel NA.1(RO).

2.2.3 Actiuni seismice (S)

Amplasamentul structurii este caracterizat de acceleratia maxima a terenului $a_g = 0.20g$ si perioada de colt $T_c = 0.7s$.

Pentru calculul forței seismice se respectă prevederile:

- SR EN 1998-1 – Proiectarea structurilor pentru rezistența la cutremur. Reguli generale, acțiuni seismice și reguli pentru clădiri;
- SR EN 1998-2 – Proiectarea structurilor pentru rezistența la cutremur. Poduri;
- SR EN 1998-2/NA – Proiectarea structurilor pentru rezistența la cutremur. Poduri. Anexa națională.
- P100/2013 – Cod de proiectare seismică.

Conform SE EN 1998-2/NA – clasa de importanță este "III" si $\gamma_I=1,3$ - coeficient de amplificare a forței seismice.

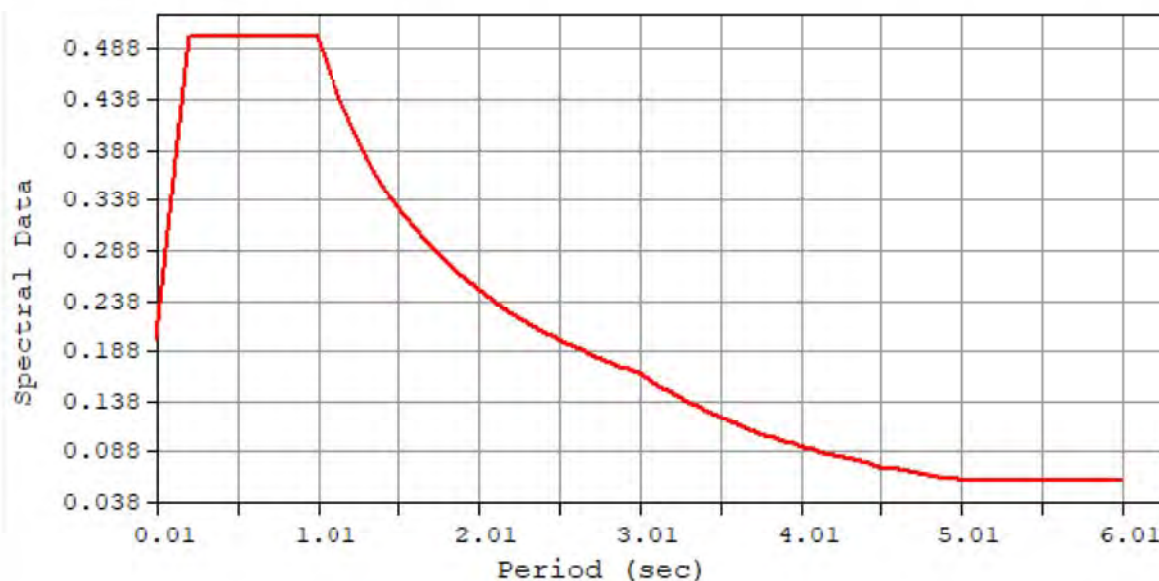


Fig. 3 Spectru de raspuns seismic orizontal

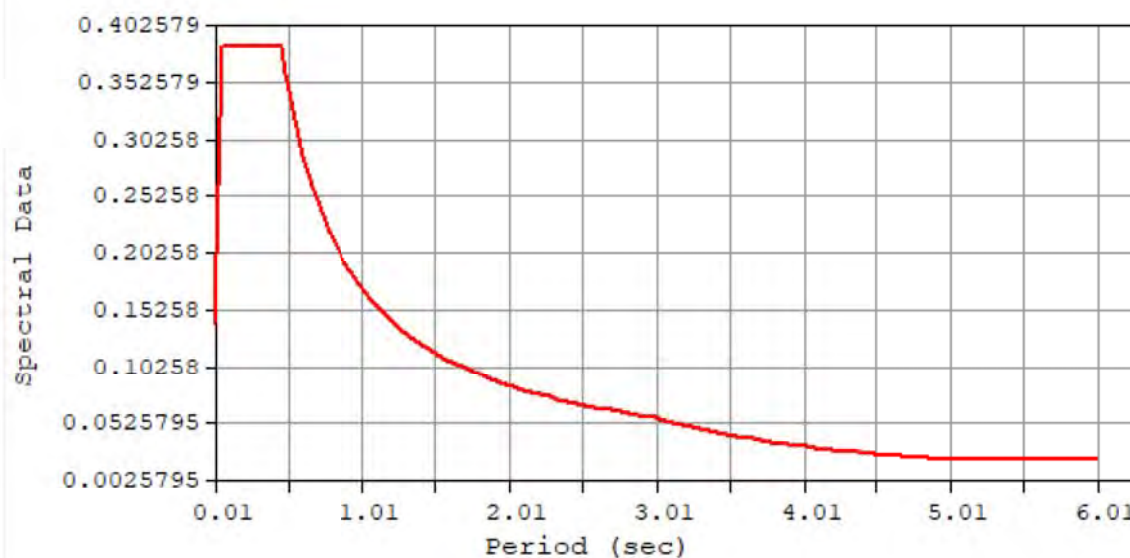


Fig. 4 Spectru de raspuns seismic vertical

3 COMBINAREA ACȚIUNILOR

Conform SR EN 1990 , Anexa A2 combinațiile sunt următoarele:

- Combinarea fundamentală din acțiuni persistente (permanente) și tranzitorii;
- Combinarea acțiunilor pentru situații accidentale;
- Combinarea acțiunilor pentru calcul seismic;

Coeficientii care intervin la combinarea acțiunilor pentru starea limită ultimă sunt:

$g_1=1,35$ – pentru acțiuni permanente (greutate proprie elemente structurale, împingere pământ);

$g_2=1,35$ – pentru acțiunea din convoi LM1 și încărcarea uniform distribuită pe trotuar;

Ψ_{i1} -0.75 pentru gruparea 1a – sistem TS tandem;

Ψ_{i2} -0.40 pentru gruparea 1a – sistem UDL forțe uniform distribuite

Ψ_{i3} -0.40 pentru gruparea 1a – încărcarea uniform distribuită pe trotuare

DESIGN TYPE : Concrete Design

LIST OF LOAD COMBINATIONS

NUM	NAME	ACTIVE	TYPE	LOADCASE(FACTOR) +	LOADCASE(FACTOR) +	LOADCASE(FACTOR)
=====						
1	cLCB1	Strength/Stress	Add			
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)		
+		Wearing Surface(1.350) +	Utility1(1.350) +	EarthLoad(1.350)		
+		EarthPressure(1.350) +	ML-caract(1.350)			

2	cLCB2	Strength/Stress	Add			
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)		
+		Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)		
+		EarthPressure(1.000) +	ML-caract(1.350)			

3	cLCB3	Strength/Stress	Add			
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)		
+		Wearing Surface(1.350) +	Utility1(1.350) +	EarthLoad(1.350)		
+		EarthPressure(1.350) +	Temperatura (+)(1.500)			

4	cLCB4	Strength/Stress	Add			
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)		
+		Wearing Surface(1.350) +	Utility1(1.350) +	EarthLoad(1.350)		
+		EarthPressure(1.350) +	Temperatura (+)(-1.500)			

5	cLCB5	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
		+ Wearing Surface(1.350) +	Utility1(1.350) +	EarthLoad(1.350)
		+ EarthPressure(1.350) +	Temperatura (-)(1.500)	
6	cLCB6	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
		+ Wearing Surface(1.350) +	Utility1(1.350) +	EarthLoad(1.350)
		+ EarthPressure(1.350) +	Temperatura (-)(-1.500)	
7	cLCB7	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	Temperatura (+)(1.500)	
8	cLCB8	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	Temperatura (+)(-1.500)	
9	cLCB9	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	Temperatura (-)(1.500)	
10	cLCB10	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	Temperatura (-)(-1.500)	
11	cLCB11	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	ML-caract(1.000)	
12	cLCB12	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	ML-caract(1.000) +	Temperatura (+)(0.600)
13	cLCB13	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
		+ EarthPressure(1.000) +	ML-caract(1.000) +	Temperatura (+)(-0.600)
14	cLCB14	Serviceability	Add	

			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	ML-caract(1.000) +	Temperatura (-)(0.600)
<hr/>					
15	cLCB15	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	ML-caract(1.000) +	Temperatura (-)(-0.600)
<hr/>					
16	cLCB16	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	Temperatura (+)(1.000)	
<hr/>					
17	cLCB17	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	Temperatura (+)(-1.000)	
<hr/>					
18	cLCB18	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	Temperatura (-)(1.000)	
<hr/>					
19	cLCB19	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	Temperatura (-)(-1.000)	
<hr/>					
20	cLCB20	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	ML-frecv(1.000) +	Temperatura (+)(0.500)
<hr/>					
21	cLCB21	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	ML-frecv(1.000) +	Temperatura (+)(-0.500)
<hr/>					
22	cLCB22	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+			EarthPressure(1.000) +	ML-frecv(1.000) +	Temperatura (-)(0.500)
<hr/>					
23	cLCB23	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)

+	EarthPressure(1.000) +	ML-frecv(1.000) +	Temperatura (-)(-0.500)
24	cLCB24 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (+)(0.600)	
25	cLCB25 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (+)(-0.600)	
26	cLCB26 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (-)(0.600)	
27	cLCB27 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (-)(-0.600)	
28	cLCB28 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (+)(0.500)	
29	cLCB29 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (+)(-0.500)	
30	cLCB30 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (-)(0.500)	
31	cLCB31 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000) +	Temperatura (-)(-0.500)	
32	cLCB32 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	EarthLoad(1.000)
+	EarthPressure(1.000)		

33 Seism_X Strength/Stress Add
 Self Weight(1.000) + Barrier(1.000) + Additional Load1(1.000)
 + Wearing Surface(1.000) + Utility1(1.000) + EarthPressure(1.000)
 + EarthLoad(1.000) + Seism_X(1.000) + Seism_Y(0.300)
 + ML-caract(0.300) + Temperatura (-)(0.500)

34 Seism_Y Strength/Stress Add
 Self Weight(1.000) + Barrier(1.000) + Additional Load1(1.000)
 + Wearing Surface(1.000) + Utility1(1.000) + EarthPressure(1.000)
 + EarthLoad(1.000) + Seism_X(1.000) + Seism_Y(0.300)
 + ML-caract(0.300) + Temperatura (-)(0.500)

4 ANALIZA STRUCTURALA

4.1 Modelul structural

Analiza globală a fost realizată cu ajutorul programului de analiză element finit MIDAS Civil 2020. Suprastructura a fost modelată cu elemente tip bară legate cu link-uri elastice. Culeele se consideră încastrate la nivelul radierului. Analiza seismică - calculul modal cu spectre de răspuns.

4.1.1 Caracteristicile materialelor utilizate:

Tabel 1 Caracteristici materiale

ID	Name	Type	Standard	DB	Density (kN/cm ³)
1	C50/60	Concrete	EN04 (RC)	C50/60	2.5000e-005
2	C35/45	Concrete	EN04 (RC)	C35/45	2.5000e-005
3	Y1860S7 (15.2mm)	Steel	EN05-PS (S)	Y1860S7 (15.2mm)	7.6980e-005
4	C25/30	Concrete	EN04 (RC)	C25/30	2.5000e-005
5	C30/37	Concrete	EN04 (RC)	C30/37	2.5000e-005
6	Dummy Material	User Defined	None	C35/45	0.0000e+000

Geometria structurii

Modelarea suprastructurii, elevatiei a fost realizata cu elemente de tip grinda (frame) pentru modelarea tablierului. (Fig. 5).

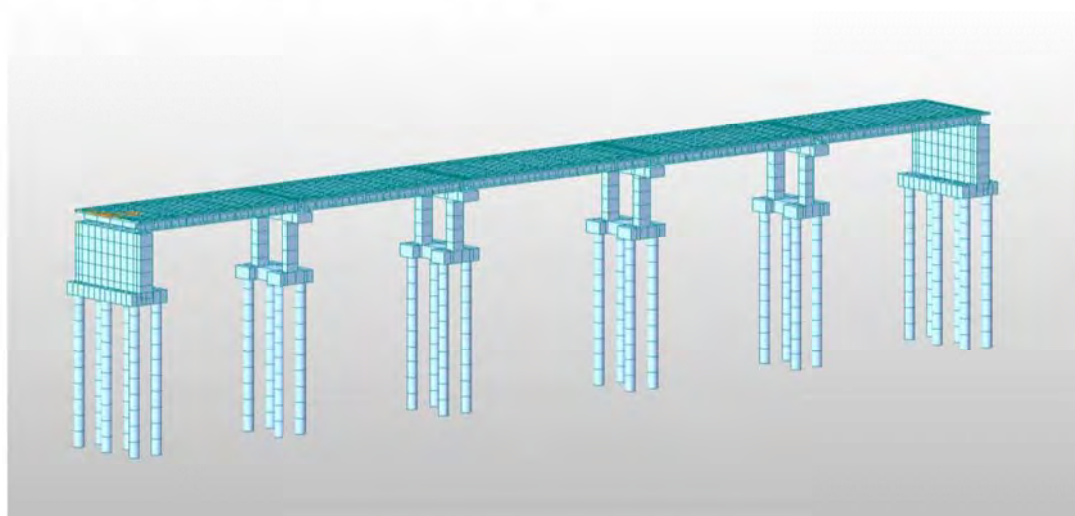


Fig. 5 Modelare structurala 3D

4.2 Centralizarea rezultatelor și verificarea secțiunilor

4.2.1 Verificarea capacității portante ultime a piloților forajați

4.2.1.1 Piloți Culee



Fig. 6 Solicitări axiale maxime la piloții din culee

Fcd= 3609 kN – valoarea de calcul a încărcării axiale de compresiunea asupra unui pilot corespunzător stării limite ultime – Gruparea cLCB1

Rcd= 4033 kN – capacitatea portantă ultimă la compresiune a pilotului, determinată cu programul de calcul GeoStru, anexat breviarului.

$$F_{cd} < R_{cd} - \text{Adevărat}$$

4.2.1.2 Piloți Pile

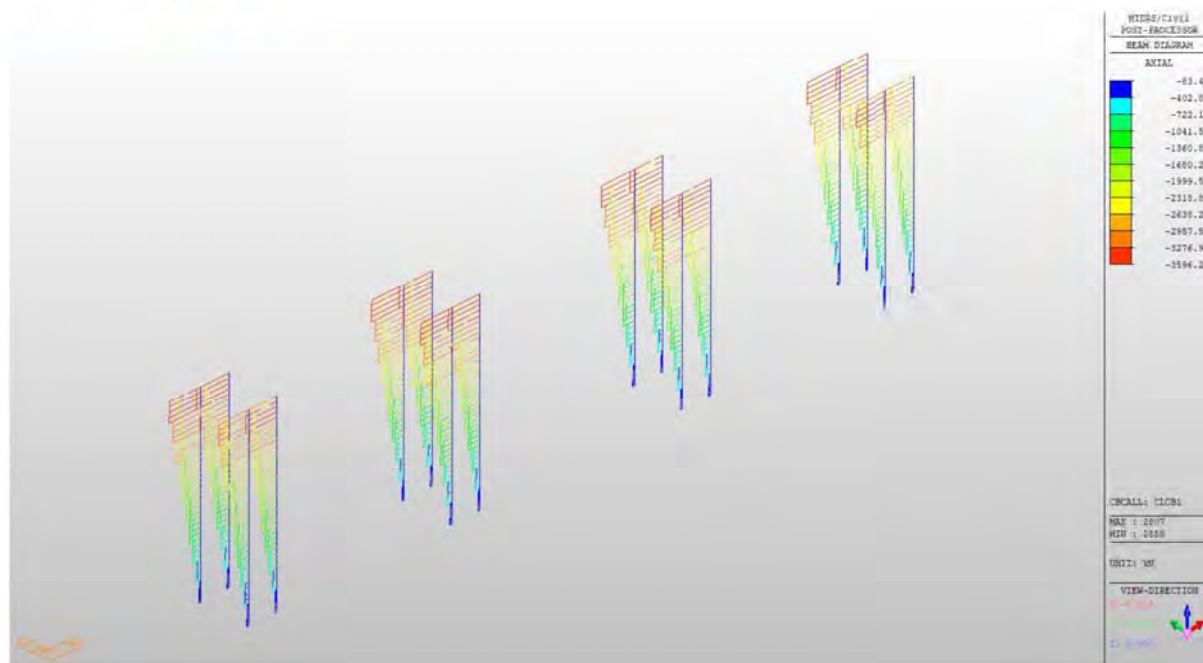


Fig. 7 Solicitări axiale maxime la piloții din pile

Fcd= 3596 kN – valoarea de calcul a încărcării axiale de compresiune asupra unui pilot corespunzător stării limite ultime – Gruparea CLCB1

Rcd= 4033 kN – capacitatea portantă ultimă la compresiune a pilotului, determinată cu programul de calcul GeoStru, anexat breviarului.

Fcd < Rcd – Adevărat

4.2.2 Dimensionare armătură piloți la moment încovoietor și forță tăietoare

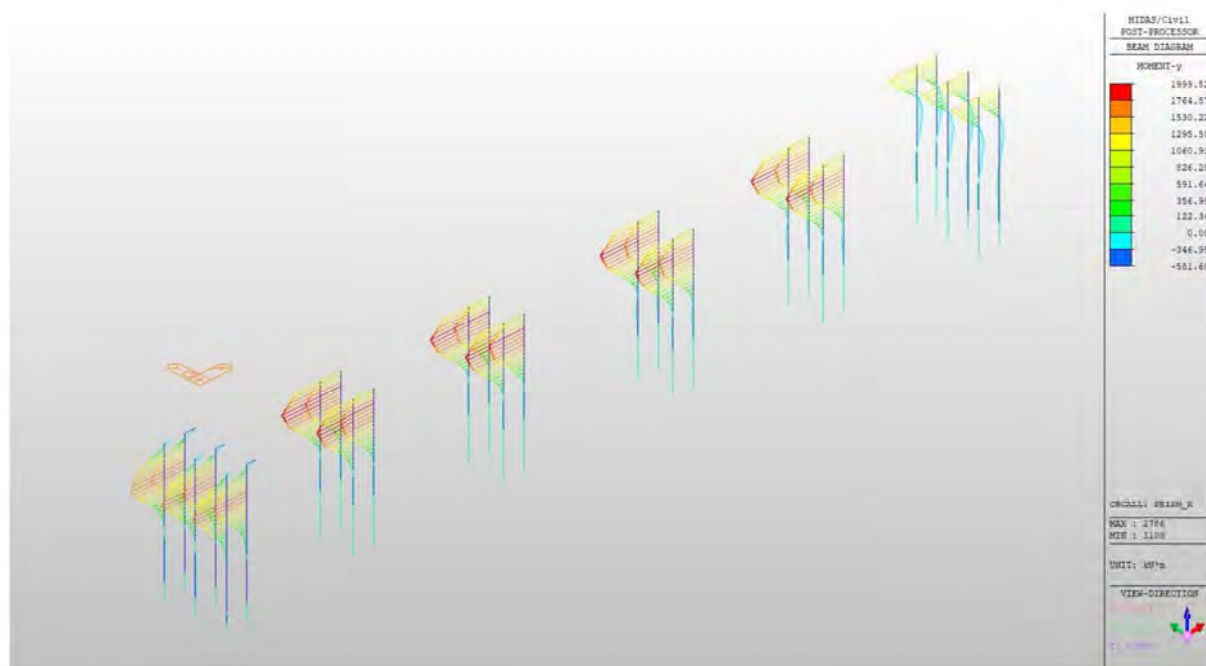


Fig. 8 Diagrama de moment încovoietor din combinația seismică pe direcția X

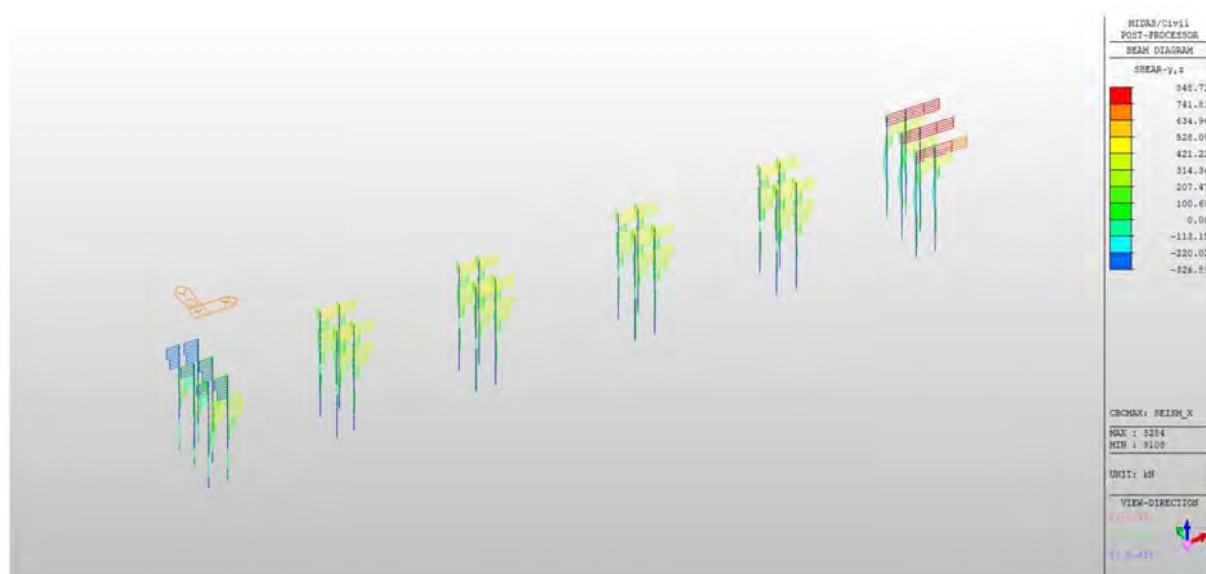


Fig. 9 Diagrama de forță tăietoare din combinația seismică pe direcția X

PILOT

MIDAS/Civil

RC Column Checking Result

	Company		Project Title	
	Author	Atti	File Name	C:\...\Str4 - 5 grnzi in sect.mcb

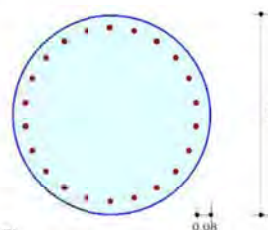
1. Design Condition

Design Code Eurocode2-2:05
 Unit System kN, m
 Member Number 2790 (PM), 2940 (Shear)
 Material Data fck = 30000, fyk = 500000, fyw = 500000 KPa
 Column Height 2.08 m
 Section Property Pilot (No : 6)

Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	22-P25	—	—

Total Rebar Area Ast = 0.0107891 m² (Rhost = 0.0095)



2. Applied Loads

Load Combination 33+ AT (I) Point

N_{Ed} = -148.88 kN, M_{Edy} = 1999.52, M_{Edz} = 346.591, M_{Ed} = 2029.33 kN-m

3. Axial Forces and Moments Resistance Check

Concentric Max. Axial Load N_{Rdmax} = 21833.9 kN

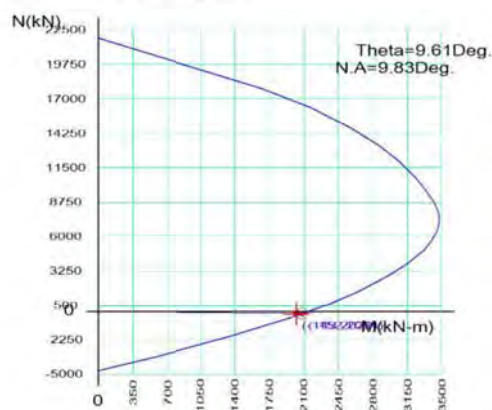
Axial Load Ratio N_{Ed}/N_{Rd} = -148.88 / -151.74 = 0.981 < 1.000 O.K

Moment Ratio M_{Edy}/M_{Rdy} = 1999.52 / 2054.37 = 0.973 < 1.000 O.K

M_{Edz}/M_{Rdz} = 346.591 / 347.938 = 0.996 < 1.000 O.K

M_{Ed}/M_{Rd} = 2029.33 / 2083.62 = 0.974 < 1.000 O.K

4. P-M Interaction Diagram



N _{Rd} (kN)	M _{Rd} (kN-m)
21833.94	0.00
16283.25	2178.87
13318.49	2860.43
10795.02	3229.97
8745.43	3424.38
7194.37	3476.98
6036.00	3426.75
4978.19	3325.91
3824.18	3149.95
2427.16	2854.25
588.64	2334.51
-1915.22	1372.10
-4695.28	0.00

5. Shear Force Resistance Check

Applied Shear Resistance V_{Ed} = 521.160 kN (Load Combination 33+)

Shear Strength by Conc V_{Rdc} = 0.00000 kN

Shear Resistance by Rebar V_{Rds} = 558.371 kN (2.0-P10 @90)

Shear Ratio V_{Ed}/V_{Rd} = 521.160 / 558.371 = 0.933 < 1.000 O.K

4.2.3 DIMENSIONARE ARMĂTURĂ RADIER ȘI ELEVATIE CULEE

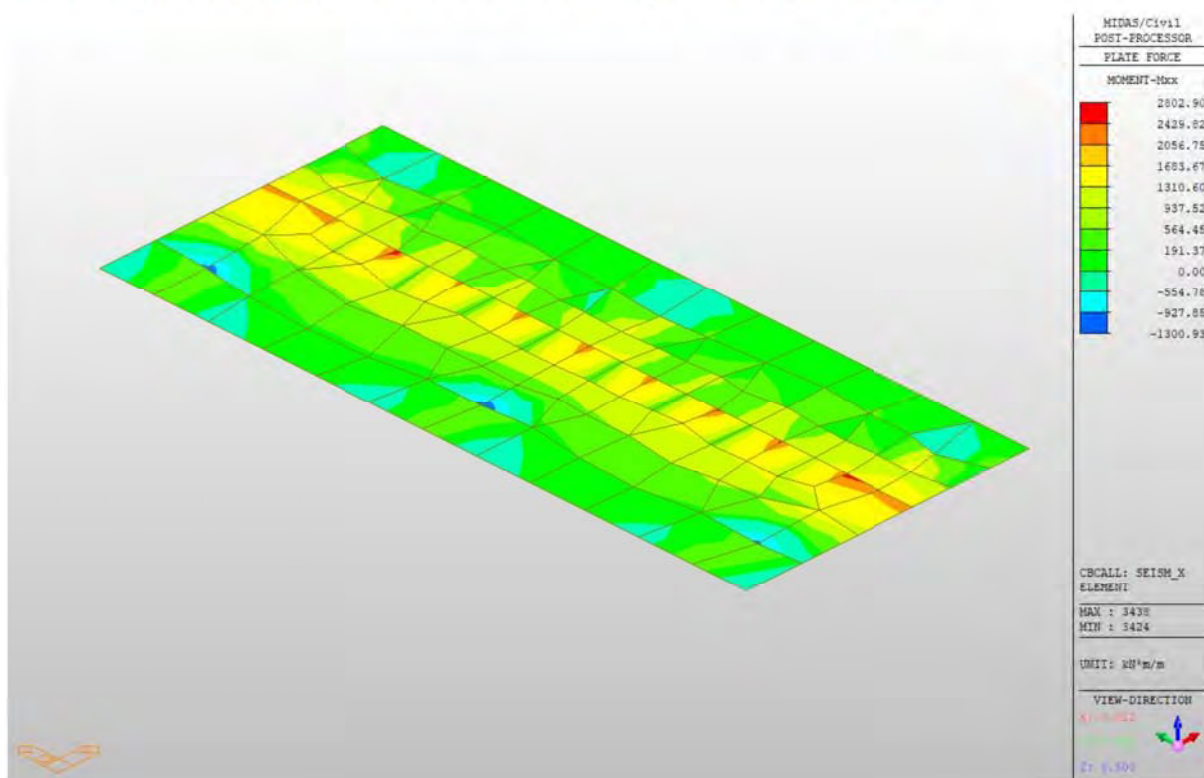


Fig. 10 Moment încovoietor maxim în radier culee

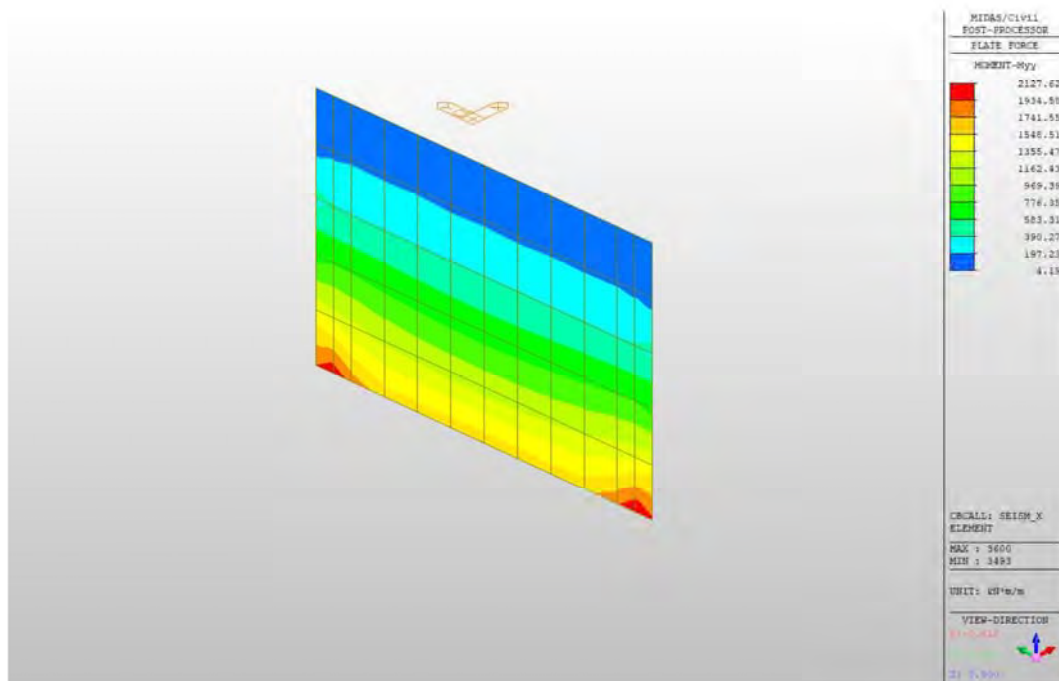


Fig. 11 Moment încovoietor maxim în elevația culeei

RADIER CULEE

MIDAS/Civil

Plate Beam Check Result Dialog

MIDAS	Company		Project Title	
	Author	Atti	File Name	C:\...\Str4 - 5 grinzi in sect.mcb

1. Design Condition

Design Type : Plate Beam (1D)
 Sub-Domain : racier
 Design Code : Eurocode2-2.05
 Unit System : kN, m, / m
 Material Data : fck = 30000, tyk = 500000, fyw = 500000 KPa
 Thickness : 1.6 m

2. Section Diagram



Element No : 3438

Rebar Pattern	Top(Negative)	Bottom(Positive)
Layer 1	P25@0.15	P32@0.15
Total Rebar Area Ast = 0.00863413 m ² /m		
Using Stirrups Spacing : 4.0-P20 @200		

3. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
Mu	1986.44	2802.90
Element No.	3239	3438
Load Combination	cLCB1	Seism_X
Mr	2141.41	3448.28
Check Ratio (Mu/Mr)	0.9276	0.8128
Using Rebar(As)	0.0033	0.0054

4. Shear Capacity

Element No. : 3417
 Load Combination : cLCB1
 Applied Shear Force : V_Ed = 551.208
 Shear Strength (Out of plane) V_Rdc = 551.437 V_Rds = 0.00000
 Shear Ratio : V_Ed/V_Rd = 551.208 / 551.437 = 1.000 < 1.000 O.K

ELEVAȚIE CULEE

MIDAS/Civil

Plate Column Check Result Dialog



Company		Project Title	
Author	Atti	File Name	C:\...\Str4 - 5 girzi in sect.mcb

1. Design Condition

Design Type	Plate Column (1D)						
Sub-Domain	elev-culee						
Design Code	Eurocode2-2.05						
Unit System	kN, m, / m						
Material Data	fck = 30000. fyk = 500000. fyw = 500000 KPa						
Rebar Pattern	<table><tr><td></td><td>Top(Negative)</td><td>Bottom(Positive)</td></tr><tr><td>Layer 1</td><td>P20@0.15</td><td>P25@0.15</td></tr></table>		Top(Negative)	Bottom(Positive)	Layer 1	P20@0.15	P25@0.15
	Top(Negative)	Bottom(Positive)					
Layer 1	P20@0.15	P25@0.15					
Total Rebar Area	Ast = 0.00536687 m²/m						
Using Stirrups Spacing	2.0-P12 @150						



2. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
M _{Ed}	1325.27	2127.62
Element No.	3552	3600
Load Combination	cLCB1	Seism_X
M _{Rd}	1567.34	2426.81
Check Ratio (M _{Ed} /M _{Rd})	0.8456	0.8767
Using Rebar(As)	0.0021	0.0033

3. Axial Force Resistance Check

Element No.	3552
Load Combination	cLCB1
Applied Axial Force Strength N _{Ed}	= 2174.01
Axial Force Strength N _{Rd}	= 32842.2
Axial Force Ratio	N _{Ed} /N _{Rd} 2174.01 / 32842.2 = 0.066 < 1.000 O.K

4. Shear Capacity

Element No.	3552
Load Combination	Seism_X
Applied Shear Force V _{Ed}	= 458.042
Shear Strength (Out of plane) V _{Rdc}	= 519.255
Shear Ratio	V _{Ed} /V _{Rdc} 458.042 / 519.255 = 0.882 < 1.000 O.K

4.2.4 DIMENSIONARE ARMĂTURĂ RADIER ȘI STÂLP PILĂ

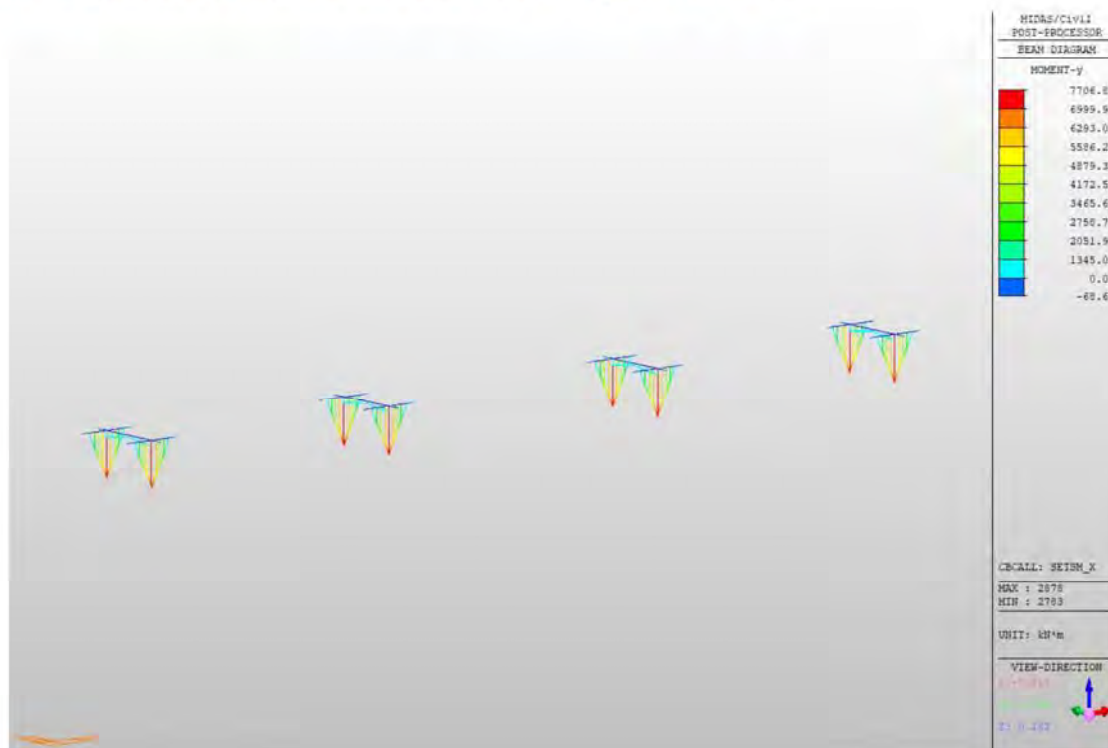


Fig. 12 Moment încovoietor maxim în radierul pilei

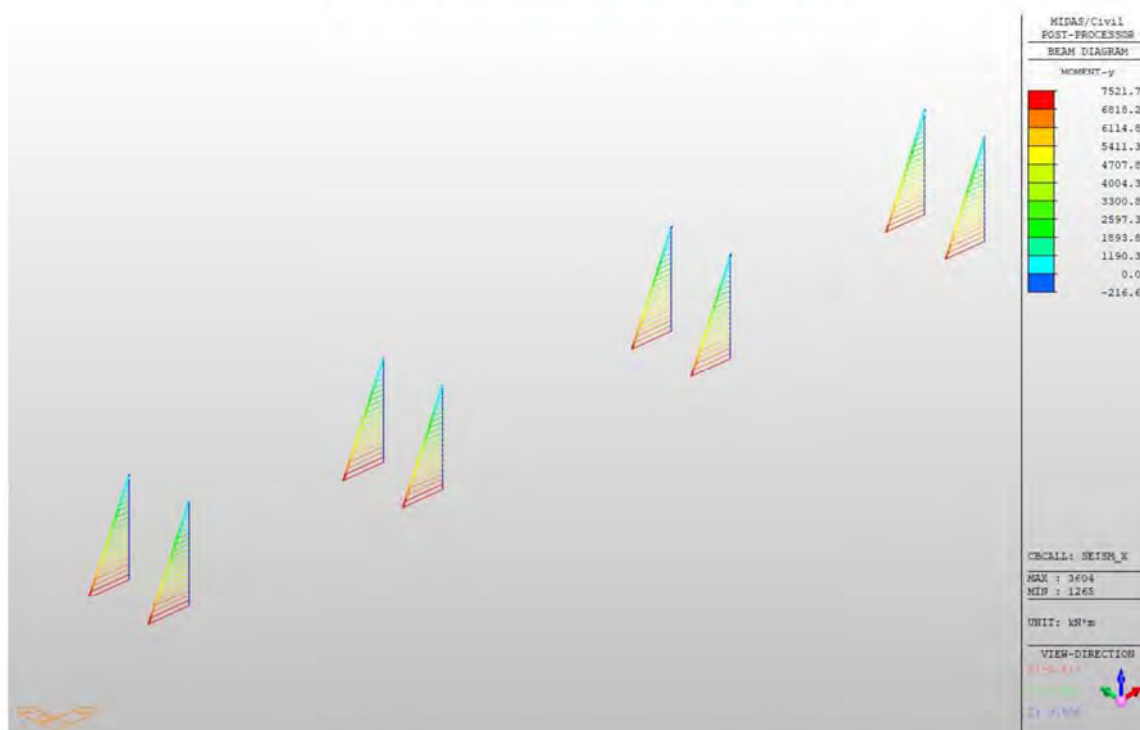


Fig. 13 Moment încovoietor maxim în stâlpii pilei

RADIER PILĂ

MIDAS/Civil

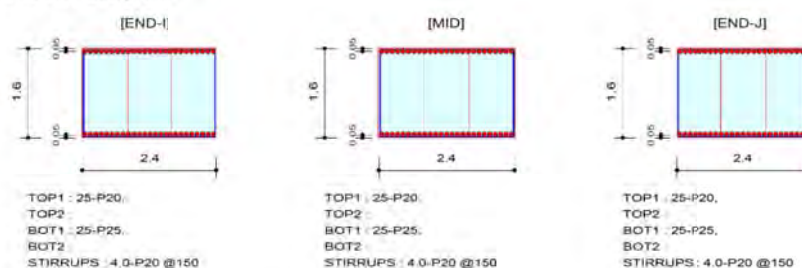
RC Beam Checking Result

MIDAS	Company		Project Title	
	Author	Atti	File Name	C:\... \Str4 - 5 grinzi in sect.mcb

1. Design Information

Design Code : Eurocode2-2:05
 Unit System : kN, m
 Material Data : fck = 30000, fyk = 500000, fyw = 500000 KPa
 Beam Span : 1.2 m
 Section Property : Radier (No. 7)

2. Section Diagram



3. Bending Moment Capacity

	END-I	MID	END-J
Negative Moment (M _{Ed})	478.45	59.73	630.23
(-) Load Combination No.	1-	2-	1-
Factored Strength (M _{Rd})	5139.42	5139.42	5139.42
Check Ratio (M _{Ed} /M _{Rd})	0.0931	0.0116	0.1285
Positive Moment (M _{Ed})	7719.35	5993.60	7719.42
(+) Load Combination No.	33+	33+	33+
Factored Strength (M _{Rd})	7900.10	7900.10	7900.10
Check Ratio (M _{Ed} /M _{Rd})	0.9771	0.7587	0.9771
Using Rebar Top (A _{s_top})	0.0079	0.0079	0.0079
Using Rebar Bot (A _{s_bot})	0.0123	0.0123	0.0123

4. Shear Capacity

	END-I	MID	END-J
Load Combination No.	33+	33+	33+
Factored Shear Force (V _{Ed})	4203.89	4283.09	4322.69
Shear Strength by Conc. (V _{Rdc})	1302.68	1302.68	1302.68
Shear Strength by Rebar (V _{Rds})	5081.20	5081.20	5081.20
Using Shear Reinf. (A _{sw})	0.0013	0.0013	0.0013
Using Stirrups Spacing	4 0-P20 @150	4 0-P20 @150	4 0-P20 @150
Check Ratio	0.8273	0.8429	0.8507

STÂLP PILĂ

MIDAS/Civil

RC Column Checking Result

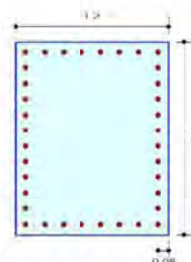
MIDAS	Company		Project Title	
	Author	Atti	File Name	C:\...\Str4 - 5 grinzi in sect.mcb

1. Design Condition

Design Code Eurocode2-2:05
 Unit System kN, m
 Member Number 3604 (PM), 1276 (Shear)
 Material Data $f_{ck} = 30000$, $f_{yk} = 500000$, $f_{yw} = 500000$ kPa
 Column Height 1.73 m
 Section Property Stalp pila (No : 5)
 Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	8-P32	10-P32	--

Total Rebar Area $A_{st} = 0.028953 \text{ m}^2$ ($R_{hst} = 0.0161$)



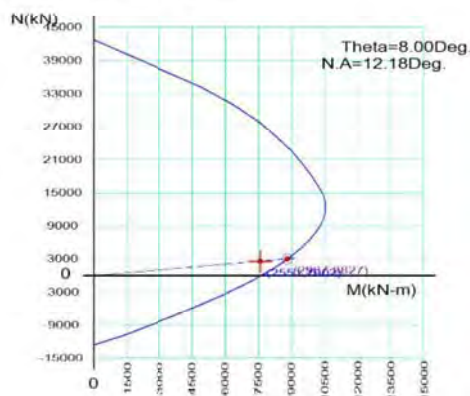
2. Applied Loads

Load Combination 33+ AT (J) Point
 $N_{Ed} = 2550.81 \text{ kN}$, $M_{Edy} = 7521.78$, $M_{Edz} = 1104.86$, $M_{Ed} = 7602.49 \text{ kN-m}$

3. Axial Forces and Moments Resistance Check

Concentric Max. Axial Load $N_{Rdmax} = 42696.1 \text{ kN}$
 Axial Load Ratio $N_{Ed}/N_{Rd} = 2550.81 / 2966.74 = 0.860 < 1.000$ O.K
 Moment Ratio $M_{Edy}/M_{Rdy} = 7521.78 / 8741.08 = 0.861 < 1.000$ O.K
 $M_{Edz}/M_{Rdz} = 1104.86 / 1227.80 = 0.900 < 1.000$ O.K
 $M_{Ed}/M_{Rd} = 7602.49 / 8826.89 = 0.861 < 1.000$ O.K

4. P-M Interaction Diagram



$N_{Rd}(kN)$	$M_{Rd}(kN-m)$
42696.06	0.00
30964.15	6408.21
25166.12	8372.43
20337.51	9517.19
16266.91	10233.23
13115.97	10570.05
10827.00	10537.07
8701.95	10257.40
6373.81	9808.26
3369.72	8962.50
-786.66	7279.36
-6788.92	3998.11
-12588.26	0.00

5. Shear Force Resistance Check

Applied Shear Resistance $V_{Edy} = 765.861$, $V_{Edz} = 274.577 \text{ kN}$ (Load Combination 34+)
 Shear Resistance (Y-Direction) $V_{Rdcy} = 563.616$, $V_{Rdsy} = 2837.18 \text{ kN}$ (4.0-P16 @100)
 Shear Resistance (Z-Direction) $V_{Rdcz} = 514.603$, $V_{Rdsz} = 3340.70 \text{ kN}$ (4.0-P16 @100)
 Shear Ratio $V_{Ed}/V_{Rd} = 274.577 / 514.603 = 0.534 < 1.000$ O.K

4.2.5 Eforturi in grinzi

Grinzile care intra in alcatuirea tablierului si ale caror caracteristici geometrice au fost descrise mai sus, au fost calculate tinandu-se seama de etapele de executie si de influenta fenomenelor reologice (contractie si curgere lenta).

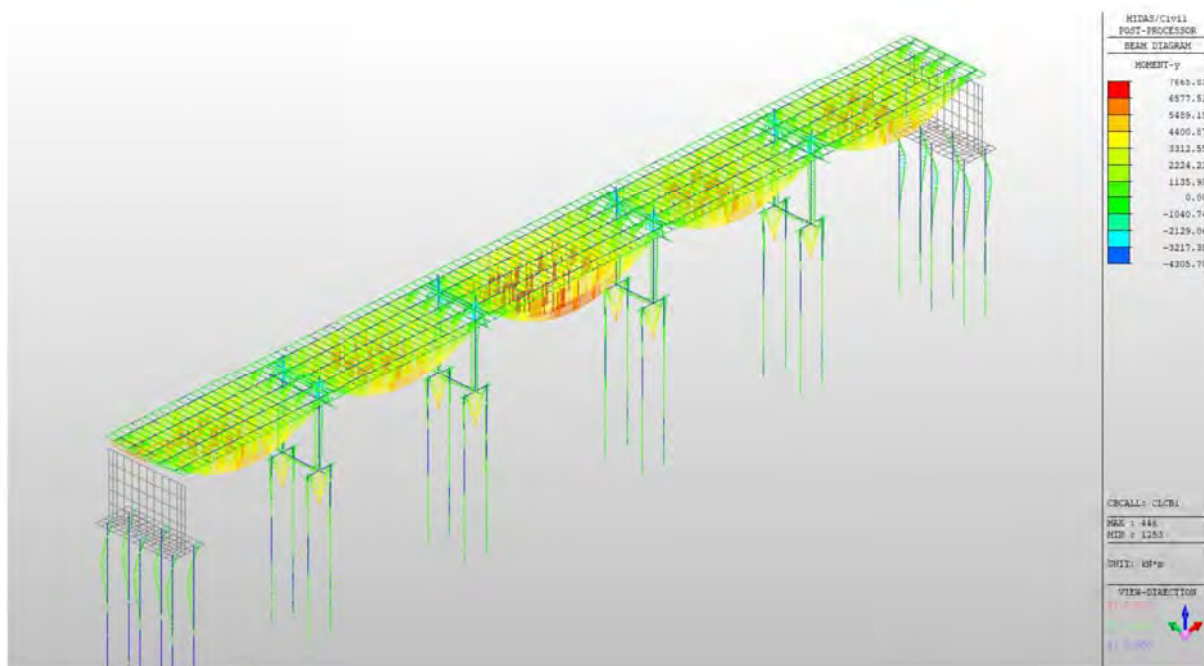


Fig. 14 Diagramă moment incovoiator grinzi

Dimensionarea grinzilor precomprimate s-a detaliat în breviare distincte pentru fiecare tip de grindă în parte.

5 ANEXA

PILOTI DE FUNDATIE

NORME DE REFERINTA

Pentru fazele de calcul se face referire la normele:

Eurocodul 7: Proiectare geotehnica – Partea 1: Reguli generale.

Eurocodul 8: Indicatii de proiect pentru rezistenta seismică a structurilor - Partea 5: Fundatii, structuri de sprijin, si specte geotehnice.

Sarcina Limita Verticala

Sarcina limită verticală a fost calculată cu formule statistice, care o exprimă în funcție de geometria coloanei consolidate, caracteristicile terenului si interfata pilot-teren. Astfel, întrucât realizarea unui pilot, bătut sau forat, modifică întotdeauna caracteristicile terenului în jurul acestuia, se propune considerarea unui unghi de rezistență la tăiere egal cu:

$$\begin{aligned}\phi' &= \frac{3}{4}\phi + 10^\circ \text{ pentru piloti bătuti} \\ \phi' &= \phi - 3^\circ \text{ pentru piloti forati}\end{aligned}$$

unde ϕ este unghiul de rezistență la tăiere înainte de executie. În continuare vom indica cu ϕ parametrul de rezistență ales.

La sfârșitul calculului, sarcina limită Q_{lim} este divizată în mod conventional în două cote, rezistenta la vârf Q_p si rezistenta laterală Q_l .

Resistenta Unitara La Varf

Formula lui Terzaghi

Solutia propusă de Terzaghi consideră că terenul existent deasupra adâncimii la care a ajuns vârful pilotului poate fi înlocuit de o suprasarcină echivalentă cu tensiunea verticală eficac (neglijând faptul că interacțiunea dintre pilot si fundatie ar putea modifica această valoare) si conduce analiza la problema capacității portante a unei fundatii superficiale.

Formula lui Terzaghi poate fi scrisă:

$$Q_p = c \times N_c \times s_c + \gamma \times L \times N_q + 0.5 \times \gamma \times D \times N_\gamma \times s_\gamma$$

unde:

$$N_q = \frac{a^2}{2 \cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2) \tan \phi}$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = \frac{\tan \phi}{2} \left(\frac{K_{p\gamma}}{\cos^2 \phi} - 1 \right)$$

Metoda lui Berezantzev

În principiu Berezantzev face referire la o suprafață de alunecare “alla Terzaghi” care se opreste pe planul de sprijin (vârful pilotului); totusi acesta consideră că cilindrul de teren coaxial pilotului are diametrul egal cu

extensia în secțiune a suprafeței de alunecare, este în parte “sustinut” prin acțiunea tangentială de către terenul rămas de-a lungul suprafeței laterale. Acesta dă o valoare a presiunii la baza inferioară a lui γD , și mai mică cu cât acest efect de “siloz” este marcant, adică cu cât mai mare este raportul D/B ; de acesta ține cont coeficientul N_q , care este deci funcție descrescătoare a lui D/B .

Rezistența unitară Q_p la vârf, pentru cazul terenului cu forfecarea (ϕ) și coeziunea (c), este dată de expresia:

$$Q_p = c \times N_c + \gamma \times L \times N_q$$

Indicând cu:

γ greutatea unității volumice a terenului;
 L lungimea coloanei;
 N_c e N_q sunt factorii capacității portante afectați de efectul formă (circulară);

Metoda lui Vesic

Vesic a asimilat problema rupturii în jurul vârfului pilotului și aceea a expansiunii unei cavități cilindrice în mediu elasto-plastic, în așa fel încât să se țină cont și de compresibilitatea mediului.

După Vesic coeficienții capacității portante N_q și N_c se pot calcula după cum urmează:

$$N_q = \frac{3}{3 - \sin \phi} \left\{ \exp \left[\left(\frac{\pi}{2} - \phi \right) \tan \phi \right] \tan^2 \left(45 + \frac{\phi}{2} \right) I_{rr}^{(4 \sin \phi) / [3(1 + \sin \phi)]} \right\}$$

Indicele de rigiditate redus I_{rr} , în expresia precedentă este calculat plecând de la deformarea volumică ε_v .

Indicele de rigiditate I_r se calculează utilizând modulul de elasticitate elastică tangentială G' și rezistența la tăiere a terenului s .

Când avem de-a face cu condiții nedrenate sau solul se găsește într-o stare de densă, termenul ε_v poate fi considerat egal cu zero și se obține $I_{rr} = I_r$.

Este posibilă estimarea lui I_r cu următoarele valori:

TEREN	I_r
Nisip	75-150
Praf	50-75
Argilă	150-250

Termenul N_c al capacității portante este calculat:

$$N_c = (N_q - 1) \cot \phi \quad (a)$$

Când $\phi = 0$ (condiții nedrenate)

$$N_c = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

Metoda lui Janbu

Janbu calculează N_q (cu unghiul ψ) după cum urmează:

$$N_q = \left(\tan \phi + \sqrt{1 + \tan^2 \phi} \right)^2 \exp(2\psi \tan \phi)$$

N_c se poate calcula de la (a) când $\phi > 0$.

Per $\phi = 0$ se folosește $N_c = 5.74$

Formula lui Hansen

Formula lui Hansen este valabilă pentru orice raport D/B , deci pentru fundatii de suprafață, dar si pentru cel profunde, acelasi autor a introdus coeficienti pentru o mai bună interpretare a comportamentului real al fundatiei, fără acestia, sarcina limită ar fi prea mult mărită odată cu adâncimea.

Pentru valori $L/D > 1$:

$$d_c = 1 + 0.4 \tan^{-1} \frac{L}{D}$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 \tan^{-1} \frac{L}{D}$$

În cazul $\phi = 0$

D/B	0	1	1.1	2	5	10	20	100
d'_c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

În factorii următori, exeresiile cu acest semn (') sunt egale cu $\phi = 0$.

Factor de formă:

$$s'_c = 0.2 \frac{D}{L}$$

$$s_c = 1 + \frac{N_q}{N_c} \frac{D}{L}$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{D}{L}$$

Factor de adâncime:

$$d'_c = 0.4k$$

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)k$$

$$d_\gamma = 1 \text{ per qualsiasi } k$$

$$k = \tan^{-1} \frac{L}{D} \text{ se } \frac{L}{D} > 1$$

Rezistenta Trunchiului

Metoda utilizată pentru calcularea capacității portante laterale este metoda α , propusă de Tomlinson (1971); rezistenta laterală este calculată după cum urmează:

$$Q_1 = (\alpha c + \sigma K \tan \delta) \cdot A_l \cdot f_w$$

A_l = suprafața laterală a pilotului;

f_w = factor de corectie legat de conicitatea trunchiului pilotului, sau reducerea diametrului pilotului;

c = valoarea medie a coeziunii (sau a rezistentei la tăiere în conditii nedrenate);

σ = presiunea verticală eficace a terenului;

K = coeficientul de alunecare orizontală, depinde de tehnologia de executie a pilotului si de starea precedentă de densitate, care se calculează după cum urmează:

Pentru piloti bătuti

$$K = 1 - \tan^2 \phi$$

Sau, în cazul specific, este posibilă desemnarea valorilor propuse în tabel:

Pilot	K	
	Teren afânat	Teren dens
Otel	0.5	1
Beton pref.	1	2
Lemn	1	3

Pentru piloti forati:

$$K = 1 - \sin \phi$$

δ = forfecarea pilot-teren în functie de asprimea suprafetei pilotului;

Pentru piloti bătuti

$$\delta = 3/4 \tan \phi$$

Pentru piloti forati

$$\delta = \tan \phi$$

α = coeficientul de adeziune găsit astfel:

Piloti forati:

$$\alpha = \frac{100 + c^2}{100 + 7c^2}$$

Caquot – Kerisel

Meyerhof – Murdock (1963)

$$\alpha = 1 - 0.1 \cdot c \text{ pentru } c < 5 \text{ t/m}^2$$

$$\alpha = 0.525 - 0.005 \cdot c \text{ pentru } c \geq 5 \text{ t/m}^2$$

Whitaker – Cooke (1966)

$$\alpha = 0.9 \text{ pentru } c < 2.5 \text{ t/m}^2$$

$$\alpha = 0.8 \text{ pentru } 2.5 \leq c < 5 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 5 \leq c \leq 7.5 \text{ t/m}^2$$

$$\alpha = 0.9 \text{ pentru } c > 7.5 \text{ t/m}^2$$

Woodward (1961)

$$\alpha = 0.9 \text{ pentru } c < 4 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 4 \leq c < 8 \text{ t/m}^2$$

$$\alpha = 0.5 \text{ pentru } 8 \leq c < 12 \text{ t/m}^2$$

$$\alpha = 0.4 \text{ pentru } 12 \leq c \leq 20 \text{ t/m}^2$$

$$\alpha = 0.20 \text{ pentru } c > 20 \text{ t/m}^2$$

Piloti bătuti

Coeficientul α pentru pilot bătut	
$2.5 \leq c < 5 \text{ t/m}^2$	$\alpha = 1.00$
$5 \leq c < 10$	$\alpha = 0.70$
$10 \leq c < 15$	$\alpha = 0.50$
$15 \leq c < 20$	$\alpha = 0.40$
$c \geq 20$	$\alpha = 0.30$

Forfecarea Negativa

Când un pilot este bătut sau trece printr-un strat de material compresibil înainte de a se fi terminat procesul de consolidare, terenul se va misca în raport cu pilotul făcând să apară eforturi de forfecare între pilot și teren care duc la așa-zisul fenomen de forfecare negativă. Efectul forfecării negative este acela de a crește sarcina axială asupra pilotului, având ca și consecință creșterea tasărilor. Forța care ia naștere ca rezultat al forfecării negative este estimată ca fiind egală cu componenta activă a rezistenței laterale (v. Rezistența Trunchiului) de-a lungul suprafeței laterale la contactul cu stratul în care se generează astfel de fenomen, dar din partea opusă forfecării negative. Rezultanta astfel determinată nu este dedusă din sarcina limită, ci din aceea de exercițiu.

Factorul De Corectie În Condiții Seismice

Criteriul lui Vesic

Vesic apreciază că pentru a ține cont de fenomenul de creștere în calcul a capacității portante este suficientă reducerea cu 2° a unghiului de forfecare a straturilor de fundație. Limita acestei sugestii este faptul că nu ține cont de intensitatea solicitării seismice (exprimată prin parametrul accelerației seismice orizontale maxime). Acest criteriu pare să găsească confirmarea în observațiile făcute în prezenta diverselor fenomene seismice.

Criteriul lui Sano

Autorul propune reducerea unghiului de forfecare a straturilor portante cu o cantitate dată în ecuația:

$$D_p = \arctg\left(\frac{A_{\max}}{\sqrt{2}}\right)$$

unde A_{\max} reprezintă accelerația seismică orizontală maximă.

Acest criteriu, față de cel al lui *Vesic*, reprezintă avantajul de a lua în considerare și intensitatea solicitării seismice. Experiența demonstrează totuși că aplicarea fără apelul la critică a acestei realități poate conduce la valori prea mici ale lui *Qlim*.

Corecțiile lui *Sano* și *Vesic* se aplică exclusiv pentru terenuri necoezive dense. Este greșită aplicarea lor la terenurile afânate sau cu densitate medie, unde vibrațiile seismice produc fenomenul opus celui de dilatare, cu creșterea gradului de densitate și a unghiului de forfecare.

TASARI

Tasarea verticală a fost calculată cu metoda lui Davis-Poulos, în funcție de care pilotul este considerat rigid (nedeformabil) înecat într-un mediu elastic, semispațiu sau strat de grosime finită.

Se ipotizează că interacțiunea pilot-teren este constantă la fâșii de-a lungul a n suprafețe cilindrice în care se împarte suprafața laterală a pilotului.

Tasarea suprafeței generice i ca efect la sarcina limită transmisă de la pilot la teren de-a lungul suprafeței cu nr. j poate fi exprimat:

$$W_{i,j} = (\tau_j / E) \times B \times I_{i,j}$$

Unde:

τ_j = Creșterea tensiunii în funcție de punctul mediu al fâșiei;

E = Modulul elastic al terenului;

B = Diametrul pilotului;

$I_{i,j}$ = Coeficientul de influență.

Tasarea compresivă se obține însumând $W_{i,j}$ cu toate ariile j .

SARCINA LIMITĂ ORIZONTALĂ

Sarcina limită orizontală a fost calculată după teoria dezvoltată de Broms, care presupune că comportamentul interfetei pilot-teren este rigidă perfect elastică, și astfel rezistența terenului să fluctueze cu orice valoare, nu nulă, a deplasării și să rămână constantă la creșterea deplasării.

Se presupune că comportamentul flexionar al pilotului este de tip rigid, perfect elastic, trebuie să spunem că rotațiile elastice ale pilotului sunt neglijabile până când momentul încovoietor ajunge la valoarea M_y de plasticizare.

Pentru terenuri coesive Broms propune adoptarea unei reacții a terenului constantă cu adâncimea egală cu:

$$p = 9 \times c_u \times B$$

cu reactia nulă până la adâncimea de 1.5 d; indicând cu:

c_u = Coeziunea nedrenată,

B = Diametrul pilotului;

p = Reactia terenului pentru unitatela de lungime a pilotului.

Pentru terenurile necoezive se presupune că rezistenta variază liniar în functie de adâncime după formula:

$$p = 3K_p \gamma zB$$

indicând cu:

p = Reactia terenului pentru unitatela de lungime a pilotului;

K_p = Coeficientul de împingere pasivă;

γ = Greutatea volumică a terenului;

z = Adâncimea;

B = Diametrul pilotului

PILOT ÎN CONDITII DE EXERCITIU

Analizza pilotului în conditii de exercitiu: **Metoda elementelor finite.**

Metoda elementelor finite modelează pilotul de fundatie supus la sarcini transversale, în mod realist întrucât face uz, fie de împingeri cât si de rotatiile în noduri pentru definirea linieri elastice a pilotului, deci reprezintă metoda cea mai ratională si eficientă disponibilă actualmente pentru analizarea acestor tipuri de structuri.

În ceea ce urmează se face apel la teoria metodi, indicând cu **P** matricea fortelor nodale externe, cu **F** aceea a fortelor interne si cu **A** matricea coeficientilor de influentă care, pentru echilibrul între fortele externe si interne, leagă primele două în functie de formula:

$$\mathbf{P} = \mathbf{A}\mathbf{F}$$

Deplasările interne **e** (împingerile si rotatiile) ale elementului noduli generic sunt legate de deplasările externe **X** (împingerile si rotatiile) aplicate la noduri, ale următoarei formule:

$$\mathbf{e} = \mathbf{B}\mathbf{X}$$

unde matricea **B** s-a demonsatrat a fi transpunerea matricei **A**.

De altă parte, fortele interne **F** sunt legate de deplasările interne **e** ale următoarei expresii:

$$\mathbf{F} = \mathbf{S}\mathbf{e}$$

Aplicând substituirile de rigoare se obtine:

$$\mathbf{F} = \mathbf{S}\mathbf{A}^T\mathbf{X}$$

Si deci:

$$\mathbf{P} = \mathbf{A}\mathbf{F} = \mathbf{A}\mathbf{S}\mathbf{A}^T\mathbf{X}$$

Deci, calculând inversa matricei **A S A^T** se obtine expresia deplasărilor externe **X**:

$$\mathbf{X} = (\mathbf{A}\mathbf{S}\mathbf{A}^T)^{-1}\mathbf{P}$$

Notati, deci, că cu ajutorul deplasărilor **X** este posibilă aflarea fortelor interne **F** necesare pentru proiectul structurii.

Matricea **A S A^T** este cunoscută ca si matrice de rigiditate globală deoarece caracterizează legătura dintre deplasări si fortele externe nodale.

Metoda elementelor finite are, printre altele, avantajul că dă posibilitatea de a lua în considerare, ca și condiții la contur, rotații și deplasări cunoscute.

Rotările nodale care schematizează terenul sunt considerate foarte globale legate de modulul de reacție și de zona de influență a nodului. În cadrul soluției elementelor finite pentru piloti supuși la sarcini transversale, modulul de reacție este considerat în forma:

$$k_s = A_s + B_s Z^n$$

sau, nevrînd să creștem nelimitat k_s cu adâncimea, în forma:

$$k_s = A_s + B_s \tan^{-1}(Z/B)$$

în care Z reprezintă adâncimea iar B reprezintă diametrul pilotului.

Valorile lui A_s și $B_s Z^n$ sunt obținute din expresia capacității portante (Bowles) cu factori corectori s_i , d_i , și i_i egali cu 1:

$$k_s = q_{ult}/\Delta H = C(cN_c + 0.5\gamma B N_\gamma)$$

$$B_s Z^n = C(\gamma N_q Z^1)$$

Unde $C = 40$ este obținut în corespondență cu o tasare maximă de 25 mm.

Date generale...

Descriere	pilot dn 1200mm
Diametru varf	1.20 m
Lungime	20.00 m
Tip	Forat
Capacitatea de varf calculata cu:	Terzaghi
Tip beton	1
Tip otel	1

Arhiva materiale

Beton

Nr.	Clasa Beton	f _{ck} ,cubic [Kg/cm ²]	E _c [Kg/cm ²]	f _{ck} [Kg/cm ²]	f _{cd} [Kg/cm ²]	f _{ctd} [Kg/cm ²]	f _{ctm} [Kg/cm ²]
1	C20/25	250	299600	200	113.3	10.1	22.1
2	C25/30	300	314750	250	141.6	11.4	25.6
3	C28/35	350	323080	280	158.6	12.6	27.6
4	C40/50	500	352200	400	226.6	16.3	35

Oteluri:

Nr.	Steel Class	E _s [Kg/cm ²]	f _{yk} [Kg/cm ²]	f _{yd} [Kg/cm ²]	f _{tk} [Kg/cm ²]	f _{td} [Kg/cm ²]	ep _{tk}	ep _{d_ult}	β1*β2 in.	β1*β2 fin.
1	B450C	2000000	4500	3913	4500	3913	.075	.0675	1	0.5
2	B450C*	2000000	4500	3913	5400	4500	.075	.0675	1	0.5
3	B450C**	2000000	4500	3913	4582	3985	.012	.01	1	0.5
4	S235H	2141370	2447.28	2128.11	3670.92	2128.11	0.012	0.01	1	0.5
5	S275H	2141370	2855.16	2482.97	4384.71	2482.97	0.012	0.01	1	0.5
6	S355H	2141370	3670.92	3191.66	5200.47	3670.92	0.012	0.01	1	0.5

Stratigrafie

Nr.: Numarul stratului Hs: Grosimea stratului Fi: Unghi de frecare c: Coeziune Alfa: Coef. aderenta frecari laterale. Vs: Viteza unde de forfecare.

Stratigrafie...1

Varianța de ocolire Timisoara Sud - Pasaj km 18+117 peste DJ592 si linii de tramvai

Nr.	Hs	Greutate volumica [kg/m³]	Greutate volumica saturata [kg/m³]	c [kg/cm²]	Fi (°)	Frecare negativa	Alfa	Modul elasticitate [kg/cm²]	Vs [m/s]	Descriere litologica
1	3.00	2000.00	0.00	0.04	20.00	Nu	0.00	40.00	0	
2	1.00	2100.00	0.00	0.06	25.00	Nu	0.00	60.00	0	
3	4.00	1850.00	0.00	0.00	32.00	Nu	0.00	300.00	0	
4	3.30	2100.00	0.00	0.06	25.00	Nu	0.00	60.00	0	
5	2.10	1900.00	0.00	0.02	15.00	Nu	0.00	20.00	0	
6	0.90	2100.00	0.00	0.06	25.00	Nu	0.00	60.00	0	
7	2.50	1950.00	0.00	2.00	40.00	Nu	0.00	300.00	0	
8	6.20	2100.00	0.00	0.06	25.00	Nu	0.00	60.00	0	
9	2.00	1900.00	0.00	0.00	36.00	Nu	0.00	500.00	0	nisip prăfos

Sacina limita

Stratigrafie	Nq	Nc	Fi/C strat varf pilot (°)/[kg/cm²]	Greutate pilot [kg]	Sarcina limita varf [kg]	Sarcina limita laterala [kg]	Sarcina limita [kg]	Frecare negativa [kg]	Sarcina limita orizontala [kg]
A1+M1+R3	13.21	25.78	25.34338/0.05	56548.67	0.00	766170.30	709621.60	--	--

REZISTENTA DE PROIECTARE SARCINI AXIALE

Rezistenta caracteristica sarcini axiale. Nume combinatie:				A1+M1+R3
Numar de verticale inspectate				1
Factor de corelatie pentru verticale inspectate mediu (xi3)				1.70
Factor de corelatie pentru vertical inspectate minim (xi4)				1.70
	Rc, Min [kg]	Rc, Media [kg]	Rc, Max [kg]	
Baza	--	--	--	
Lateral	766170.30	766170.30	766170.30	
Total	709621.60	709621.60	709621.60	
Coeficient partial rezistenta caracteristica				R1
Lateral				1.00
Rezistenta de proiectare laterala				450688.40 kg
Rezistenta de proiectare				394139.70 kg

Tasare (Fleming 1992)

Lungime	20.00 m
Diametru capat	1.20 m
Diametru varf	1.20 m
Tronson frecare laterala nula	0.00 m
Modul elasticitate sectiune	2.94E+07 kN/mq
Punct de aplicare a rezultantei rezistentei active	0.45
Factor de flexibilitate teren/pilot	1.00E+03
Sarcina aplicata	3460.00 kN
Sarcina limita laterala	5241.00 kN
Sarcina limita de varf	5286.00 kN
Modul elasticitate teren corespondent	1.47E+03 kN/mq
Scurtarea elastica	0.94 mm
Tasare rigida	2.32 mm
Tasare totala	3.25 mm



intocmit ing. Attila Takacs



PASAJ KM 18+926 PESTE CANAL DE IRIGATII



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1 DESCRIERE SI BAZA DE PROIECTARE

1.1 Descrierea structurii

Varianta Ocolitoare Timisoara Sud traverseaza un canal de irigatii la km 18+926 printr-un pod, traseul in plan fiind pe o curba cu raza de $R=1500\text{m}$.

Suprastructura pasajului este alcatuita din 5 grinzi prefabricate, precomprimate cu lungimea de 27 m si inaltimea de 1.30 m si antretoaze la capete.

Rezemarea tablierului la fiecare capat pe culei se va face in cate doua puncte sub antretoazele de capat, unde se vor dispune aparate de reazem din neopren asezate pe cuzineti din beton armat.

Placa de suprabetonare se va executa din beton armat, turnat direct peste grinzile joantive din componenta tablierului. Marginal, longrinele monolite in consola care sustin parapetele pietonale se vor executa cu cofraj fixat de grinzile marginale. Peste placa de suprabetonare se va aterne hidroizolatie in grosime de 1 cm, protectia hidroizolatiei in grosime de 3 cm din BA8 si straturile rutiere in grosime de 2x4 cm din BAP 16 respectiv MAS 16 conform Proiect Ilustrativ.

Pasajul fiind amplasat in aliniament, profilul acestuia pe zona carosabilului in sens transversal este in profil de acoperis cu pante transversale de 2.5% dinspre ax spre trotuare, iar pe zona trotuarelor la nivel panta transversala a acestora este de 1% inspre axul structurii.

In sectiune transversala suprastructura prezinta urmatoarele caracteristici: 7.80 m parte carosabila, incadrata cu parapete New Jersey 0.60m, doua trotuare la nivel de 1.00 m latime utila fiecare, si longrine cu latimea de 0.30 m, latimea totala a suprastructurii fiind de 11.60 m.

Conform proiectului ilustrativ, la marginea partii carosabile au fost prevazute parapete de siguranta de tip rigid (parapete New Jersey din beton armat cu grad de asigurare H4b) pentru protectia pietonilor iar pe partile exterioare ale trotuarelor se vor prevedea parapete pietonale metalice.

Pe ambele parti ale structurii, marginal acesteia in deschidere se vor amplasa panouri de protectie din plasa de sarma.

Infrastructurile sunt prevazute a fi executate din beton armat, fundate indirect, pe piloti forati de diametru mare $d=1200\text{mm}$, avand capetele solidarizate la partea superioara cu radiere din beton armat.

Racordarea cu terasamentele se va realiza cu sferturi de con pereate cu beton. Pe partea carosabila pentru tranzitia de la tablierul podului la rambteau drumului se vor executa placi de racordare din beton armat cu lungimea de 6.00 m.

Urmatoarele faze de executie sunt considerate:

- Executarea piloților și a radierelor;
- Executarea elevatiilor pentru culei;
- Poziționarea grinzilor;
- Poziționarea predalelor, armarea placii de suprabetonare și a antretoazelor;
- Betonarea placii de suprabetonare și a antretoazelor;
- Executarea terasamentelor;
- Executarea caii, parapetilor, lisei și a bordurilor.

Clasa de incarcare: Eurocode: LM1

Clasa de importanta: "C"

Cerinte de calitate: A4, B2, D2.

Proiectare seismica: $a_g = 0.20g$, $T=0.7s$ conform P100/2013

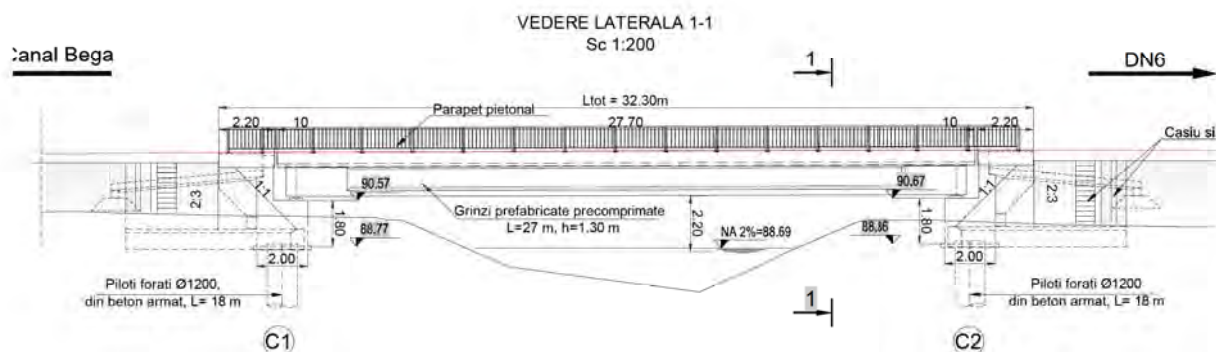


Fig. 1 Elevatie

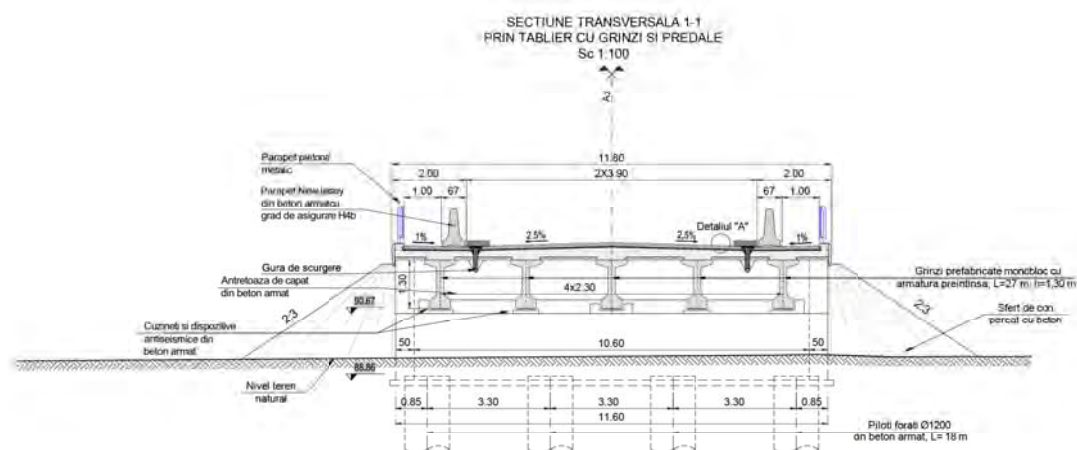


Fig. 2 Sectiune transversala

1.2 Baza de proiectare si materiale

Instructioni si standarde considerate in proiectarea structurii:

- Eurocode 1: Actiuni asupra structurilor
- Eurocode 2: Proiectarea structurilor de beton
- Eurocode 7: Proiectarea geotehnica
- Eurocode 8: Proiectarea pentru rezistenta la cutremur a structurilor
- P100/2013: Cod de proiectare seismica

Caracteristicile minime ale materialelor:

- Otel pentru armatura: BST 500 C
- Otel pentru precomprimare: Y 1860
- Beton in coloane: C25/30
- Beton in radier culei: C25/30
- Beton in elevatii culei: C30/37
- Beton pentru turnarea placii in-situ, pentru grinzi de parapet si lisele monolite: C35/45;
- Beton pentru grinzi prefabricate: C50/60

2 ACTIUNI

2.1 Actiuni considerate

In concordanta cu Eurocode 1, actiunile considerate in proiectarea structurii sunt:

- Actiuni permanente (G):

- Greutatea proprie a grinzilor, placilor, elevatiei, radielor si a pilotilor;
- Greutatea caii, parapetelor, trotuar;
- Impingerea pamantului;
- Impingerea pamantului din seism;

- Actiuni variabile (Q):

- Actiuni din trafic la poduri (SR EN 1991-2:2005) (convoiul LM1, încărcări din oameni, frânarea și forța centrifugă);

- Actiuni termice, temperatura (T-) si T(+);

- Actiuni accidentale (A):

- Actiuni seismice, Seism X (longitudinal), Seism Y (transversal) , Seism Z (vertical).

2.2 Valori caracteristice

2.2.1 Actiuni permanente (G).

2.2.1.1 Greutatea proprie a elementelor structurale

Element	buc	Arie m ²	Densitate kN/m ³	Gn kN/m	γ	Gc kN/m
Grinzi H=1.30m	5	0.467	25	58.38	1.35	78.81
Placa grinzi marginale	2	0.578	25	28.90	1.35	39.02
Placa grinzi centrale	3	0.732	25	54.90	1.35	74.12
Cale	1	1.3464	24	32.31	1.35	43.62
Lisa prefabricata	2	0.043	25	2.15	1.35	2.90
Parapet directiona	2	-	-	11.00	1.35	14.85
Parapet pietonal	2	-	-	1.50	1.35	2.03
Grinda parapet	2	0.042	25	6.50	1.35	8.78
TOTAL / ml				195.64		264.11

2.2.1.2 Impingerea pamantului

$h=$	1.80	m	Inaltime elevatie
$H_{zid}=$	1.75	m	inaltime zid de garda
$H_T=$	3.55	m	Inaltime totala
$\phi_{df}=\phi_{kf}=$	33	grade	
$g_{kd}=g_{kf}=$	19.00	kN/m ³	
$K_{ad}=tg(\pi/4-\phi_{df}/2)^2=$	0.333	coef. presiune activa	

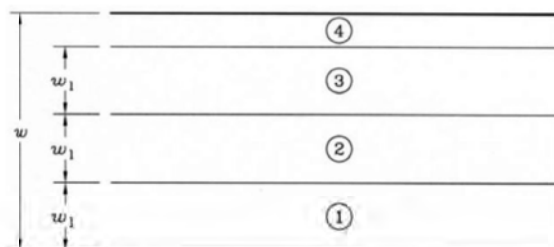
2.2.2 Actiuni variabile (Q)

2.2.2.1 Acțiuni din trafic la poduri (SR EN 1991-2:2005) (LM1);

Pentru determinarea efectelor zonelor de incarcarea a partii carosabile a unui pod aceasta este divizata in benzi teoretice.

Lățimea părții carosabile w	Numărul benzilor teoretice de circulație	Lățimea unei benzi teoretice de circulație	Lățimea zonei rămase
$w < 5,4 \text{ m}$	$n_l = 1$	3 m	$w - 3 \text{ m}$
$5,4 \text{ m} \leq w < 6 \text{ m}$	$n_l = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_l = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTĂ – De exemplu, pentru o parte carosabilă cu lățimea de 11 m, $n_l = \text{Int}\left(\frac{w}{3}\right) = 3$, și lățimea zonei rămase este: $11 - 3 \times 3 = 2 \text{ m}$.			

Numărul, poziția și numerotarea benzilor teoretice încărcate, se realizează astfel incat efectele convoaielor de calcul să fie cele mai defavorabile in scopul unei verificări individuale.



w =Lățimea părții carosabile

w_1 =Lățimea unei benzi teoretice de circulație

1=Banda teoretică nr.1

2=Banda teoretică nr.2

3=Banda teoretică nr.3

4=Zona rămasă

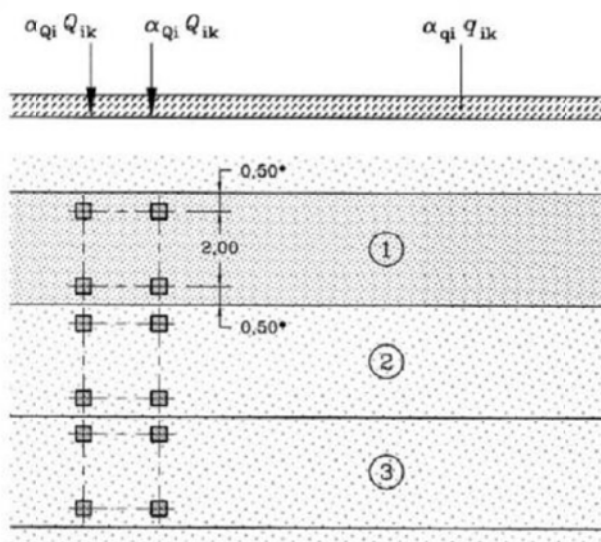
2.2.2.1.1 Convoiul de calcul 1 (LM1) :

Acest convoi este alcătuit din încărcări concentrate și încărcări uniform distribuite ce acoperă cele mai multe efecte din traficul alcătuit din camioane și automobile, fiind utilizat atât pentru verificări locale cât și globale.

Încărcările concentrate sunt reprezentate de sistemul tandem TS alcătuit din 2 osii cu greutatea " αQ_k " pe fiecare osie.

Notă: atât pentru verificări locale cât și generale se va aplica un singur sistem tandem pe fiecare bandă de circulație cu precizarea că poziționarea acestuia se va face centrat pe axul fiecărei benzi, în cazul verificărilor globale, și în poziția cea mai defavorabilă în cazul celor locale, fără însă a depăși limita de 0.5 m între două sisteme de pe două benzi adiacente.

Încărcările uniform distribuite sunt reprezentate de sistemul UDL cu greutatea " αq_k " pe metru pătrat;



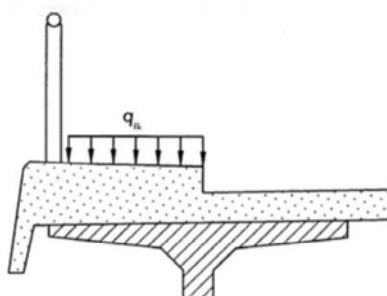
Coeficienții de corecție pentru încărcările uniform distribuite și concentrate " α_q " și " α_Q " se iau egali cu 1 (conform secțiunea 4-4.3.2(3)-SR EN 1991-2:2004/NB:2006).

Poziția	Sistemul tandem TS	Sistemul UDL
	Încărcarea pe osie Q_{ik} (kN)	q_{ik} (sau q_{rk}) (kN/m ²)
Banda numărul 1	300	9,0
Banda numărul 2	200	2,5
Banda numărul 3	100	2,5
Alte benzi	0	2,5
Zona rămasă (q_{rk})	0	2,5

Nota: Valorile încărcărilor prezentate in tabel includ amplificarea dinamică.

Încărcarea uniform distribuită

Pentru podurile de șosea care susțin trotuare pentru pietoni sau piste pentru cicliști, se definește o forță uniform distribuită q_{fk} .



Valoarea $q_{fk}=3\text{kN/mp}$ pentru gruparea gr1a.

2.2.2.1.2 Forțe orizontale

Forța de franare (SR EN 1991-2:2005)

Forța de franare " Q_{fk} " este o forță ce acționează în sens longitudinal podului la nivelul suprafeței de rulare a părții carosabile.

Valoarea caracteristică Q_{fk} este limitată la 900 kN pentru întreaga lățime a podului, și trebuie calculată ca fracțiune din totalul sarcinilor verticale maxime corespunzătoare convoiului LM1, aplicabile benzii nr.1 după cum urmează:

$$Q_{fk}=0.60\alpha_{Q1} (2Q_{1k})+0.10 \alpha_{q1}q_{1k}w_1L \leq 900(\text{kN})$$

L =lungimea tablierului sau a părții considerate în calcul.

Pentru cazurile curente când lungimea de încărcare $L > 1,2m$ forța de franare este egală cu $Q_{lk} = 360 + 2,7L \leq 900$ (kN).

Limita inferioară a forței de franare la podurile de șosea este de 300kN.

$L_{cale} =$	27.7	m	$w_l =$	3	m
$Q_{1k} =$	300	kN	$\alpha_{Q1} =$	1	
$q_{1k} =$	9	kN/m ²	$\alpha_{q1} =$	1	
$Q_{lk} =$	434.79	kN	< 900	kN	
$Q_{lk} =$	434.79	kN			
Nr Grinzi	5				
$Q_{lk} / grinda =$	86.958	kN - forta orizontala			

2.2.2.2 Actiuni termice (SR EN 1991-1-5:2005), temperatura (T_-) și T_+

Valoarea caracteristică a componentei de temperatură uniformă este asociată fie contracției, fie dilatării fiind notată cu $\Delta T_{N,contractie}$ sau cu $\Delta T_{N,dilatate}$ pentru fiecare caz în parte.

$$\Delta T_{N,contractie} = T_0 - T_{e,min} \text{ - temperatura, } T(-)$$

$$\Delta T_{N,dilatate} = T_{e,max} - T_0 \text{ - temperatura, } T(+)$$

T_0 - reprezintă temperatura inițială, se utilizează valoarea recomandată 10°C (SR EN 1991-1-5:2004/NA:2008).

$T_{e,min}$ și $T_{e,max}$ reprezintă componentele de temperatură uniformă minimă și maximă.

$T_{e,min} = -32^\circ C$ și $T_{e,max} = 42^\circ C$ – conf. SR EN 1991-1-5:2004/NA:2008, tabel NA.1(RO).

2.2.3 Actiuni seismice (S)

Amplasamentul structurii este caracterizat de accelerația maximă a terenului $a_g = 0.20g$ și perioada de colt $T_c = 0.7s$.

Pentru calculul forței seismice se respectă prevederile:

- SR EN 1998-1 – Proiectarea structurilor pentru rezistența la cutremur. Reguli generale, acțiuni seismice și reguli pentru clădiri;
- SR EN 1998-2 – Proiectarea structurilor pentru rezistența la cutremur. Poduri;
- SR EN 1998-2/NA – Proiectarea structurilor pentru rezistența la cutremur. Poduri. Anexa națională.
- P100/2013 – Cod de proiectare seismică.

Conform SE EN 1998-2/NA – clasa de importanță este "III" și $\gamma_I=1,3$ - coeficient de amplificare a forței seismice.

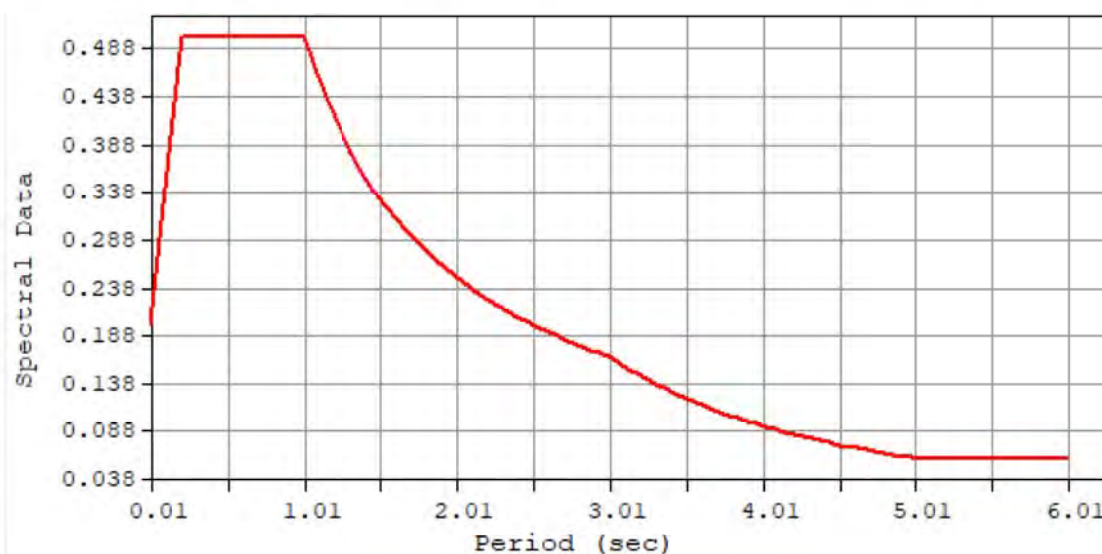


Fig. 3 Spectru de raspuns seismic orizontal

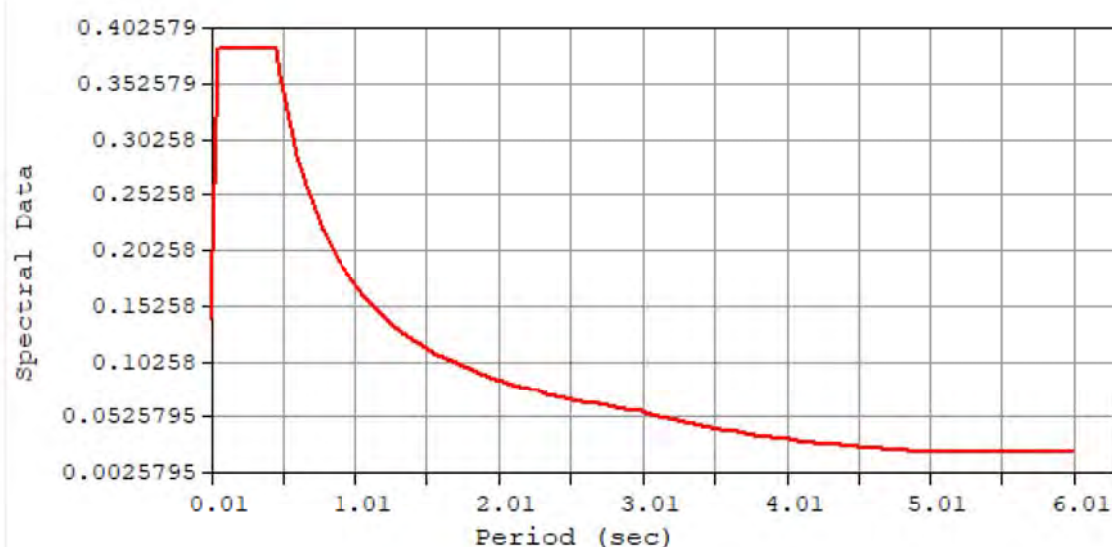


Fig. 4 Spectru de raspuns seismic vertical

3 COMBINAREA ACȚIUNILOR

Conform SR EN 1990 , Anexa A2 combinațiile sunt următoarele:

- Combinarea fundamentală din acțiuni persistente (permanente) și tranzitorii;
- Combinarea acțiunilor pentru situații accidentale;
- Combinarea acțiunilor pentru calcul seismic;

Coeficientii care intervin la combinarea acțiunilor pentru starea limită ultimă sunt:

$g_1=1,35$ – pentru acțiuni permanente (greutate proprie elemente structurale, împingere pământ);

$g_2=1,35$ – pentru acțiunea din convoi LM1 și încărcarea uniform distribuită pe trotuar;

$\Psi_{i1} -0.75$ pentru gruparea 1a – sistem TS tandem;

$\Psi_{i2} -0.40$ pentru gruparea 1a – sistem UDL forțe uniform distribuite

$\Psi_{i3} -0.40$ pentru gruparea 1a – încărcarea uniform distribuită pe trotuare

 DESIGN TYPE : Concrete Design

LIST OF LOAD COMBINATIONS

NUM	NAME	ACTIVE	TYPE	LOADCASE(FACTOR) +	LOADCASE(FACTOR) +	LOADCASE(FACTOR)
=====						
1	cLCB1	Strength/Stress	Add	Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
				+ Wearing Surface(1.350) +	Utility1(1.350) +	Imp_Pamant(1.350)
				+ ML-caract(1.350)		

2	cLCB2	Strength/Stress	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
				+ ML-caract(1.350)		

3	cLCB3	Strength/Stress	Add	Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
				+ Wearing Surface(1.350) +	Utility1(1.350) +	Imp_Pamant(1.350)
				+ Temp (+)(1.500)		

4	cLCB4	Strength/Stress	Add	Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
				+ Wearing Surface(1.350) +	Utility1(1.350) +	Imp_Pamant(1.350)
				+ Temp (+)(-1.500)		

5	cLCB5	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
		+ Wearing Surface(1.350) +	Utility1(1.350) +	Imp_Pamant(1.350)
		+ Temp (-)(1.500)		
6	cLCB6	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
		+ Wearing Surface(1.350) +	Utility1(1.350) +	Imp_Pamant(1.350)
		+ Temp (-)(-1.500)		
7	cLCB7	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ Temp (+)(1.500)		
8	cLCB8	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ Temp (+)(-1.500)		
9	cLCB9	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ Temp (-)(1.500)		
10	cLCB10	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ Temp (-)(-1.500)		
11	cLCB11	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ ML-caract(1.000)		
12	cLCB12	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ ML-caract(1.000) +	Temp (+)(0.600)	
13	cLCB13	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
		+ Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
		+ ML-caract(1.000) +	Temp (+)(-0.600)	
14	cLCB14	Serviceability	Add	

			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			ML-caract(1.000) +	Temp (-)(0.600)	
<hr/>					
15	cLCB15	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			ML-caract(1.000) +	Temp (-)(-0.600)	
<hr/>					
16	cLCB16	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			Temp (+)(1.000)		
<hr/>					
17	cLCB17	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			Temp (+)(-1.000)		
<hr/>					
18	cLCB18	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			Temp (-)(1.000)		
<hr/>					
19	cLCB19	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			Temp (-)(-1.000)		
<hr/>					
20	cLCB20	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			ML-frecv(1.000) +	Temp (+)(0.500)	
<hr/>					
21	cLCB21	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			ML-frecv(1.000) +	Temp (+)(-0.500)	
<hr/>					
22	cLCB22	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+			ML-frecv(1.000) +	Temp (-)(0.500)	
<hr/>					
23	cLCB23	Serviceability	Add		
			Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+			Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)

+	ML-frecv(1.000) +	Temp (-)(-0.500)	
24	cLCB24 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (+)(0.600)		
25	cLCB25 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (+)(-0.600)		
26	cLCB26 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (-)(0.600)		
27	cLCB27 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (-)(-0.600)		
28	cLCB28 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (+)(0.500)		
29	cLCB29 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (+)(-0.500)		
30	cLCB30 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (-)(0.500)		
31	cLCB31 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
+	Temp (-)(-0.500)		
32	cLCB32 Serviceability	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Imp_Pamant(1.000)
33	Seism_X Strength/Stress	Add	

	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Prestress(0.500)
+	Temp (+)(1.000) +	Imp_Pamant(1.000) +	Rs_X(1.000)
+	Rs_Y(0.300) +	ML-caract(0.300)	

34	Seism_Y	Strength/Stress	Add
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)
+		Wearing Surface(1.000) +	Utility1(1.000) + Prestress(1.000)
+		Temp (+)(0.500) +	Imp_Pamant(1.000) + Rs_X(0.300)
+		Rs_Y(1.000) +	ML-caract(0.300)

4 ANALIZA STRUCTURALA

4.1 Modelul structural

Analiza globală a fost realizată cu ajutorul programului de analiză element finit MIDAS Civil 2020. Suprastructura a fost modelată cu elemente tip bară legate cu link-uri elastice. Culeele se consideră încastrate la nivelul radierului. Analiza seismică - calculul modal cu spectre de răspuns.

4.1.1 Caracteristicile materialelor utilizate:

Tabel 1 Caracteristici materiale

ID	Name	Type	Standard	Code	DB	Density (kN/m ³)
1	C50/60	Concrete	EN04 (RC)		C50/60	2.5000e+001
2	C35/45	Concrete	EN04 (RC)		C35/45	2.5000e+001
3	Y1860S7 (15.2mm)	Steel	EN05-PS (S)		Y1860S7 (15.2mm)	7.6980e+001
5	C30/37	Concrete	EN04 (RC)		C30/37	2.5000e+001
6	Dummy Material	User Defined	None		C35/45	0.0000e+000

Geometria structurii

Modelarea suprastructurii, elevatiei a fost realizata cu elemente de tip grinda (frame) pentru modelarea tablierului. (Fig. 5).

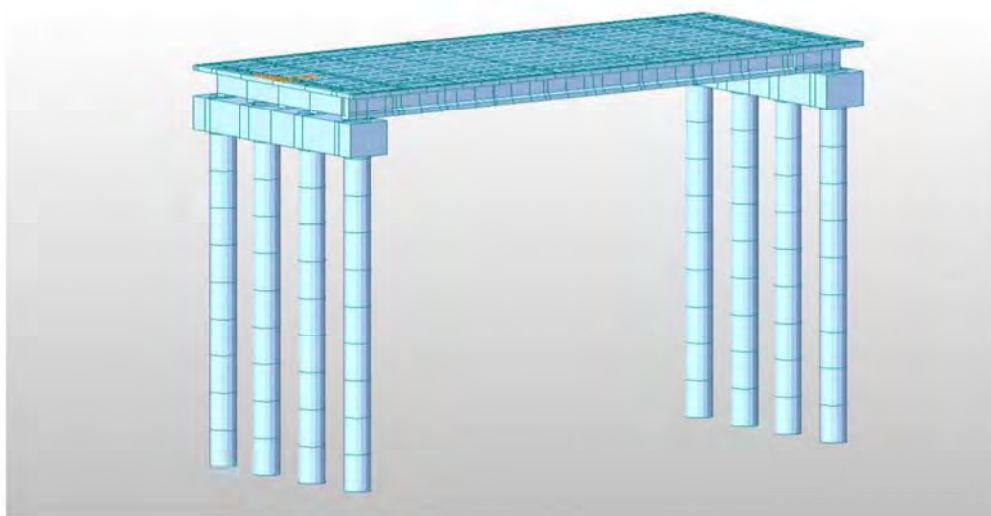


Fig. 5 Modelare structurala 3D

4.2 Centralizarea rezultatelor și verificarea secțiunilor

4.2.1 Verificarea capacității portante ultime a piloților forati

Piloți Culee

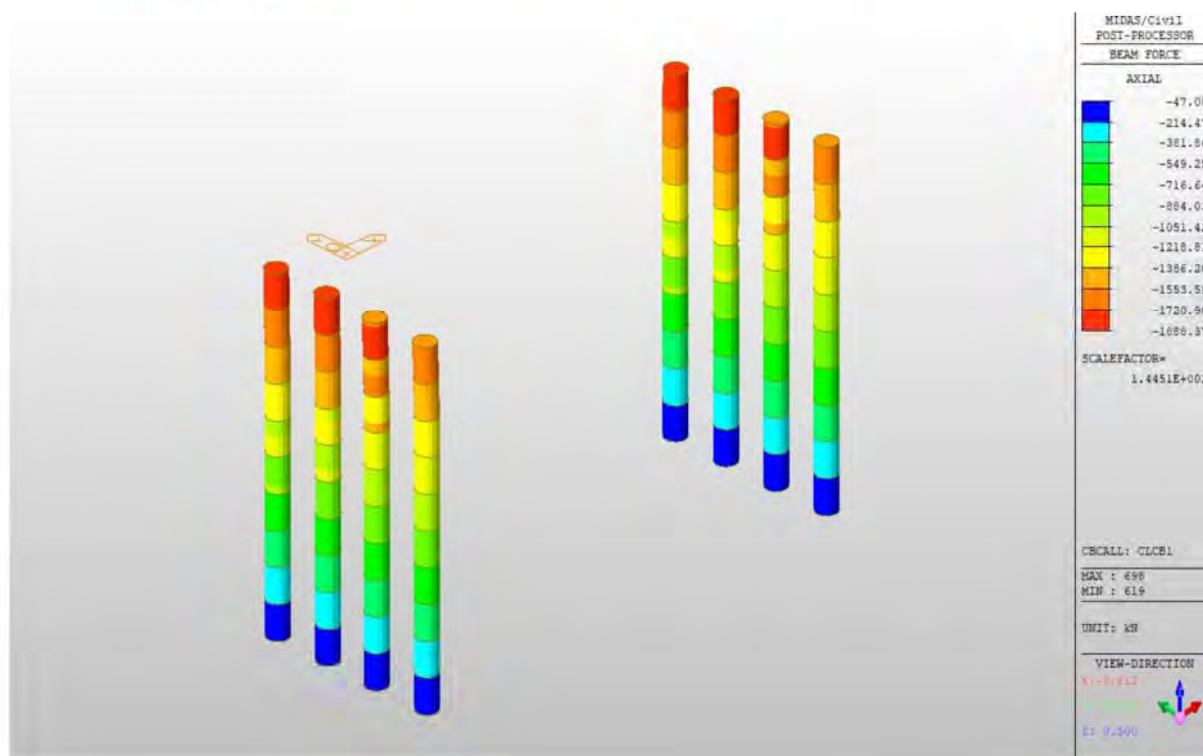


Fig. 6 Solicități axiale maxime la piloții din culee

Fcd= 1888 kN – valoarea de calcul a încărcării axiale de compresiune asupra unui pilot corespunzător stării limite ultime – Gruparea CLCB1

Rcd= 5550 kN – capacitatea portantă ultimă la compresiune a pilotului, determinată cu programul de calcul GeoStru, anexat breviarului.

Fcd < Rcd – Adevărat

4.2.2 Dimensionare armătură piloți la moment încovoiator și forță tăietoare

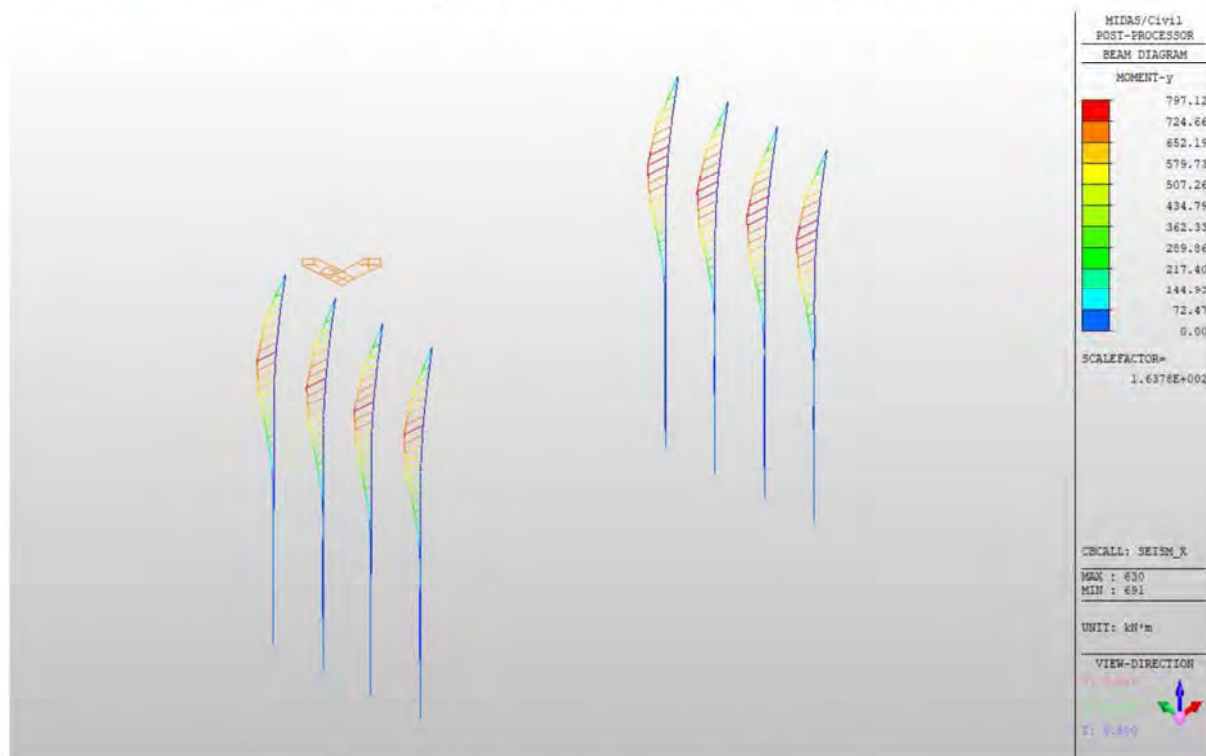


Fig. 7 Diagrama de moment încovoiator maxim din gruparea cLCB1

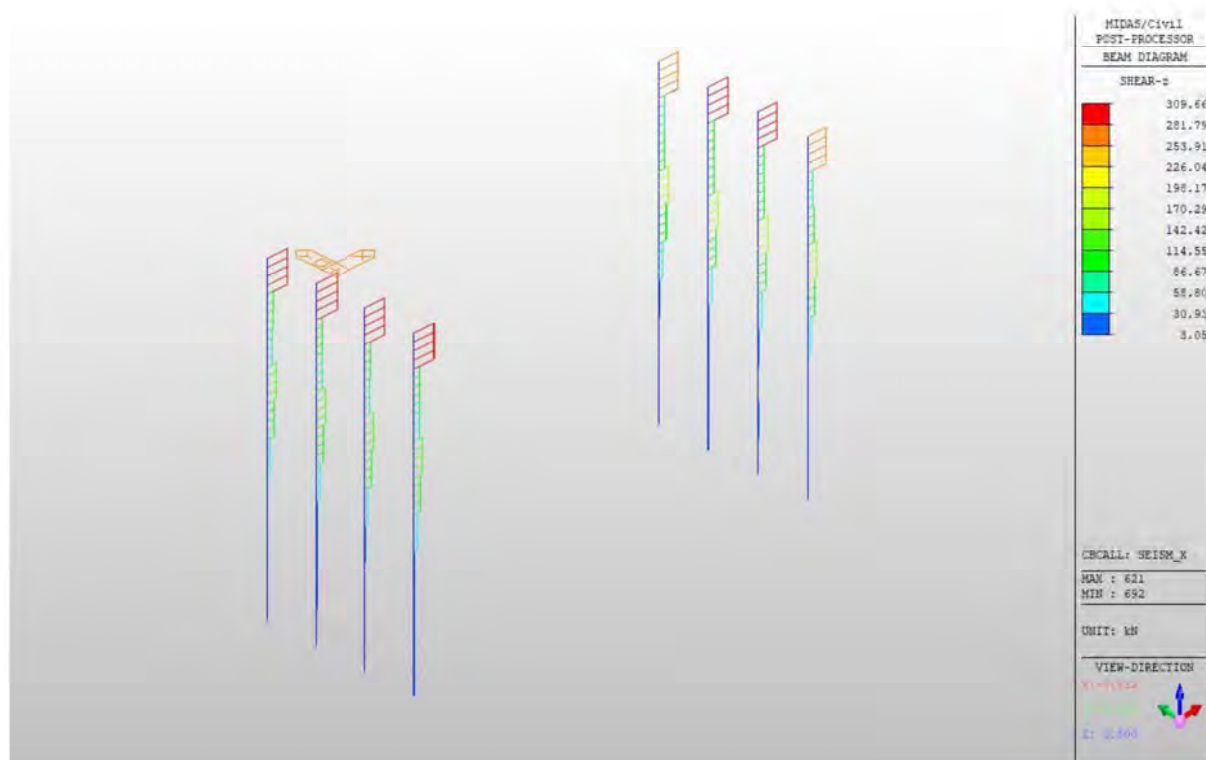


Fig. 8 Diagrama de forță tăietoare maximă din combinația cLCB1

PILOT CULEE**MIDAS/Civil****RC Column Checking Result**

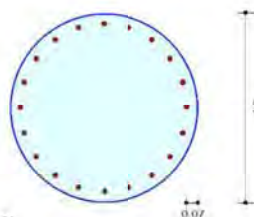
MIDAS	Company		Project Title	
	Author	Atti	File Name	CA...Str 10 - 5 grinzi de 27m.mcb

1. Design Condition

Design Code Eurocode2-2:05
 Unit System kN, m
 Member Number 642 (PM), 625 (Shear)
 Material Data $f_{ck} = 30000$, $f_{yk} = 500000$, $f_{yw} = 500000$ KPa
 Column Height 1.98 m
 Section Property Pilot_culee (No : 4)
 Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	20-P20	--	--

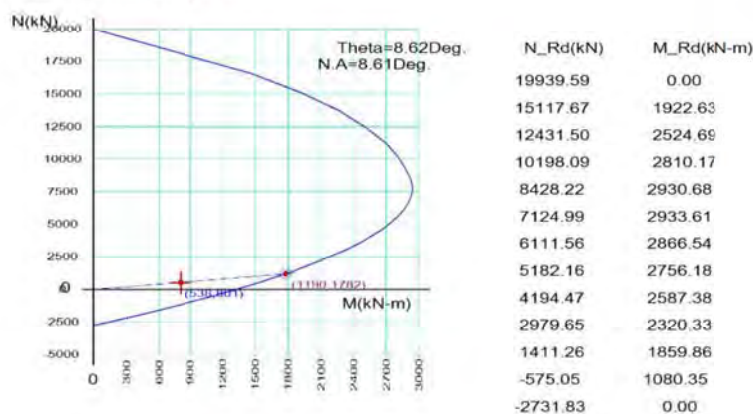
Total Rebar Area $A_{st} = 0.0062832 \text{ m}^2$ ($R_{hoist} = 0.0056$)

**2. Applied Loads**

Load Combination 33+ AT (I) Point
 $N_{Ed} = 538.118 \text{ kN}$, $M_{Edy} = 792.287$, $M_{Edz} = 119.926$, $M_{Ed} = 801.312 \text{ kN-m}$

3. Axial Forces and Moments Resistance Check

Concentric Max. Axial Load $N_{Rdmax} = 19939.6 \text{ kN}$
 Axial Load Ratio $N_{Ed}/N_{Rd} = 538.118 / 1190.41 = 0.452 < 1.000$ O.K
 Moment Ratio $M_{Edy}/M_{Rdy} = 792.287 / 1761.89 = 0.450 < 1.000$ O.K
 $M_{Edz}/M_{Rdz} = 119.926 / 267.141 = 0.449 < 1.000$ O.K
 $M_{Ed}/M_{Rd} = 801.312 / 1782.02 = 0.450 < 1.000$ O.K

4. P-M Interaction Diagram**5. Shear Force Resistance Check**

Applied Shear Resistance $V_{Ed} = 319.171 \text{ kN}$ (Load Combination 33+)
 Shear Strength by Conc $V_{Rdc} = 421.018 \text{ kN}$
 Shear Resistance by Rebar $V_{Rds} = 483.637 \text{ kN}$ (2.0-P10 @100)
 Shear Ratio $V_{Ed}/V_{Rd} = 319.171 / 421.018 = 0.758 < 1.000$ O.K

4.2.3 DIMENSIONARE ARMĂTURĂ RADIER

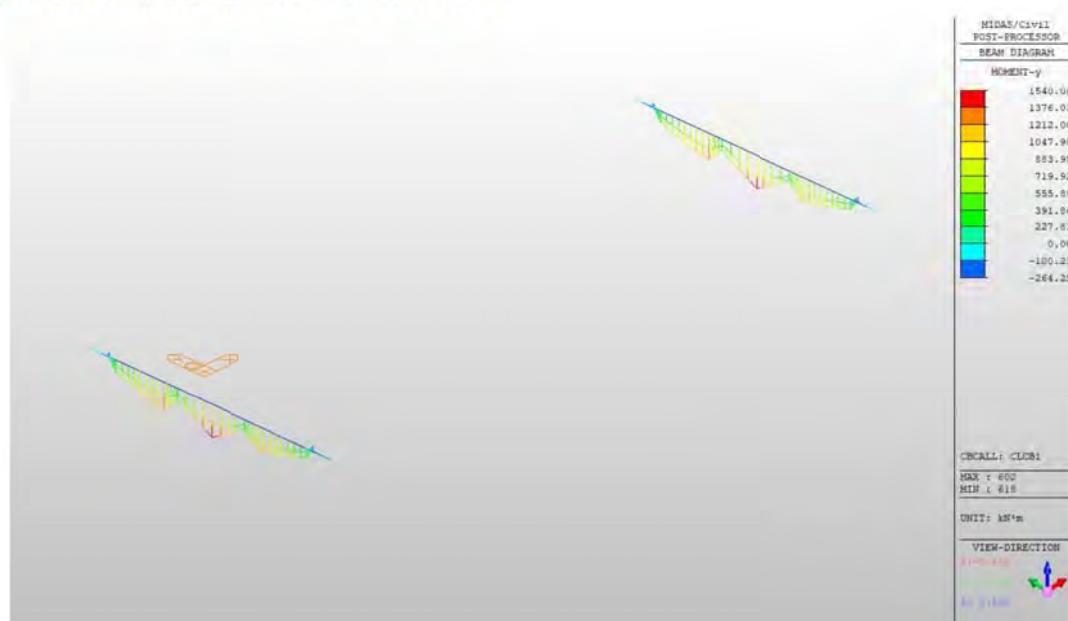



Fig. 9 Moment încovoietor în radierul de legătură

RADIER CULEE

MIDAS/Civil

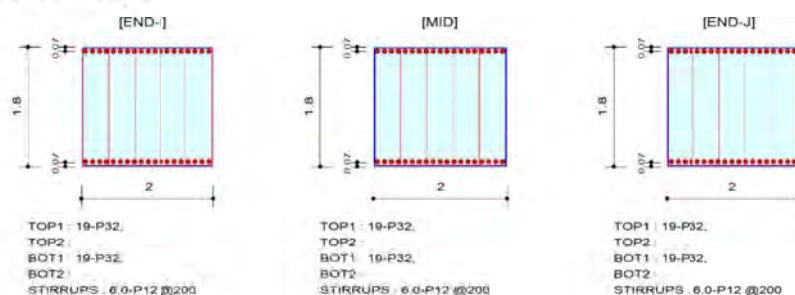
RC Beam Checking Result

	Company		Project Title	
	Author	Atti	File Name	C:\...Str 10 - 5 grinzi de 27m.mcb

1. Design Information

Design Code : Eurocode2-2:05
 Unit System : kN, m
 Material Data : fck = 35000, fyk = 500000, fyw = 500000 KPa
 Beam Span : 0.35 m
 Section Property: Radier (No : 9)

2. Section Diagram



3. Bending Moment Capacity

	END-I	MID	END-J
Negative Moment (M _{Ed})	264.25	154.75	264.25
(-) Load Combination No.	1-	1-	1-
Factored Strength (M _{Rd})	10919.49	10919.49	10919.49
Check Ratio (M _{Ed} /M _{Rd})	0.0242	0.0142	0.0242
Positive Moment (M _{Ed})	1540.06	1304.98	1540.06
(+) Load Combination No.	1+	1+	1+
Factored Strength (M _{Rd})	10919.49	10919.49	10919.49
Check Ratio (M _{Ed} /M _{Rd})	0.1410	0.1195	0.1410
Using Rebar Top (A _{s_top})	0.0153	0.0153	0.0153
Using Rebar Bot (A _{s_bot})	0.0153	0.0153	0.0153

4. Shear Capacity

	END-I	MID	END-J
Load Combination No.	1-	1+	1+
Factored Shear Force (V _{Ed})	1850.07	1839.43	1850.07
Shear Strength by Conc. (V _{Rdc})	1385.94	1385.94	1385.94
Shear Strength by Rebar (V _{Rds})	2296.91	2296.91	2296.91
Using Shear Reinf. (A _{sw})	0.0007	0.0007	0.0007
Using Stirrups Spacing	6.0-P12 @200	6.0-P12 @200	6.0-P12 @200
Check Ratio	0.8055	0.8008	0.8055

4.2.4 Eforturi in grinzi

Grinzile care intra in alcatuirea tablierului si ale caror caracteristici geometrice au fost descrise mai sus, au fost calculate tinandu-se seama de etapele de executie si de influenta fenomenelor reologice (contractie si curgere lenta).

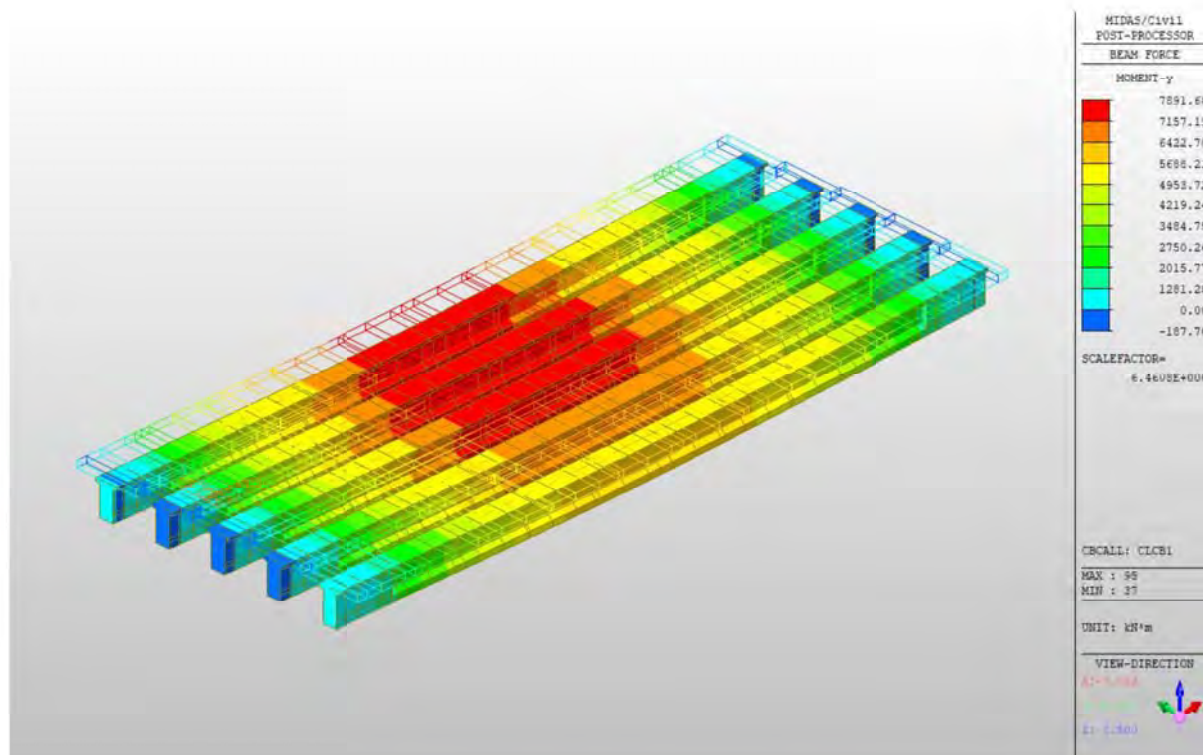


Fig. 10 Diagrama moment incovoietor grinzi

Dimensionarea grinzilor precomprimate s-a detaliat în breviare dinstincte pentru fiecare tip de grindă în parte.

5 ANEXA

PILOTI DE FUNDATIE

NORME DE REFERINTA

Pentru fazele de calcul se face referire la normele:

Eurocodul 7: Proiectare geotehnica – Partea 1: Reguli generale.

Eurocodul 8: Indicatii de proiect pentru rezistenta seismică a structurilor - Partea 5: Fundatii, structuri de sprijin, si specte geotehnice.

Sarcina Limita Verticala

Sarcina limită verticală a fost calculată cu formule statistice, care o exprimă în funcție de geometria coloanei consolidate, caracteristicile terenului și interfata pilot-teren. Astfel, întrucât realizarea unui pilot, bătut sau forat, modifică întotdeauna caracteristicile terenului în jurul acestuia, se propune considerarea unui unghi de rezistență la tăiere egal cu:

$$\begin{aligned}\phi' &= \frac{3}{4} \phi + 10^\circ \quad \text{pentru piloti bătuti} \\ \phi' &= \phi - 3^\circ \quad \text{pentru piloti forati}\end{aligned}$$

unde ϕ este unghiul de rezistență la tăiere înainte de execuție. În continuare vom indica cu ϕ parametrul de rezistență ales.

La sfârșitul calculului, sarcina limită Q_{lim} este divizată în mod conventional în două cote, rezistența la vârf Q_p și rezistența laterală Q_l .

Rezistența Unitară La Varf

Formula lui Terzaghi

Soluția propusă de Terzaghi consideră că terenul existent deasupra adâncimii la care a ajuns vârful pilotului poate fi înlocuit de o suprasarcină echivalentă cu tensiunea verticală efectivă (neglijând faptul că interacțiunea dintre pilot și fundație ar putea modifica această valoare) și conduce analiza la problema capacității portante a unei fundații superficiale.

Formula lui Terzaghi poate fi scrisă:

$$Q_p = c \times N_c \times s_c + \gamma \times L \times N_q + 0.5 \times \gamma \times D \times N_\gamma \times s_\gamma$$

unde:

$$N_q = \frac{a^2}{2 \cos^2(45^\circ + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2) \tan \phi}$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = \frac{\tan \phi}{2} \left(\frac{K_{p\gamma}}{\cos^2 \phi} - 1 \right)$$

Metoda lui Berezantzev

În principiu Berezantzev face referire la o suprafață de alunecare "alla Terzaghi" care se oprește pe planul de sprijin (vârful pilotului); totuși acesta consideră că cilindrul de teren coaxial pilotului are diametrul egal cu

extensia în secțiune a suprafeței de alunecare, este în parte “susținut” prin acțiunea tangentială de către terenul rămas de-a lungul suprafeței laterale. Acesta dă o valoare a presiunii la baza inferioară a lui γD , și mai mică cu cât acest efect de “siloz” este marcant, adică cu cât mai mare este raportul D/B ; de acesta ține cont coeficientul N_q , care este deci funcție descrescătoare a lui D/B .

Rezistența unitară Q_p la vârf, pentru cazul terenului cu forfecarea (ϕ) și coeziunea (c), este dată de expresia:

$$Q_p = c \times N_c + \gamma \times L \times N_q$$

Indicând cu:

γ greutatea unității volumice a terenului;
 L lungimea coloanei;
 N_c e N_q sunt factorii capacității portante afectați de efectul formă (circulară);

Metoda lui Vesic

Vesic a asimilat problema rupturii în jurul vârfului pilotului și aceea a expansiunii unei cavități cilindrice în mediu elasto-plastic, în așa fel încât să se țină cont și de compresibilitatea mediului.

După Vesic coeficienții capacității portante N_q și N_c se pot calcula după cum urmează:

$$N_q = \frac{3}{3 - \sin \phi} \left\{ \exp \left[\left(\frac{\pi}{2} - \phi \right) \tan \phi \right] \tan^2 \left(45 + \frac{\phi}{2} \right) I_{rr}^{(4 \sin \phi) / [3(1 + \sin \phi)]} \right\}$$

Indicele de rigiditate redus I_{rr} , în expresia precedentă este calculat plecând de la deformarea volumică ε_v .

Indicele de rigiditate I_r se calculează utilizând modulul de elasticitate elastică tangentială G' și rezistența la tăiere a terenului s .

Când avem de-a face cu condiții nedrenate sau solul se găsește într-o stare de densă, termenul ε_v poate fi considerat egal cu zero și se obține $I_{rr} = I_r$.

Este posibilă estimarea lui I_r cu următoarele valori:

TEREN	I_r
Nisip	75-150
Praf	50-75
Argilă	150-250

Termenul N_c al capacității portante este calculat:

$$N_c = (N_q - 1) \cot \phi \quad (a)$$

Când $\phi = 0$ (condiții nedrenate)

$$N_c = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

Metoda lui Janbu

Janbu calculează N_q (cu unghiul ψ) după cum urmează:

$$N_q = \left(\tan \phi + \sqrt{1 + \tan^2 \phi} \right)^2 \exp(2\psi \tan \phi)$$

N_c se poate calcula de la (a) când $\phi > 0$.

Per $\phi = 0$ se folosește $N_c = 5.74$

Formula lui Hansen

Formula lui Hansen este valabilă pentru orice raport D/B , deci pentru fundatii de suprafață, dar si pentru cel profunde, acelasi autor a introdus coeficienti pentru o mai bună interpretare a comportamentului real al fundatiei, fără acestia, sarcina limită ar fi prea mult mărită odată cu adâncimea.

Pentru valori $L/D > 1$:

$$d_c = 1 + 0.4 \tan^{-1} \frac{L}{D}$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 \tan^{-1} \frac{L}{D}$$

În cazul $\phi = 0$

D/B	0	1	1.1	2	5	10	20	100
d'_c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

În factorii următori, exeresiile cu acest semn (') sunt egale cu $\phi = 0$.

Factor de formă:

$$s'_c = 0.2 \frac{D}{L}$$

$$s_c = 1 + \frac{N_q}{N_c} \frac{D}{L}$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{D}{L}$$

Factor de adâncime:

$$d'_c = 0.4k$$

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)k$$

$$d_\gamma = 1 \text{ per qualsiasi } k$$

$$k = \tan^{-1} \frac{L}{D} \text{ se } \frac{L}{D} > 1$$

Rezistenta Trunchiului

Metoda utilizată pentru calcularea capacității portante laterale este metoda α , propusă de Tomlinson (1971); rezistenta laterală este calculată după cum urmează:

$$Q_1 = (\alpha c + \sigma K \tan \delta) \cdot A_l \cdot f_w$$

A_l = suprafața laterală a pilotului;

f_w = factor de corectie legat de conicitatea trunchiului pilotului, sau reducerea diametrului pilotului;

c = valoarea medie a coeziunii (sau a rezistentei la tăiere în conditii nedrenate);

σ = presiunea verticală eficace a terenului;

K = coeficientul de alunecare orizontală, depinde de tehnologia de executie a pilotului si de starea precedentă de densitate, care se calculează după cum urmează:

Pentru piloti bătuti

$$K = 1 - \tan^2 \phi$$

Sau, în cazul specific, este posibilă desemnarea valorilor propuse în tabel:

Pilot	K	
	Teren afânat	Teren dens
Otel	0.5	1
Beton pref.	1	2
Lemn	1	3

Pentru piloti forati:

$$K = 1 - \sin \phi$$

δ = forfecarea pilot-teren în functie de asprimea suprafetei pilotului;

Pentru piloti bătuti

$$\delta = 3/4 \tan \phi$$

Pentru piloti forati

$$\delta = \tan \phi$$

α = coeficientul de adeziune găsit astfel:

Piloti forati:

$$\alpha = \frac{100 + c^2}{100 + 7c^2}$$

Caquot – Kerisel

Meyerhof – Murdock (1963)

$$\alpha = 1 - 0.1 \cdot c \quad \text{pentru } c < 5 \text{ t/m}^2$$

$$\alpha = 0.525 - 0.005 \cdot c \quad \text{pentru } c \geq 5 \text{ t/m}^2$$

Whitaker – Cooke (1966)

$$\alpha = 0.9 \text{ pentru } c < 2.5 \text{ t/m}^2$$

$$\alpha = 0.8 \text{ pentru } 2.5 \leq c < 5 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 5 \leq c \leq 7.5 \text{ t/m}^2$$

$$\alpha = 0.9 \text{ pentru } c > 7.5 \text{ t/m}^2$$

Woodward (1961)

$$\alpha = 0.9 \text{ pentru } c < 4 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 4 \leq c < 8 \text{ t/m}^2$$

$$\alpha = 0.5 \text{ pentru } 8 \leq c < 12 \text{ t/m}^2$$

$$\alpha = 0.4 \text{ pentru } 12 \leq c \leq 20 \text{ t/m}^2$$

$$\alpha = 0.20 \text{ pentru } c > 20 \text{ t/m}^2$$

Piloti bătuti

Coeficientul α pentru pilot bătut	
$2.5 \leq c < 5 \text{ t/m}^2$	$\alpha = 1.00$
$5 \leq c < 10$	$\alpha = 0.70$
$10 \leq c < 15$	$\alpha = 0.50$
$15 \leq c < 20$	$\alpha = 0.40$
$c \geq 20$	$\alpha = 0.30$

Forfecarea Negativa

Când un pilot este bătut sau trece printr-un strat de material compresibil înainte de a se fi terminat procesul de consolidare, terenul se va mișca în raport cu pilotul făcând să apară eforturi de forfecare între pilot și teren care duc la așa-zisul fenomen de forfecare negativă. Efectul forfecării negative este acela de a crește sarcina axială asupra pilotului, având ca și consecință creșterea tasărilor. Forța care ia naștere ca rezultat al forfecării negative este estimată ca fiind egală cu componenta activă a rezistenței laterale (v. Rezistența Trunchiului) de-a lungul suprafeței laterale la contactul cu stratul în care se generează astfel de fenomen, dar din partea opusă forfecării negative. Rezultatul astfel determinat nu este dedusă din sarcina limită, ci din aceea de exercițiu.

Factorul De Corecție În Condiții Seismice

Criteriul lui Vesic

Vesic apreciază că pentru a ține cont de fenomenul de creștere în calcul a capacității portante este suficientă reducerea cu 2° a unghiului de forfecare a straturilor de fundație. Limita acestei sugestii este faptul că nu ține cont de intensitatea solicitării seismice (exprimată prin parametrul accelerației seismice orizontale maxime). Acest criteriu pare să găsească confirmarea în observațiile făcute în prezenta diverselor fenomene seismice.

Criteriul lui Sano

Autorul propune reducerea unghiului de forfecare a straturilor portante cu o cantitate dată în ecuația:

$$D_p = \arctg\left(\frac{A_{\max}}{\sqrt{2}}\right)$$

unde A_{\max} reprezintă accelerația seismică orizontală maximă.

Acest criteriu, față de cel al lui *Vesic*, reprezintă avantajul de a lua în considerare și intensitatea solicitării seismice. Experiența demonstrează totuși că aplicarea fără apelul la critică a acestei realități poate conduce la valori prea mici ale lui *Qlim*.

Corecțiile lui *Sano* și *Vesic* se aplică exclusiv pentru terenuri necoezive dense. Este greșită aplicarea lor la terenurile afânate sau cu densitate medie, unde vibrațiile seismice produc fenomenul opus celui de dilatare, cu creșterea gradului de densitate și a unghiului de forfecare.

TASARI

Tasarea verticală a fost calculată cu metoda lui Davis-Poulos, în funcție de care pilotul este considerat rigid (nedeformabil) înecat într-un mediu elastic, semispațiu sau strat de grosime finită.

Se ipotizează că interacțiunea pilot-teren este constantă la fâșii de-a lungul a n suprafețe cilindrice în care se împarte suprafața laterală a pilotului.

Tasarea suprafeței generice i ca efect la sarcina limită transmisă de la pilot la teren de-a lungul suprafeței cu nr. j poate fi exprimat:

$$W_{i,j} = (\tau_j / E) \times B \times I_{i,j}$$

Unde:

τ_j = Creșterea tensiunii în funcție de punctul mediu al fâșiei;

E = Modulul elastic al terenului;

B = Diametrul pilotului;

$I_{i,j}$ = Coeficientul de influență.

Tasarea compresivă se obține însumând $W_{i,j}$ cu toate ariile j .

SARCINA LIMITĂ ORIZONTALĂ

Sarcina limită orizontală a fost calculată după teoria dezvoltată de Broms, care presupune că comportamentul interfetei pilot-teren este rigidă perfect elastică, și astfel rezistența terenului să fluctueze cu orice valoare, nu nulă, a deplasării și să rămână constantă la creșterea deplasării.

Se presupune că comportamentul flexionar al pilotului este de tip rigid, perfect elastic, trebuie să spunem că rotațiile elastice ale pilotului sunt neglijabile până când momentul încovoietor ajunge la valoarea M_y de plasticizare.

Pentru terenuri coesive Broms propune adoptarea unei reacții a terenului constantă cu adâncimea egală cu:

$$p = 9 \times c_u \times B$$

cu reactia nulă până la adâncimea de 1.5 d; indicând cu:

c_u = Coeziunea nedrenată,

B = Diametrul pilotului;

p = Reactia terenului pentru unitatela de lungime a pilotului.

Pentru terenurile necoezive se presupune că rezistenta variază liniar în functie de adâncime după formula:

$$p = 3K_p \gamma zB$$

indicând cu:

p = Reactia terenului pentru unitatela de lungime a pilotului;

K_p = Coeficientul de împingere pasivă;

γ = Greutatea volumică a terenului;

z = Adâncimea;

B = Diametrul pilotului

PILOT ÎN CONDITII DE EXERCITIU

Analizza pilotului în conditii de exercitiu: **Metoda elementelor finite.**

Metoda elementelor finite modelează pilotul de fundatie supus la sarcini transversale, în mod realist întrucât face uz, fie de împingeri cât si de rotatiile în noduri pentru definirea linieri elastice a pilotului, deci reprezintă metoda cea mai ratională si eficientă disponibilă actualmente pentru analizarea acestor tipuri de structuri.

În ceea ce urmează se face apel la teoria metodi, indicând cu **P** matricea fortelor nodale externe, cu **F** aceea a fortelor interne si cu **A** matricea coeficientilor de influentă care, pentru echilibrul între fortele externe si interne, leagă primele două în functie de formula:

$$\mathbf{P} = \mathbf{A}\mathbf{F}$$

Deplasările interne **e** (împingerile si rotatiile) ale elementului noduli generic sunt legate de deplasările externe **X** (împingerile si rotatiile) aplicate la noduri, ale următoarei formule:

$$\mathbf{e} = \mathbf{B}\mathbf{X}$$

unde matricea **B** s-a demonsatrat a fi transpunerea matricei **A**.

De altă parte, fortele interne **F** sunt legate de deplasările interne **e** ale următoarei expresii:

$$\mathbf{F} = \mathbf{S}\mathbf{e}$$

Aplicând substituirile de rigoare se obtine:

$$\mathbf{F} = \mathbf{S}\mathbf{A}^T\mathbf{X}$$

Si deci:

$$\mathbf{P} = \mathbf{A}\mathbf{F} = \mathbf{A}\mathbf{S}\mathbf{A}^T\mathbf{X}$$

Deci, calculând inversa matricei **A S A^T** se obtine expresia deplasărilor externe **X**:

$$\mathbf{X} = (\mathbf{A}\mathbf{S}\mathbf{A}^T)^{-1}\mathbf{P}$$

Notati, deci, că cu ajutorul deplasărilor **X** este posibilă aflarea fortelor interne **F** necesare pentru proiectul structurii.

Matricea **A S A^T** este cunoscută ca si matrice de rigiditate globală deoarece caracterizează legătura dintre deplasări si fortele externe nodale.

Varianța de ocolire Timisoara Sud - Pasaj km 18+926 peste canal de irigații

Metoda elementelor finite are, printre altele, avantajul că dă posibilitatea de a lua în considerare, ca și condiții la contur, rotații și deplasări cunoscute.

Rotările nodale care schematizează terenul sunt considerate foarte globale legate de modulul de reacție și de zona de influență a nodului. În cadrul soluției elementelor finite pentru piloni supuși la sarcini transversale, modulul de reacție este considerat în forma:

$$k_s = A_s + B_s Z^n$$

sau, nevrând să creștem nelimitat k_s cu adâncimea, în forma:

$$k_s = A_s + B_s \tan^{-1}(Z/B)$$

în care Z reprezintă adâncimea iar B reprezintă diametrul pilotului.

Valorile lui A_s și $B_s Z^n$ sunt obținute din expresia capacității portante (Bowles) cu factori corectori s_i , d_i , și i_i egali cu 1:

$$k_s = q_{ult}/\Delta H = C(cN_c + 0.5\gamma B N_\gamma)$$

$$B_s Z^n = C(\gamma N_q Z^1)$$

Unde $C = 40$ este obținut în corespondență cu o tasare maximă de 25 mm.

Date generale...

Descriere	dn 1200mm, km 18+926
Diametru varf	1.20 m
Lungime	14.00 m
Tip	Forat
Capacitatea de varf calculata cu:	Terzaghi
Adancime panza freatica de la nivelul solului	2.40 m
Tip beton	1
Tip otel	1

SEISM

Acceleratie seismica	0.2
Coefficient de intensitate seismica [Kh]	0.1
Coefficient de intensitate seismica [Kv]	0.07

Arhiva materiale

Beton

Nr.	Clasa Beton	fck,cubic [Kg/cm2]	Ec [Kg/cm2]	fck [Kg/cm2]	fcd [Kg/cm2]	fctd [Kg/cm2]	fctm [Kg/cm2]
1	C20/25	250	299600	200	113.3	10.1	22.1
2	C25/30	300	314750	250	141.6	11.4	25.6
3	C28/35	350	323080	280	158.6	12.6	27.6
4	C40/50	500	352200	400	226.6	16.3	35

Oteluri:

Nr.	Steel Class	Es [Kg/cm2]	fyk [Kg/cm2]	fyd [Kg/cm2]	ftk [Kg/cm2]	ftd [Kg/cm2]	ep_tk	epd_ult	B1*B2 in.	B1*B2 fin.
1	B450C	2000000	4500	3913	4500	3913	.075	.0675	1	0.5
2	B450C*	2000000	4500	3913	5400	4500	.075	.0675	1	0.5
3	B450C**	2000000	4500	3913	4582	3985	.012	.01	1	0.5
4	S235H	2141370	2447.28	2128.11	3670.92	2128.11	0.012	0.01	1	0.5
5	S275H	2141370	2855.16	2482.97	4384.71	2482.97	0.012	0.01	1	0.5
6	S355H	2141370	3670.92	3191.66	5200.47	3670.92	0.012	0.01	1	0.5

Stratigrafie

Varianta de ocolire Timisoare Sud - Pasaj km 18+926 peste canal de irigatii

Nr.: Numarul stratului Hs: Grosimea stratului Fi: Unghi de frecare c: Coeziune Alfa: Coef. aderenta frecari laterale. Vs: Viteza unde de forfecare.

Strat. 1

Nr.	Hs	Greutate volumica [kg/m³]	Greutate volumica saturata [kg/m³]	c [kg/cm²]	Fi (°)	Frecare negativa	Alfa	Modul elasticitate [kg/cm²]	Vs [m/s]	Descriere litologica
1	0.20	1650.00	1650.00	0.00	15.00	Nu	1.00	15.00	0	
2	4.30	2000.00	1950.00	0.04	20.00	Nu	0.00	40.00	0	argila si praf
3	1.50	2000.00	1925.00	0.04	20.00	Nu	0.00	40.00	0	argila si praf
4	2.20	1850.00	1800.00	0.00	32.00	Nu	0.00	300.00	0	nisip usor praos
5	2.30	2000.00	1900.00	0.04	20.00	Nu	0.00	40.00	0	argila cu nisip
6	1.90	2100.00	2050.00	0.06	25.00	Nu	0.00	60.00	0	argila si praf
7	2.00	1900.00	1850.00	0.00	36.00	Nu	0.00	500.00	0	nisip praos
8	5.10	1900.00	1850.00	0.00	36.00	Nu	0.00	500.00	0	nisip si praf
9	1.50	1900.00	1875.00	0.00	36.00	Nu	0.00	500.00	0	nisip usor praos
10	2.50	1900.00	1875.00	0.00	36.00	Nu	0.00	500.00	0	nisip praos
11	2.00	1900.00	1850.00	0.00	36.00	Nu	0.00	500.00	0	nisip si praf

Sacina limita

Stratigrafie	Nq	Nc	Fi/C strat varf pilot (°)/[kg/cm²]	Greutate pilot [kg]	Sarcina limita varf [kg]	Sarcina limita laterala [kg]	Sarcina limita [kg]	Frecare negativa [kg]	Sarcina limita orizontala [kg]
Teoria clasica	44.19	60.55	35.5/0.00	39584.07	764506.80	246386.30	971309.00	--	--

REZISTENTA DE PROIECTARE SARCINI AXIALE

Rezistenta caracteristica sarcini axiale. Nume combinatie:				Teoria clasica
Numar de verticale inspectate				1
Factor de corelatie pentru verticale inspectate mediu (xi3)				1.70
Factor de corelatie pentru vertical inspectate minim (xi4)				1.70
	Rc, Min [kg]	Rc, Media [kg]	Rc, Max [kg]	
Baza	764506.80	764506.80	764506.80	
Lateral	246386.30	246386.30	246386.30	
Total	971309.00	971309.00	971309.00	
Coeficient partial rezistenta caracteristica				R1
Baza				1.00
Lateral				1.00
Rezistenta de proiectare baza				449709.80 kg
Rezistenta de proiectare laterala				144933.10 kg
Rezistenta de proiectare				555058.90 kg

Tasare (Fleming 1992)

Lungime	14.00 m
Diametru capat	1.20 m
Diametru varf	1.20 m
Tronson frecare laterala nula	0.00 m

Modul elasticitate sectiune	2.94E+07 kN/mq
Punct de aplicare a rezultantei rezistentei active	0.45
Factor de flexibilitate teren/pilot	1.00E-03
Sarcina aplicata	2070.00 kN
Sarcina limita laterala	1443.00 kN
Sarcina limita de varf	4497.00 kN
Modul elasticitate teren corespondent	3.26E+03 kN/mq
Scurtarea elastica	0.54 mm
Tasare rigida	114.95 mm
Tasare totala	115.49 mm



intocmit ing. Attila Takacs



POD KM 20+875 PESTE PESTE CANAL DE IRIGATII

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1 DESCRIERE SI BAZA DE PROIECTARE

1.1 Descrierea structurii

Varianta de ocolire Timisoara Sud intersecteaza un canal de irigatii la km 20+875 motiv pentru care se impune construirea unui pod cu o singura deschidere. Zidurile intoarse situate in apropierea canalului de irigatii vor fi continuate cu ziduri de sprijin pentru amplasarea optima a sferturilor de con. Podul va fi amplasat pe o clotoida si va avea o oblicitate de 75° , podul intesectand canalul cu o oblicitate de 40° .

Suprastructura podului este alcatuita din 6 grinzi prefabricate precomprimate cu armatura preintinsa, avand lungimea de 40 m si inaltimea de 2 m, solidarizate la partea superioara prin intermediul unei placi monolite din beton armat.

Schema statica adoptata este cea de grinda simplu rezemata, rezemarea facandu-se pe infrastructuri cu aparate de reazem din neopren dispuse pe cuzineti din beton armat.

Pe fiecare deschidere, pentru solidarizarea transversala a grinzilor se vor executa o antretoaza de camp si antretoaze de capat avand si rol de sprijin pentru liftarea tablierului in viitor in vederea inlocuirii aparatelor de reazem din neopren.

Placa de suprabetonare se va executa din beton armat, turnat peste predale prefabricate din beton armat care au rol de cofraj pierdut. Consolele trotuarelor se vor executa monolit (fara cofraj pierdut din predale prefabricate). Peste placa de suprabetonare se va aterne hidroizolatie avand 1 cm grosime, protectia hidroizolatiei in grosime de 3 cm din BA8 si straturile rutiere in grosime de 2x4 cm din BAP 16 respectiv MAS 16 conform Proiect Ilustrativ.

Suprastructura va avea rosturi de dilatare la ambele capete (la culei).

Podul fiind amplasat in curba, profilul acestuia pe zona carosabilului in sens transversal este cu panta unica de 2,5% ,iar pe zona trotuarelor la nivel panta transversala a acestora este de 1% inspre axul structurii.

Conform proiectului ilustrativ, in sectiune transversala suprastructura prezinta urmatoarele caracteristici: 7.80 m parte carosabila, incadrata cu parapete New Jersey 0.60m, doua trotuare la nivel de 1.00 m latime utila fiecare, si longrine cu latimea de 0.30 m, latimea totala a suprastructurii fiind de 11.60 m.

La marginea partii carosabile au fost prevazute parapete de siguranta de tip rigid (parapete New Jersey din beton armat cu grad de asigurare H4b) pentru protectia pietonilor iar pe partile exterioare ale trotuarelor se vor prevedea parapete pietonale metalice.

Lungimea totala a pasajului este $L_{tot}=47.95$ m.

Infrastructurile sunt prevazute a fi executate din beton armat, fundate indirect, pe piloti forati de diametru mare $d=1200$ mm dispusi pe un singur rand. Culeele vor avea elevatie "plina" (de tip perete din beton armat), iar pe banchetele de rezemare se vor executa cuzineti si dispozitive antiseismice din beton armat. Pe suprafetele elevatiilor se va prevedea protectie anticoroziva de suprafata.

Racordarea cu terasamentele se va realiza cu sferturi de con pereate cu beton si cu ziduri de sprijin. Pe partea carosabila pentru tranzitia de la

tablierul podului la rambleul drumului se vor executa placi de racordare din beton armat cu lungimea de 6.00 m. In spatele elevatiilor, pentru drenarea apelor de infiltratie se vor executa drenuri din piatra bruta invelite in geotextil. Pe taluzele sferturilor de con se vor prevedea cascari pentru scurgerea apelor si scari de acces din beton, dupa caz.

Urmatoarele faze de executie sunt considerate:

- Executarea piloților și a radierelor;
- Executarea elevatiilor pentru culei si pile;
- Poziționarea grinzilor;
- Pozitionarea predalelor, armarea placii de suprabetonare si a antretoazelor;
- Betonarea placii de suprabetonare si a antretoazelor;
- Executarea terasamentelor;
- Executarea caii, parapetilor, lisei si a bordurilor.

Clasa de incarcare: Eurocode: LM1

Clasa de importanta: "C"

Cerinte de calitate: A4, B2, D2.

Proiectare seismica: $a_g = 0.20g$, $T=0.7s$ conform P100/2013

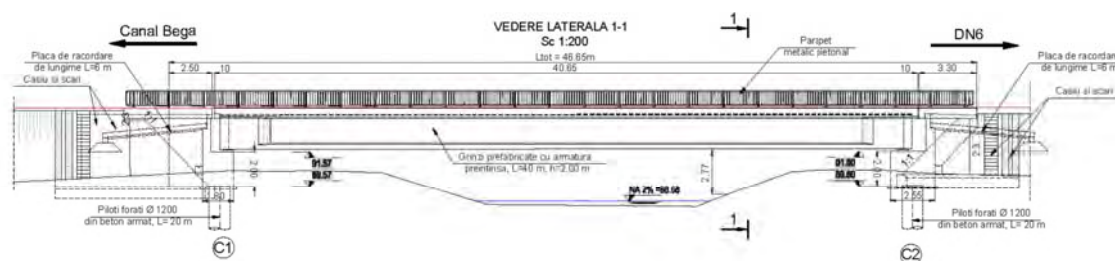


Fig. 1 Elevatie

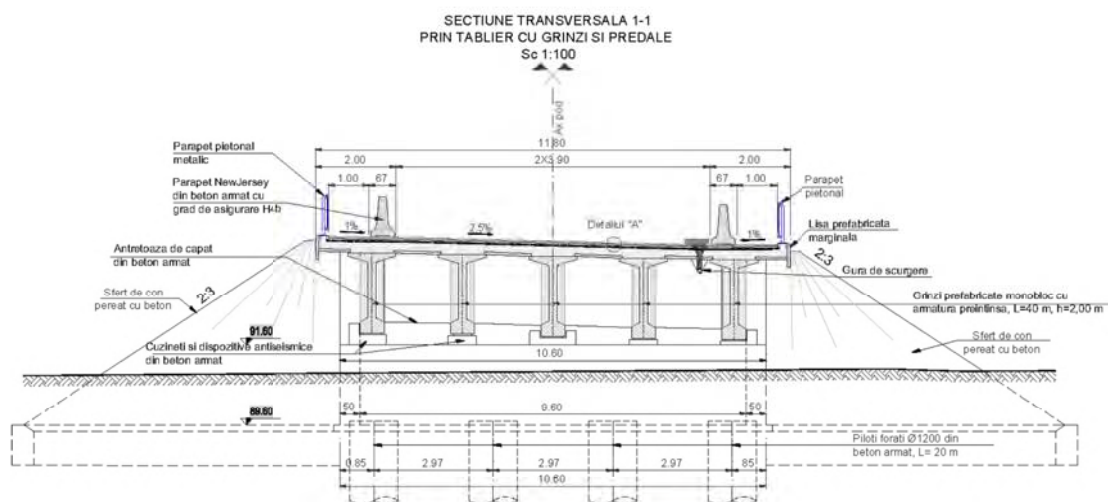


Fig. 2 Sectiune transversala

1.2 Baza de proiectare si materiale

Instructiuni si standarde considerate in proiectarea structurii:

- Eurocode 1: Actiuni asupra structurilor
- Eurocode 2: Proiectarea structurilor de beton
- Eurocode 7: Proiectarea geotehnica
- Eurocode 8: Proiectarea pentru rezistenta la cutremur a structurilor
- P100/2013: Cod de proiectare seismica

Caracteristicile minime ale materialelor:

- Otel pentru armatura: BST 500 C
- Otel pentru precomprimare: Y 1860
- Beton in coloane: C25/30
- Beton in radier culei si pile: C25/30
- Beton in elevatii culei si pile: C30/37
- Beton pentru turnarea placii in-situ, pentru grinziile de parapet si lisele monolite: C35/45;
- Beton pentru grinzi prefabricate: C50/60

2 ACTIUNI

2.1 Actiuni considerate

In concordanta cu Eurocode 1, actiunile considerate in proiectarea structurii sunt:

- Actiuni permanente (G):
 - Greutatea proprie a grinzilor, placilor, elevatiei, radierelor si a pilotilor (Gp);
 - Greutatea caii, parapetelor, trotuar (Cpt);
 - Impingerea pamantului (Imp);
 - Impingerea pamantului din seism (Imp_s);
- Actiuni variabile (Q):
 - Actiuni din trafic la poduri (SR EN 1991-2:2005) (convoiul LM1, franarea si forta centrifuga, F);
 - Actiuni termice, temperatura (T-) si T(+) si gradient de temperatura (TG-) si (TG+).
- Actiuni accidentale (A):
 - Actiuni seismice, Seism x (longitudinal), Seism y (transversal) , Seism z (vertical).

2.2 Valori caracteristice

2.2.1 Actiuni permanente (G).

2.2.1.1 Greutatea proprie, G_p si C_{pt}

Element	buc	Arie m^2	Densitate kN/m^3	G_n kN/m	γ	G_c kN/m
Grinzi $H=2.00m$	5	0.673	25	84.13	1.35	113.57
Placa grinzi marginale	2	0.68	25	34.00	1.35	45.90
Placa grinzi centrale	2	0.765	25	38.25	1.35	51.64
Cale	1	1.3464	24	32.31	1.35	43.62
Lisa prefabricata	2	0.043	25	2.15	1.35	2.90
Parapet directional	2	-	-	11.00	1.35	14.85
Parapet pietonal	2	-	-	1.50	1.35	2.03
Grinda parapet	2	0.042	25	6.50	1.35	8.78
TOTAL / ml				209.84		283.28

2.2.1.2 Impingerea pamantului (I_{mp})

$h=$	2	m	Inaltime elevatie
$H_{zid}=$	2.5	m	inaltime zid de garda
$H_T=$	4.5	m	Inaltime totala
$\phi_{df}=\phi_{kf}=$	33	grade	
$g_{kd}=g_{kf}=$	20	kN/m^3	
$K_{ad}=tg(\pi/4-\phi_{df}/2)^2=$	0.333	coef. presiune activa	
$p=K_{ad} \cdot \gamma_{kf} \cdot h=$	29.97	KN / m^2 ; presiune activa	

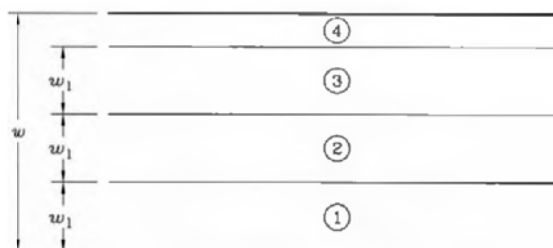
2.2.2 Actiuni variabile (Q)

2.2.2.1 Actiuni din trafic la poduri (SR EN 1991-2:2005) (LM1);

Pentru determinarea efectelor zonelor de incarcarea a partii carosabile a unui pod aceasta este divizata in benzi teoretice.

Lățimea părții carosabile w	Numărul benzilor teoretice de circulație	Lățimea unei benzi teoretice de circulație	Lățimea zonei rămase
$w < 5,4 \text{ m}$ $5,4 \text{ m} \leq w < 6 \text{ m}$	$n_l = 1$ $n_l = 2$	3 m $\frac{w}{2}$	$w - 3 \text{ m}$ 0
$6 \text{ m} \leq w$	$n_l = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTĂ – De exemplu, pentru o parte carosabilă cu lățimea de 11 m , $n_l = \text{Int}\left(\frac{w}{3}\right) = 3$, și lățimea zonei rămase este: $11 - 3 \times 3 = 2 \text{ m}$.			

Numărul, poziția și numerotarea benzilor teoretice încărcate, se realizează astfel încât efectele convoaielor de calcul să fie cele mai defavorabile în scopul unei verificări individuale.



w = Lățimea părții carosabile

w_1 = Lățimea unei benzi teoretice de circulație

1 = Banda teoretică nr. 1

2 = Banda teoretică nr. 2

3 = Banda teoretică nr. 3

4 = Zona rămasă

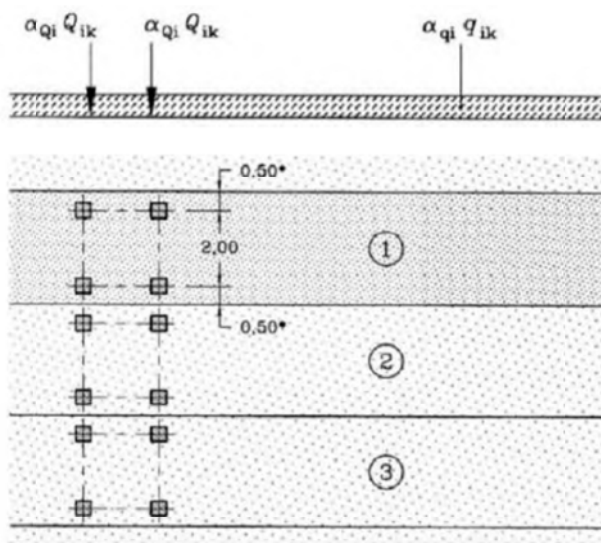
2.2.2.1.1 Convoiul de calcul 1 (LM1) :

Acest convoi este alcătuit din încărcări concentrate și încărcări uniform distribuite ce acoperă cele mai multe efecte din traficul alcătuit din camioane și automobile, fiind utilizat atât pentru verificări locale cât și globale.

Încărcările concentrate sunt reprezentate de sistemul tandem TS alcătuit din 2 osii cu greutatea " $\alpha_Q Q_k$ " pe fiecare osie.

Notă: atât pentru verificări locale cât și generale se va aplica un singur sistem tandem pe fiecare bandă de circulație cu precizarea că poziționarea acestuia se va face centrat pe axul fiecărei benzi, în cazul verificărilor globale, și în poziția cea mai defavorabilă în cazul celor locale, fără însă a depăși limita de 0.5 m între două sisteme de pe două benzi adiacente.

Încărcările uniform distribuite sunt reprezentate de sistemul UDL cu greutatea "α_{qk}" pe metru pătrat;



Coeficienții de corecție pentru încărcările uniform distribuite și concentrate "α_q" și "α_Q" se iau egali cu 1 (conform secțiunea 4-4.3.2(3)-SR EN 1991-2:2004/NB:2006).

Poziția	Sistemul tandem TS	Sistemul UDL
	Încărcarea pe osie Q_{ik} (kN)	q_{ik} (sau q_{rk}) (kN/m ²)
Banda numărul 1	300	9,0
Banda numărul 2	200	2,5
Banda numărul 3	100	2,5
Alte benzi	0	2,5
Zona rămasă (q_{rk})	0	2,5

Nota: Valorile încărcărilor prezentate în tabel includ amplificarea dinamică.

2.2.2.1.2 Forta de franare (SR EN 1991-2:2005), F

Forța de franare “ Q_{lk} ” este o forță ce acționează în sens longitudinal podului la nivelul suprafeței de rulare a părții carosabile.

Valoarea caracteristică Q_{lk} este limitată la 900 kN pentru întreaga lățime a podului, și trebuie calculată ca fracțiune din totalul sarcinilor verticale maxime corespunzătoare convoiului LM1, aplicabile benzii nr.1 după cum urmează:

$$Q_{lk} = 0.60\alpha_{Q1} (2Q_{1k}) + 0.10 \alpha_{q1} q_{1k} w_1 L \leq 900 (\text{kN})$$

L = lungimea tablierului sau a părții considerate în calcul.

Pentru cazurile curente când lungimea de încărcare $L > 1,2\text{m}$ forța de franare este egală cu $Q_{lk} = 360 + 2,7L \leq 900 (\text{kN})$.

Limita inferioară a forței de franare la podurile de șosea este de 300kN.

$L_{cale} =$	40.6	m	$w_l =$	3	m
$Q_{lk} =$	300	kN	$\alpha_{Q1} =$	1	
$q_{lk} =$	9	kN/m ²	$\alpha_{q1} =$	1	
$Q_{lk} =$	469.62	kN	<	900	kN
$Q_{lk} =$	469.62	kN			
Nr Grinzi	5				
$Q_{lk} / grinda =$	93.924	kN/m - forta uniform distribuita pe lungimea grinzii			

2.2.2.2 Actiuni termice (SR EN 1991-1-5:2005), temperatura (T_-) și T_+

Valoarea caracteristică a componentei de temperatură uniformă este asociată fie contracției, fie dilatării fiind notată cu $\Delta_{TN, \text{contracție}}$ sau cu $\Delta_{TN, \text{dilatare}}$ pentru fiecare caz în parte.

$$\Delta_{TN, \text{contracție}} = T_0 - T_{e, \text{min}} \quad \text{- temperatura, } T(-)$$

$$\Delta_{TN, \text{dilatare}} = T_{e, \text{max}} - T_0 \quad \text{- temperatura, } T(+)$$

T_0 - reprezintă temperatura inițială, se utilizează valoarea recomandată 10°C (SR EN 1991-1-5:2004/NA:2008).

$T_{e, \text{min}}$ și $T_{e, \text{max}}$ reprezintă componentele de temperatură uniformă minimă și maximă.

$T_{e,min} = -47^{\circ}\text{C}$ si $T_{e,max} = 25^{\circ}\text{C}$ – conf. SR EN 1991-1-5:2004/NA:2008, tabel NA.1(RO).

Gradientul pozitiv de temperatura considerat intre fetele tablierului este $TG+=10.5^{\circ}\text{C}$.

Gradientul negativ de temperatura considerat intre fetele tablierului este $TG-=-8^{\circ}\text{C}$.

2.2.3 Actiuni seismice (S)

Amplasamentul structurii este caracterizat de acceleratia maxima a terenului $a_g=0.20g$ si perioada de colt $T_c=0.7s$.

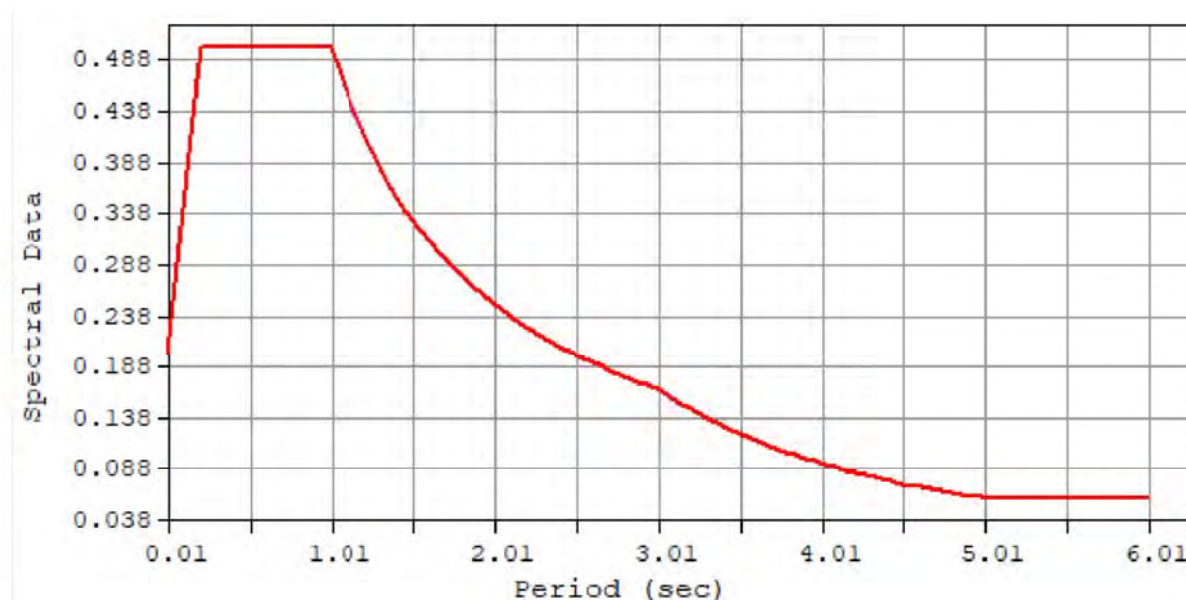


Fig. 3 Spectru de raspuns seismic orizontal

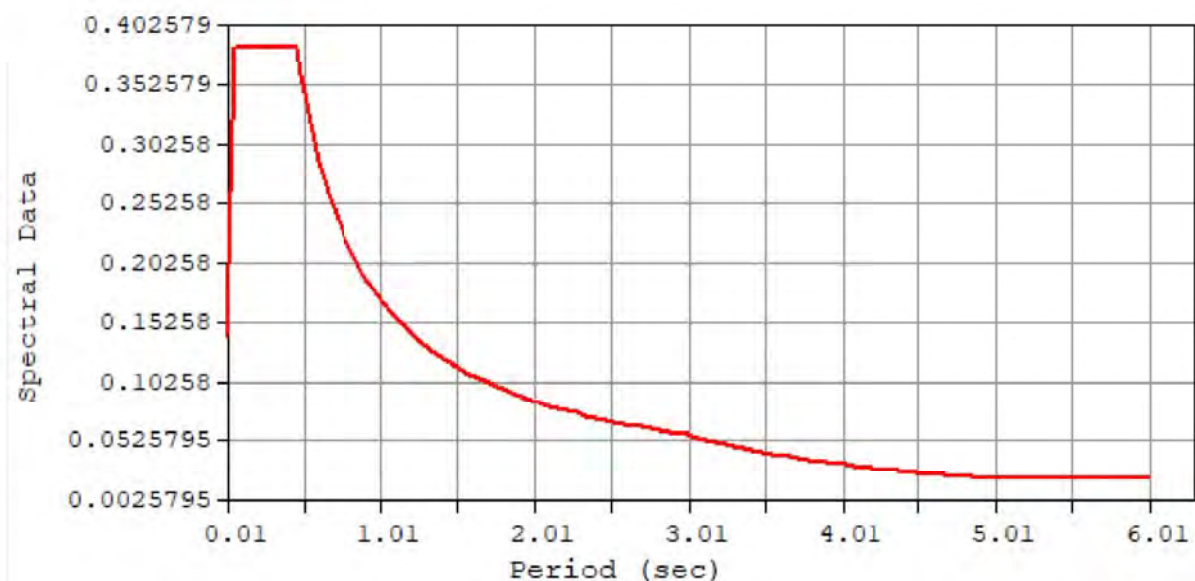


Fig. 4 Spectru de raspuns seismic vertical

3 ANALIZA STRUCTURALA

3.1 Modelul structural

Analiza globala a fost realizata in programul de analiza element finit MIDAS. Suprastructura a fost modelată cu elemente tip bara legate cu link-uri elastice. Culeele se considera incastrate la nivelul radierului. Analiza seismica - calcul modal cu spectre de răspuns.

Culeea a fost calculata intr-un spreadsheet Excel conform relațiilor din statica construcțiilor, fiind considerata ca o consola incastrata la nivelul fundației.

Pilotii au fost dimensionati cu ajutorul programului de calcul specializat Geostru.

3.1.1 Caracteristicile materialelor utilizate:

Tabel 1 Caracteristici materiale

ID	Name	Type	Standard	DB	Elasticity (kN/m ²)	Poisson	Thermal (1/[F])	Density (kN/m ³)
1	C50/60	Concrete	EN04 (RC)	C50/60	3.7277e+007	0.2	1.5560e-008	2.5000e+001
2	C35/45	Concrete	EN04 (RC)	C35/45	3.4077e+007	0.2	1.5560e-008	2.5000e+001
3	C30/37	Concrete	EN04 (RC)	C30/37	3.2836e+007	0.2	3.0864e-006	2.5000e+001
4	Dummy Material	User Defined	None	C35/45	3.4077e+007	0.2	1.5560e-008	0.0000e+000

Geometria structurii

Modelarea suprastructurii, elevatiei a fost realizata cu elemente de tip grinda (frame) pentru modelarea tablierului. (Fig. 5).

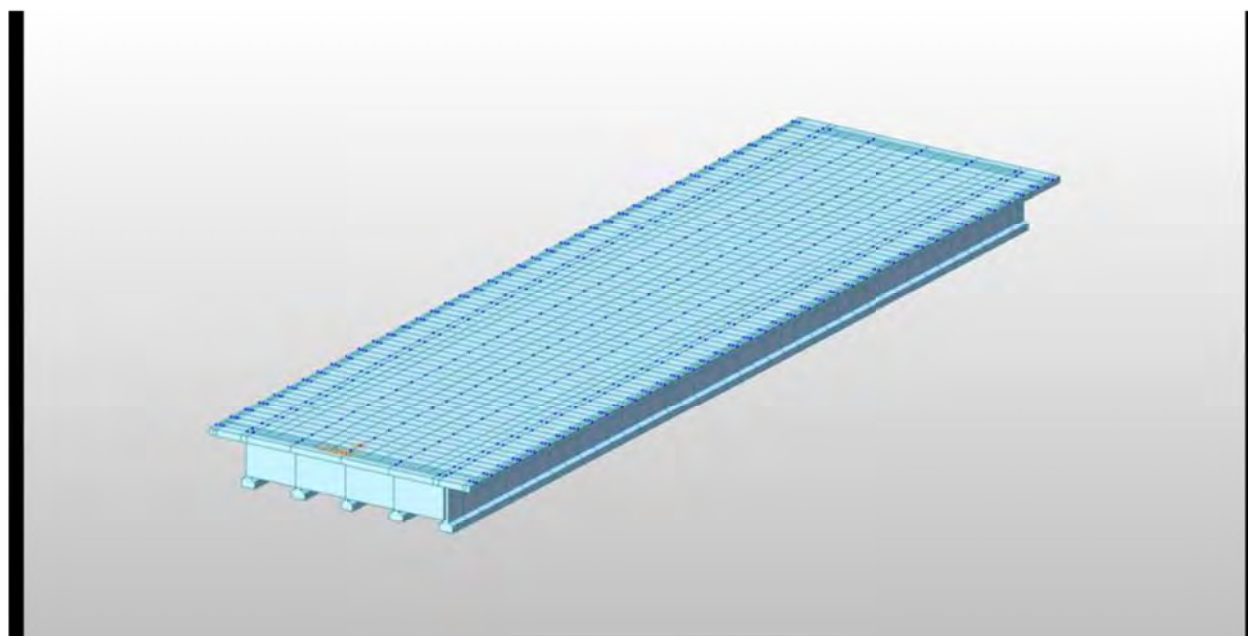


Fig. 5 Modelare structurala 3D

3.1.1.1 Sectiuni utilizate pentru rețeaua de grinzi:

Tabel 2 Sectiuni

ID	Type	Shape	Name	Area (m ²)	Asy (m ²)	Asz (m ²)	Ixx (m ⁴)	Iyy (m ⁴)	Izz (m ⁴)
1	PSC	PSCI	I=2.00	0.6750	0.3118	0.3715	0.0127	0.3479	0.0231
2	DB/User	SB	Antretoaza	0.5250	0.4375	0.4375	0.0140	0.1340	0.0039
3	Composite-CI	CPCI	Compusa	1.3762	0.8884	0.4523	0.0218	0.7628	0.5316
4	DB/User	SB	Rigla	3.5100	2.9250	2.9250	1.3802	0.4943	2.1323
5	DB/User	SB	Elev_pila	10.2000	8.5000	8.5000	6.5871	1.9125	39.3040
6	DB/User	SB	Antretoaza_0	0.0001	0.0001	0.0001	0.0000	0.0000	0.0000
7	PSC	PSCI	Compusa	0.6750	0.0000	0.0000	0.0000	0.3479	0.0231
8	PSC	PSCI	Compusa_1	0.6750	0.0000	0.0000	0.0000	0.3479	0.0231

3.1.2 Combinatii

Tabel 10.

No	Name	Type	Description
1	SLU gr. 1a T(-)	Add	$1.35 \cdot G_p + 1.35 \cdot C_{tp} + 1.35 \cdot LM1 + 1.5 \cdot 0.6 \cdot T(-) + 1.5 \cdot 0.6 \cdot TG(-)$
2	SLU gr. 1a T(+)	Add	$1.35 \cdot G_p + 1.35 \cdot C_{tp} + 1.35 \cdot LM1 + 1.5 \cdot 0.6 \cdot T(+) + 1.5 \cdot 0.6 \cdot TG(+)$
3	SLU T(-) gr. 1a	Add	$1.35 \cdot G_p + 1.35 \cdot C_{tp} + 1.5 \cdot T(-) + 1.35 \cdot LM1(-0.75-0.4) + 1.5 \cdot 0.6 \cdot TG(-)$
4	SLU T(+) gr. 1a	Add	$1.35 \cdot G_p + 1.35 \cdot C_{tp} + 1.5 \cdot T(+) + 1.35 \cdot LM1(-0.75-0.4) + 1.5 \cdot 0.6 \cdot TG(+)$
5	SLU T(-) gr. 2	Add	$1.35 \cdot G_p + 1.35 \cdot C_{pt} + 1.35 \cdot LM1 + 1.5 \cdot 0.6 \cdot T(-) + 1.5 \cdot 0.6 \cdot TG(-)$
6	SLU T(+) gr. 2	Add	$1.35 \cdot G_p + 1.35 \cdot C_{pt} + 1.35 \cdot LM1 + 1.5 \cdot 0.6 \cdot T(+) + 1.5 \cdot 0.6 \cdot TG(+)$
7	SLU Seism X	Add	$1 \cdot G_p + 1 \cdot C_{tp} + 0.2 \cdot LM1 + 0.5 \cdot T(-) + 1.1 \cdot S_x + 0.33 \cdot S_y + 0.33 \cdot S_z + 0.5 \cdot TG(-)$
8	ULS Seism Y	Add	$1 \cdot G_p + 1 \cdot C_{tp} + 0.2 \cdot LM1 + 0.5 \cdot T(-) + 1.1 \cdot S_y + 0.33 \cdot S_x + 0.33 \cdot S_z + 0.5 \cdot TG(-)$
9	Greutate proprie	Add	$1.35 \cdot G_p + 1.35 \cdot C_{tp}$
10	SLU_fara seism	Env	Maxim din toate combinatiile ULS fara seism
11	SLU_cu seism	Env	Maxim din toate combinatiile ULS cu seism

3.2 Centralizarea rezultatelor

3.2.1 Reactiuni

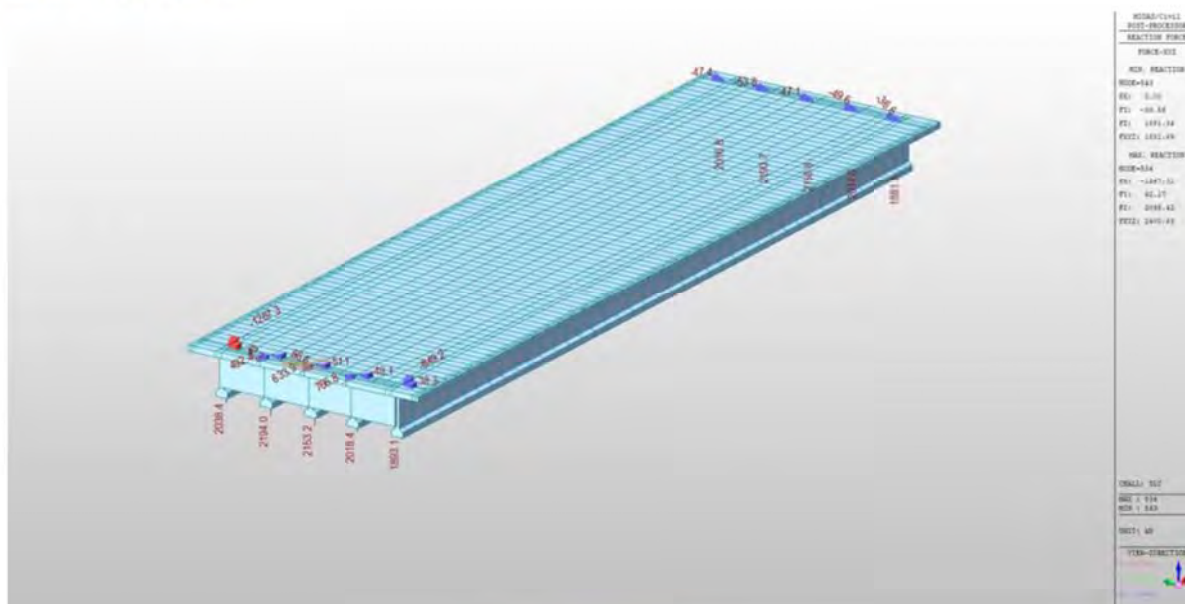


Fig. 6 Reactiuni

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
534	SLU(all)	-1267.311071	42.174301	2038.422123	0.000000	0.000000	0.000000
535	SLU(all)	0.000000	-47.442177	2016.817891	0.000000	0.000000	0.000000
536	SLU(all)	492.437445	56.590560	2193.985626	0.000000	0.000000	0.000000
537	SLU(all)	0.000000	-53.821565	2190.717896	0.000000	0.000000	0.000000
538	SLU(all)	633.914287	51.067178	2153.152612	0.000000	0.000000	0.000000
539	SLU(all)	0.000000	-47.068124	2150.811131	0.000000	0.000000	0.000000
540	SLU(all)	766.777653	49.134495	2018.410912	0.000000	0.000000	0.000000
541	SLU(all)	0.000000	-49.575837	2013.961384	0.000000	0.000000	0.000000
542	SLU(all)	-849.151983	38.301704	1893.078517	0.000000	0.000000	0.000000
543	SLU(all)	0.000000	-36.557900	1881.337333	0.000000	0.000000	0.000000
SUMMATION OF REACTION FORCES PRINTOUT							

3.2.2 Capacitate portanta piloti pile si culei

Breviarul pentru calculul capacitatii portante a pilotilor este realiza in programul specializat GeoStru.

NORME DE REFERINTA

Pentru fazele de calcul se face referire la normele:

Eurocodul 7: Proiectare geotehnica – Partea 1: Reguli generale.

Eurocodul 8: Indicatii de proiect pentru rezistenta seismică a structurilor - Partea 5: Fundatii, structuri de sprijin, si specte geotehnice.

Sarcina Limita Verticala

Sarcina limită verticală a fost calculată cu formule statistice, care o exprimă în functie de geometria coloanei consolidate, caracteristicile terenului si interfata pilot-teren. Astfel, întrucât realizarea unui pilot, bătut sau forat, modifică întotdeauna caracteristicile terenului în jurul acestuia, se propune considerarea unui unghi de rezistență la tăiere egal cu:

$$\begin{aligned}\phi' &= \frac{3}{4}\phi + 10^\circ \text{ pentru piloti bătuti} \\ \phi' &= \phi - 3^\circ \text{ pentru piloti forati}\end{aligned}$$

unde ϕ este unghiul de rezistență la tăiere înainte de executie. În continuare vom indica cu ϕ parametrul de rezistență ales.

La sfârșitul calculului, sarcina limită Q_{lim} este divizată în mod conventional în două cote, rezistenta la vârf Q_p si rezistenta laterală Q_l .

Resistenta Unitara La Varf

Formula lui Terzaghi

Solutia propusă de Terzaghi consideră că terenul existent deasupra adâncimii la care a ajuns vârful pilotului poate fi înlocuit de o suprasarcină echivalentă cu tensiunea verticală eficac (neglijând faptul că interacțiunea dintre pilot si fundatie ar putea modifica această valoare) si conduce analiza la problema capacității portante a unei fundatii superficiale.

Formula lui Terzaghi poate fi scrisă:

$$Q_p = c \times N_c \times s_c + \gamma \times L \times N_q + 0.5 \times \gamma \times D \times N_\gamma \times s_\gamma$$

unde:

$$N_q = \frac{a^2}{2 \cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2) \tan \phi}$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = \frac{\tan \phi}{2} \left(\frac{K_{py}}{\cos^2 \phi} - 1 \right)$$

Metoda lui Berezantzev

În principiu *Berezantzev* face referire la o suprafață de alunecare "alla Terzaghi" care se opreste pe planul de sprijin (vârful pilotului); totuși acesta consideră că cilindrul de teren coaxial pilotului are diametrul egal cu extensia în secțiune a suprafeței de alunecare, este în parte "sustinut" prin acțiunea tangentială de către terenul rămas de-a lungul suprafeței laterale. Acesta dă o valoare a presiunii la baza inferioară a lui γD , și mai mică cu cât acest efect de "siloz" este marcant, adică cu cât mai mare este raportul D/B ; de acesta ține cont coeficientul N_q , care este deci funcție descrescătoare a lui D/B .

Rezistența unitară Q_p la vârf, pentru cazul terenului cu forfecarea (ϕ) și coeziunea (c), este dată de expresia:

$$Q_p = c \times N_c + \gamma \times L \times N_q$$

Indicând cu:

γ greutatea unității volumice a terenului;
 L lungimea coloanei;
 N_c și N_q sunt factorii capacității portante afectați de efectul formă (circulară);

Metoda lui Vesic

Vesic a asimilat problema rupturii în jurul vârfului pilotului și aceea a expansiunii unei cavități cilindrice în mediu elasto-plastic, în așa fel încât să se țină cont și de compresibilitatea mediului.

După *Vesic* coeficienții capacității portante N_q și N_c se pot calcula după cum urmează:

$$N_q = \frac{3}{3 - \sin \phi} \left\{ \exp \left[\left(\frac{\pi}{2} - \phi \right) \tan \phi \right] \tan^2 \left(45 + \frac{\phi}{2} \right) I_{rr}^{(4 \sin \phi) / [3(1 + \sin \phi)]} \right\}$$

Indicele de rigiditate redus I_{rr} în expresia precedentă este calculat plecând de la deformarea volumică ε_v .

Indicele de rigiditate I_r se calculează utilizând modulul de elasticitate elastică tangentială G' și rezistența la tăiere a terenului s .

Când avem de-a face cu condiții nedrenate sau solul se găsește într-o stare de densă, termenul ε_v poate fi considerat egal cu zero și se obține $I_{rr} = I_r$.

Este posibilă estimarea lui I_r cu următoarele valori:

TEREN	I_r
Nisip	75-150
Praf	50-75
Argilă	150-250

Termenul N_c al capacității portante este calculat:

$$N_c = (N_q - 1) \cot \phi \quad (a)$$

Când $\phi = 0$ (condiții nedrenate)

$$N_c = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

Metoda lui Janbu

Janbu calculează N_q (cu unghiul ψ) după cum urmează:

$$N_q = \left(\tan \phi + \sqrt{1 + \tan^2 \phi} \right)^2 \exp(2\psi \tan \phi)$$

N_c se poate calcula de la (a) când $\phi > 0$.

Per $\phi = 0$ se foloseste $N_c = 5.74$

Formula lui Hansen

Formula lui Hansen este valabilă pentru orice raport D/B , deci pentru fundatii de suprafată, dar si pentru cel profunde, acelasi autor a introdus coeficienti pentru o mai bună interpretare a comportamentului real al fundatiei, fără acestia, sarcina limită ar fi prea mult mărită odată cu adâncimea.

Pentru valori $L/D > 1$:

$$d_c = 1 + 0.4 \tan^{-1} \frac{L}{D}$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 \tan^{-1} \frac{L}{D}$$

În cazul $\phi = 0$

D/B	0	1	1.1	2	5	10	20	100
d'_c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

În factorii următori, exeresiile cu acest semn (') sunt egale cu $\phi = 0$.

Factor de formă:

$$s'_c = 0.2 \frac{D}{L}$$

$$s_c = 1 + \frac{N_q}{N_c} \frac{D}{L}$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{D}{L}$$

Factor de adâncime:

$$d'_c = 0.4k$$

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi) k$$

$$d_\gamma = 1 \text{ per qualsiasi } k$$

$$k = \tan^{-1} \frac{L}{D} \text{ se } \frac{L}{D} > 1$$

Rezistenta Trunchiului

Metoda utilizată pentru calcularea capacității portante laterale este metoda α , propusă de Tomlinson (1971); rezistenta laterală este calculată după cum urmează:

$$Q_1 = (\alpha c + \sigma K \tan \delta) \cdot A_l \cdot f_w$$

A_l = suprafata laterală a pilotului;

f_w = factor de corectie legat de conicitatea trunchiului pilotului, sau reducerea diametrului pilotului;

c = valoarea medie a coeziunii (sau a rezistentei la tăiere în conditii nedrenate);

σ = presiunea verticală eficace a terenului;

K = coeficientul de alunecare orizontală, depinde de tehnologia de executie a pilotului si de starea precedentă de densitate, care se calculează după cum urmează:

Pentru piloti bătuti

$$K = 1 - \tan^2 \phi$$

Sau, în cazul specific, este posibilă desemnarea valorilor propuse în tabel:

Pilot	K	
	Teren afânat	Teren dens
Otel	0.5	1
Beton pref.	1	2
Lemn	1	3

Pentru piloti forati:

$$K = 1 - \sin \phi$$

δ = forfecarea pilot-teren în functie de asprimea suprafetei pilotului;

Pentru piloti bătuti

$$\delta = 3/4 \tan \phi$$

Pentru piloti forati

$$\delta = \tan \phi$$

α = coeficientul de adeziune găsit astfel:

Piloti forati:

$$\alpha = \frac{100 + c^2}{100 + 7c^2}$$

Caquot – Kerisel

$$\alpha = 1 - 0.1 \cdot c \quad \text{pentru } c < 5 \text{ t/m}^2$$

$$\alpha = 0.525 - 0.005 \cdot c \quad \text{pentru } c \geq 5 \text{ t/m}^2$$

Meyerhof – Murdock (1963)

$$\alpha = 0.9 \text{ pentru } c < 2.5 \text{ t/m}^2$$

$$\alpha = 0.8 \text{ pentru } 2.5 \leq c < 5 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 5 \leq c \leq 7.5 \text{ t/m}^2$$

$$\alpha = 0.9 \text{ pentru } c > 7.5 \text{ t/m}^2$$

Whitaker – Cooke (1966)

$$\alpha = 0.9 \text{ pentru } c < 4 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 4 \leq c < 8 \text{ t/m}^2$$

$$\alpha = 0.5 \text{ pentru } 8 \leq c < 12 \text{ t/m}^2$$

$$\alpha = 0.4 \text{ pentru } 12 \leq c \leq 20 \text{ t/m}^2$$

$$\alpha = 0.20 \text{ pentru } c > 20 \text{ t/m}^2$$

Woodward (1961)

Piloti bătuti

Coeficientul α pentru pilot bătut

$2.5 \leq c < 5 \text{ t/m}^2$	$\alpha = 1.00$
$5 \leq c < 10$	$\alpha = 0.70$
$10 \leq c < 15$	$\alpha = 0.50$
$15 \leq c < 20$	$\alpha = 0.40$
$c \geq 20$	$\alpha = 0.30$

Forfecarea Negativa

Când un pilot este bătut sau trece printr-un strat de material compresibil înainte de a se fi terminat procesul de consolidare, terenul se va misca în raport cu pilotul făcând să apară eforturi de forfecare între pilot și teren care duc la așa-zisul fenomen de forfecare negativă. Efectul forfecării negative este acela de a crește sarcina axială asupra pilotului, având ca și consecință creșterea tasărilor. Forța care ia naștere ca rezultat al forfecării negative este estimată ca fiind egală cu componenta activă a rezistenței laterale (v. Rezistența Trunchiului) de-a lungul suprafeței laterale la contactul cu stratul în care se generează astfel de fenomen, dar din partea opusă forfecării negative. Rezultatul astfel determinat nu este dedusă din sarcina limită, ci din aceea de exercițiu.

Factorul De Corectie În Condiții Seismice

Criteriul lui Vesic

Vesic apreciază că pentru a ține cont de fenomenul de creștere în calcul a capacității portante este suficientă reducerea cu 2° a unghiului de forfecare a straturilor de fundație. Limita acestei sugestii este faptul că nu ține cont de intensitatea solicitării seismice (exprimată prin parametrul accelerației seismice orizontale maxime). Acest criteriu pare să găsească confirmarea în observațiile făcute în prezenta diverselor fenomene seismice.

Criteriul lui Sano

Autorul propune reducerea unghiului de forfecare a straturilor portante cu o cantitate dată în ecuația:

$$D_p = \arctg\left(\frac{A_{\max}}{\sqrt{2}}\right)$$

unde A_{\max} reprezintă accelerația seismică orizontală maximă.

Acest criteriu, față de cel al lui Vesic, reprezintă avantajul de a lua în considerare și intensitatea solicitării seismice. Experiența demonstrează totuși că aplicarea fără apelul la critică a acestei realități poate conduce la valori prea mici ale lui Q_{lim} .

Corecțiile lui Sano și Vesic se aplică exclusiv pentru terenuri necoezive dense. Este greșită aplicarea lor la terenurile afânate sau cu densitate medie, unde vibrațiile seismice produc fenomenul opus celui de dilatare, cu creșterea gradului de densitate și a unghiului de forfecare.

TASARI

Tasarea verticală a fost calculată cu metoda lui Davis-Poulos, în funcție de care pilotul este considerat rigid (nedeformabil) înecat într-un mediu elastic, semispațiu sau strat de grosime finită.

Se ipotizează că interacțiunea pilot-teren este constantă la fâșii de-a lungul a n suprafețe cilindrice în care se împarte suprafața laterală a pilotului.

Tasarea suprafeței generice i ca efect la sarcina limită transmisă de la pilot la teren de-a lungul suprafeței cu nr. j poate fi exprimat:

$$W_{i,j} = (\tau_j / E) \times B \times I_{i,j}$$

Unde:

τ_j = Creșterea tensiunii în funcție de punctul mediu al fâșiei;

E = Modulul elastic al terenului;

B = Diametrul pilotului;

$I_{i,j}$ = Coeficientul de influență.

Tasarea compresivă se obține însumând $W_{i,j}$ cu toate ariile j .

SARCINA LIMITĂ ORIZONTALĂ

Sarcina limită orizontală a fost calculată după teoria dezvoltată de Broms, care presupune că comportamentul interfeței pilot-teren este rigidă perfect elastică, și astfel rezistența terenului să fluctueze cu orice valoare, nu nulă, a deplasării și să rămână constantă la creșterea deplasării.

Se presupune că comportamentul flexionar al pilotului este de tip rigid, perfect elastic, trebuie să spunem că rotațiile elastice ale pilotului sunt neglijabile până când momentul încovoietor ajunge la valoarea M_y de plasticizare.

Pentru terenuri coesive Broms propune adoptarea unei reacții a terenului constantă cu adâncimea egală cu:

$$p = 9 \times c_u \times B$$

cu reacția nulă până la adâncimea de 1.5 d; indicând cu:

c_u = Coeziunea nedrenată,

B = Diametrul pilotului;

p = Reacția terenului pentru unitatea de lungime a pilotului.

Pentru terenurile necoesive se presupune că rezistența variază liniar în funcție de adâncime după formula:

$$p = 3K_p \gamma zB$$

indicând cu:

p = Reacția terenului pentru unitatea de lungime a pilotului;

K_p = Coeficientul de împingere pasivă;

γ = Greutatea volumică a terenului;

z = Adâncimea;

B = Diametrul pilotului

PILOT ÎN CONDITII DE EXERCITIU

Analiza pilotului în condiții de exercițiu: **Metoda elementelor finite.**

Metoda elementelor finite modelează pilotul de fundație supus la sarcini transversale, în mod realist întrucât face uz, fie de împingeri cât și de rotații în noduri pentru definirea liniei elastice a pilotului, deci reprezintă metoda cea mai rațională și eficientă disponibilă actualmente pentru analizarea acestor tipuri de structuri.

În ceea ce urmează se face apel la teoria metodelor, indicând cu \mathbf{P} matricea forțelor nodale externe, cu \mathbf{F} aceea a forțelor interne și cu \mathbf{A} matricea coeficienților de influență care, pentru echilibrul între forțele externe și interne, leagă primele două în funcție de formula:

$$\mathbf{P} = \mathbf{A}\mathbf{F}$$

Deplasările interne \mathbf{e} (împingerile și rotațiile) ale elementului noduli generic sunt legate de deplasările externe \mathbf{X} (împingerile și rotațiile) aplicate la noduri, ale următoarei formule:

$$\mathbf{e} = \mathbf{B}\mathbf{X}$$

unde matricea \mathbf{B} s-a demonstrat a fi transpunerea matricei \mathbf{A} .

De altă parte, forțele interne \mathbf{F} sunt legate de deplasările interne \mathbf{e} ale următoarei expresii:

$$\mathbf{F} = \mathbf{S}\mathbf{e}$$

Aplicând substituirile de rigoare se obține:

$$\mathbf{F} = \mathbf{S}\mathbf{A}^T\mathbf{X}$$

Si deci:

$$P = AF = A SA^T X$$

Deci, calculând inversa matricei $A SA^T$ se obține expresia deplasărilor externe X :

$$X = (A SA^T)^{-1} P$$

Notati, deci, că cu ajutorul deplasărilor X este posibilă aflarea forțelor interne F necesare pentru proiectul structurii.

Matricea $A SA^T$ este cunoscută ca și matrice de rigiditate globală deoarece caracterizează legătura dintre deplasări și forțele externe nodale.

Metoda elementelor finite are, printre altele, avantajul că dă posibilitatea de a lua în considerare, ca și condiții la contur, rotații și deplasări cunoscute.

Rotatiile nodale care schematizează terenul sunt considerate forțe globale legate de modulul de reacție și de zona de influență a nodului. În cadrul soluției elementelor finite pentru piloti supuși la sarcini transversale, modulul de reacție este considerat în forma:

$$k_s = A_s + B_s Z^n$$

sau, nevrând să creștem nelimitat k_s cu adâncimea, în forma:

$$k_s = A_s + B_s \tan^{-1}(Z/B)$$

în care Z reprezintă adâncimea iar B reprezintă diametrul pilotului.

Valorile lui A_s și $B_s Z^n$ sunt obținute din expresia capacității portante (Bowles) cu factori corectori s_i , d_i , și i_i egali cu 1:

$$k_s = q_{ult}/\Delta H = C(cN_c + 0.5\gamma B N_\gamma)$$

$$B_s Z^n = C(\gamma N_q Z^1)$$

Unde $C = 40$ este obținut în corespondență cu o tasare maximă de 25 mm.

Date generale...

Diametru varf	1.20 m
Lungime	16.00 m
Tip	Batut
Capacitatea de varf calculată cu:	Terzaghi
Tip beton	1
Tip otel	1

SEISM

Acceleratie seismica	0.2
Coefficient de intensitate seismica [Kh]	0.1
Coefficient de intensitate seismica [Kv]	0.07

Arhiva materiale

Beton

Nr.	Clasa Beton	f _{ck} ,cubic [Kg/cm ²]	E _c [Kg/cm ²]	f _{ck} [Kg/cm ²]	f _{cd} [Kg/cm ²]	f _{ctd} [Kg/cm ²]	f _{ctm} [Kg/cm ²]
1	C20/25	250	299600	200	113.3	10.1	22.1
2	C25/30	300	314750	250	141.6	11.4	25.6
3	C28/35	350	323080	280	158.6	12.6	27.6
4	C40/50	500	352200	400	226.6	16.3	35

Oteluri:

Varianta de ocolire Timisoare Sud - Pod km 20+875 peste canal de irigatii

Nr.	Steel Class	Es [Kg/cm2]	fyk [Kg/cm2]	fyd [Kg/cm2]	ftk [Kg/cm2]	ftd [Kg/cm2]	ep_tk	epd_ult	B1*B2 in.	B1*B2 fm.
1	B450C	2000000	4500	3913	4500	3913	.075	.0675	1	0.5
2	B450C*	2000000	4500	3913	5400	4500	.075	.0675	1	0.5
3	B450C**	2000000	4500	3913	4582	3985	.012	.01	1	0.5
4	S235H	2141370	2447.28	2128.11	3670.92	2128.11	0.012	0.01	1	0.5
5	S275H	2141370	2855.16	2482.97	4384.71	2482.97	0.012	0.01	1	0.5
6	S355H	2141370	3670.92	3191.66	5200.47	3670.92	0.012	0.01	1	0.5

Stratigrafie

Nr.: Numarul stratului Hs: Grosimea stratului Fi: Unghi de frecare c: Coeziune Alfa: Coef. aderenta frecari laterale, Vs: Viteza unde de forfecare.

Strat. 1

Nr.	Hs	Greutate volumica [kg/m³]	Greutate volumica saturata [kg/m³]	c [kg/cm²]	Fi (°)	Frecare negativa	Alfa	Modul elasticitat e [kg/cm²]	Vs [m/s]	Descriere litologica
1	4.00	2100.00	0.00	0.06	25.00	Nu	1.00	60.00	0	
2	11.00	1850.00	0.00	0.00	32.00	Nu	0.00	300.00	0	
3	0.30	1800.00	0.00	0.00	28.00	Nu	0.00	100.00	0	
4	5.30	2100.00	0.00	0.06	25.00	Nu	0.00	60.00	0	
5	3.00	1900.00	0.00	0.00	36.00	Nu	0.00	500.00	0	
6	1.00	1900.00	0.00	0.00	36.00	Nu	0.00	700.00	0	nisip usor prafos

Sacina limita

Stratigrafie	Nq	Nc	Fi/C strat varf pilot (°)/[kg/cm²]	Greutate pilot [kg]	Sarcina limita varf [kg]	Sarcina limita laterala [kg]	Sarcina limita [kg]	Frecare negativa [kg]	Sarcina limita orizontala [kg]
A1+M1+R3	16.35	29.81	27.25/0.06	45238.94	589059.60	591759.50	1135580.00	--	--

REZISTENTA DE PROIECTARE SARCINI AXIALE

Rezistenta caracteristica sarcini axiale. Nume combinatie: A1+M1+R3			
=====			
Numar de verticale inspectate	1		
Factor de corelatie pentru verticale inspectate mediu (xi3)	1.00		
Factor de corelatie pentru vertical inspectate minim (xi4)	1.00		
	Rc, Min [kg]	Rc, Media [kg]	Rc, Max [kg]
Baza	589059.60	589059.60	589059.60
Lateral	591759.50	591759.50	591759.50
Total	1135580.00	1135580.00	1135580.00
Coeficient partial rezistenta caracteristica	R3		
Baza	2.50		
Lateral	2.50		
Rezistenta de proiectare baza	235623.80 kg		
Rezistenta de proiectare laterala	236703.80 kg		
Rezistenta de proiectare	427088.70 kg		

3.2.3 Eforturi in grinzi

Grinzile care intra in alcatuirea tablierului si ale caror caracteristici geometrice au fost descrise mai sus, au fost calculate tinandu-se seama de etapele de executie si de influenta fenomenelor reologice (contractie si curgere lenta).

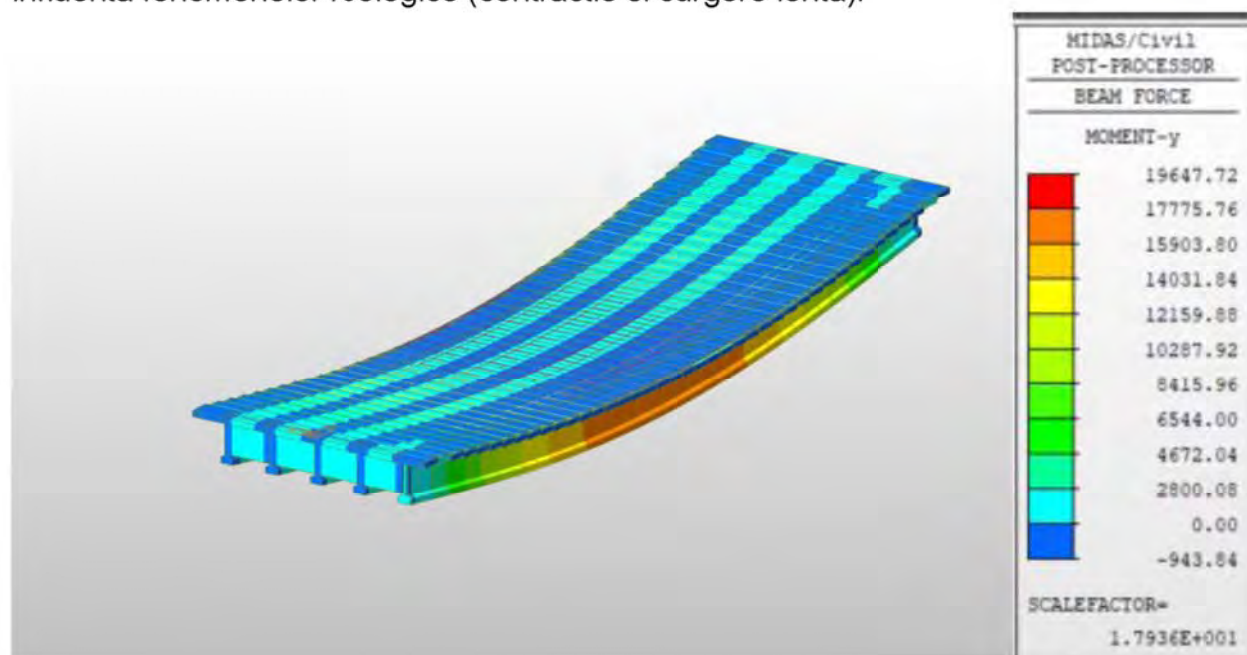


Fig.7 Diagrama moment incovoietor grinzi

Etapa 1 – Montarea grinzilor

Etapa 2 – Turnarea placii de suprabetonare

Etapa 3 – Executia caii, montarea bordurilor, parapetilor.

Întocmit: ing. Takacs Attila

Pod km 20+865 culee2

Verificarea capacității portante a piloților (R_{max}):

Valoare de calcul a capacității portante ultime la compresiune a piloților:

$$R_{c,d} = R_{b,d} + R_{s,d} = R_{b,k} / \gamma_{b2} + R_{s,k} / \gamma_{s2} = A_b \cdot q_{b,k} / \gamma_{b2} + U_p \cdot \sum q_{s,ki} \cdot l_i / \gamma_{s2}$$

Conform studiului geotehnic și normativului NP 123 valoarea caracteristică a rezistenței de bază a pilotului este:

$A_{pilot} = 1,13 \text{ m}^2$ – aria unui pilot în secțiune transversală

$h_{bază} = 15,5 \text{ m}$

$l_c = 1,00$

q_{bk} -valoarea caracteristică a presiunii de bază se determină conf. tab.5 NP 123

$q_{bk} = 4000 \text{ kN/m}^2$ – presiunea caracteristică la bază.

$R_{b,k} = A_{pilot} \cdot q_{bk}$

$R_{b,k} = 4520 \text{ kN}$

$R_{b,d} = R_{b,k} / \gamma_b$

$R_{b,d} = 3476.9 \text{ kN}$

Valoarea caracteristică a rezistenței de frecare pe suprafața laterală a pilotului, conform straturilor din studiul geotehnic este:

$R_{s,k} = \sum (q_{s,ki} \cdot l_i) \cdot U_p$

$R_{s,k} = 2933.1 \text{ kN}$

$q_{s,ki}$ - se obțin ca valori intermediare prin interpolare liniară din tab. 6 NP 123

$U_{pilot} = 3,77 \text{ m}$ – perimetrul secțiunii transversale a pilotului

$\gamma_s = 1.9$

$R_{s,d} = R_{s,k} / \gamma_s$

$R_{s,d} = 1543.7 \text{ kN}$

Rezultă, valoarea de calcul a capacității portante ultime la compresiune a pilotului:

$$R_{c,d} = R_{b,d} + R_{s,d} = 1695 + 1399 = 3094 \text{ kN}$$

$R_{c,d} = 5020.6 \text{ kN}$

Greutatea totală de preluat din structura până la nivelul radierului

$V_r = 11500 \text{ kN}$

Greutatea repartizată pe un pilot (4 piloți)

$V_{pr} = 2875 \text{ kN}$

Greutatea proprie pilot

$G_p = 452.16 \text{ kN}$

Greutate totală de preluat pe pilot

$V_p = 3327.2 \text{ kN}$

$V_p < R_{c,d}$ se verifica



intocmit ing. Attila Takacs



POD KM 24+725 PESTE CANALUL BEGA

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1 DESCRIERE SI BAZA DE PROIECTARE

1.1 Descrierea structurii

Varianta de ocolire Timisoara Sud traverseaza canalul Bega la km 24+725 motiv pentru care este necesara construirea unui pod, care in plan se afla in aliniament si intersecteaza normal cursul canalului Bega.

Suprastructura podului este diferita pe cele 5 deschideri si anume: in deschiderile marginale care au lungimea de 21,80m, suprastructura este alcatuita din 9 grinzi prefabricate precomprimate cu armatura preintinsa, cu inaltimea de 1,03m, solidarizate la partea superioara prin intermediul unei placi monolite din beton armat cu grosimea minima de 15cm. In deschiderile centrale, care au lungimile de 36,45+36,50+36,45m, suprastructura este alcatuita din cate 5 grinzi prefabricate precomprimate pe deschidere, avand lungimea de 40m si inaltimea de 2m, solidarizate la partea superioara cu o placa de suprabetonare monolite, din beton armat

Pe deschiderile centrale, pentru solidarizarea transversala a grinzilor se vor executa o antretoaza de camp si antretoaze de capat avand si rol de sprijin pentru liftarea tablierului in viitor in vederea inlocuirii aparatelor de reazem din neopren.

Placa de suprabetonare se va executa din beton armat, turnat peste predale prefabricate din beton armat care au rol de cofraj pierdut.

Peste placa de suprabetonare se va aterne hidroizolatie avand 1 cm grosime, protectia hidroizolatiei in grosime de 3 cm din BA8 si straturile rutiere in grosime de 2x4 cm din BAP 16 respectiv MAS 16 conform Proiect Ilustrativ.

Profilul transversal al podului pe zona carosabilului este "in acoperis" cu pante transversale de 2.5% dinspre ax spre trotuare, iar pe zona trotuarelor la nivel panta transversala a acestora este de 1% inspre axul structurii.

In sectiune transversala suprastructura prezinta urmatoarele caracteristici: 7.80 m parte carosabila, incadrata cu parapete New Jersey 0.60m, doua trotuare la nivel de 1.00 m latime utila fiecare, si longrine cu latimea de 0.30 m, latimea totala a suprastructurii fiind de 11.60 m. Pentru scurgerea apelor s-au dispus pe suprastructura guri de scurgere cu sistem de colectare in lungul structurii (tuburi PVC) si deversare prin burlane in dreptul infrastructurilor unde vor fi epurate.

Conform proiectului ilustrativ, la marginea partii carosabile au fost prevazute parapete de siguranta de tip rigid (parapete New Jersey din beton armat cu grad de asigurare H4b) pentru protectia pietonilor iar pe partile exterioare ale trotuarelor se vor prevedea parapete pietonale metalice.

Rosturile de dilatare se vor amplasa la culei si pe pilele P1 si P4 rezultand faptul ca deschiderile avand in componenta grinzi de lungime $L=36$ m vor fi continuate la nivelul placii de suprabetonare (in dreptul pilelor P1 si P4).

Pasajul va avea o lungimea totala (incluzand si zidurile intoarse de la culei) $L_{tot}=165,80$ m.

Infrastructurile sunt prevazute a fi executate din beton armat, fundate indirect, pe piloti forati de diametru mare $d=1200$ mm, avand capetele solidarizate la partea superioara cu radiere din beton armat.

Culeele vor avea elevatie "plina" (de tip perete din beton armat).

Pilele vor avea elevatii "pline" (de tip perete din beton armat) cu evazaj sub forma de arc de cerc la partea superioara unde se va executa rigla din beton armat. **Racordarea cu terasamentele** se va realiza cu sferturi de con pereate cu beton iar pentru drenarea apelor de infiltratie din spatele elevatiilor culeelor, se vor executa drenuri din piatra bruta invelite in geotextil. Sub partea carosabila pentru tranzitia de la tablierul podului la rambleul drumului se vor executa placi de racordare din beton armat cu lungimea de 6.00 m. Pe taluzele sferturilor de con se vor prevedea casiuri pentru scurgerea apelor si scari de acces din beton.

Urmatoarele faze de executie sunt considerate:

- Executarea piloților și a radierelor;
- Executarea elevatiilor pentru culei si pile;
- Poziționarea grinzilor;
- Pozitionarea predalelor, armarea placii de suprabetonare si a antretoazelor;
- Betonarea placii de suprabetonare si a antretoazelor;
- Executarea terasamentelor;
- Executarea caii, parapetilor, lisei si a bordurilor.

Clasa de incarcare: Eurocode: LM1

Clasa de importanta: "C"

Cerinte de calitate: A4, B2, D2.

Proiectare seismica: $a_g = 0.20g$, $T=0.7s$ conform P100/2013

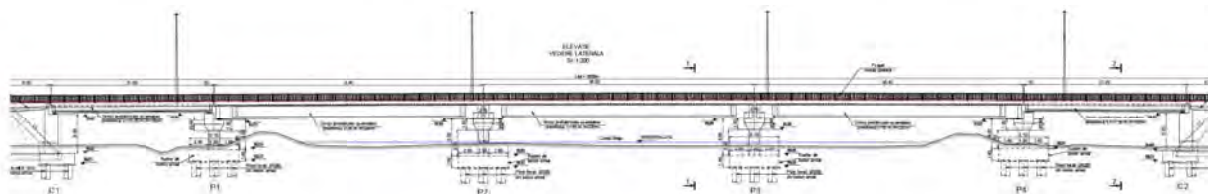


Fig. 1 Elevatie



2 ACTIUNI

2.1 Actiuni considerate

In concordanta cu Eurocode 1, actiunile considerate in proiectarea structurii sunt:

- Actiuni permanente (G):

- Greutatea proprie a grinzilor, placilor, elevatiei, radielor si a pilotilor;
- Greutatea caii, parapetelor, trotuar;
- Impingerea pamantului;
- Impingerea pamantului din seism;

- Actiuni variabile (Q):

- Actiuni din trafic la poduri (SR EN 1991-2:2005) (convoiul LM1, încărcări din oameni pe trotuare, franarea si forta centrifuga);

- Actiuni termice, temperatura (T-) si T(+).

- Actiuni accidentale (A):

- Actiuni seismice, Seism X (longitudinal), Seism Y (transversal) , Seism Z (vertical).

2.2 Valori caracteristice

2.2.1 Actiuni permanente (G).

2.2.1.1 Greutatea proprie

Element	buc	Arie m ²	Densitate kN/m ³	Gn kN/m	γ	Gc kN/m
Grinzi H=2.00m	4	0.673	25	67.30	1.35	90.86
Placa grinzi marginale	2	0.68	25	34.00	1.35	45.90
Placa grinzi centrale	2	0.765	25	38.25	1.35	51.64
Cale	1	1.3464	24	32.31	1.35	43.62
Lisa prefabricata	2	0.043	25	2.15	1.35	2.90
Parapet directiona	2	-	-	11.00	1.35	14.85
Parapet pietonal	2	-	-	1.50	1.35	2.03
Grinda parapet	2	0.042	25	6.50	1.35	8.78
TOTAL / ml				193.01		260.57

Element	buc	Arie m ²	Densitate kN/m ³	Gn kN/m	γ	Gc kN/m
Grinzi H=1.05m	9	0.4	25	90.00	1.35	121.50
Placa grinzi marginale	2	0.23	25	11.50	1.35	15.53
Placa grinzi centrale	7	0.337	25	58.98	1.35	79.62
Cale	1	1.3464	24	32.31	1.35	43.62
Lisa prefabricata	2	0.043	25	2.15	1.35	2.90
Parapet directional	2	-	-	11.00	1.35	14.85
Parapet pietonal	2	-	-	1.50	1.35	2.03
Grinda parapet	2	0.042	25	6.50	1.35	8.78
TOTAL / ml (L=21.00m)				213.94		288.82

2.2.1.2 Impingerea pamantului

$h =$	5.40	m	Inaltime elevatie
$H_{zid} =$	1.50	m	inaltime zid de garda
$H_T =$	6.90	m	Inaltime totala
$\phi_{df} = \phi_{kf} =$	33	grade	
$g_{kd} = g_{kf} =$	19.00	kN/m ³	
$K_{ad} = \tan(\pi/4 - \phi_{df}/2) =$	0.333	coef. presiune activa	

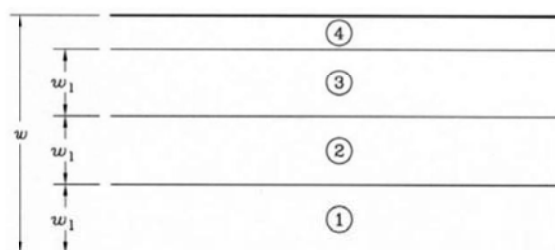
2.2.2 Actiuni variabile (Q)

2.2.2.1 Acțiuni din trafic la poduri (SR EN 1991-2:2005) (LM1);

Pentru determinarea efectelor zonelor de incarcarea a partii carosabile a unui pod aceasta este divizata in benzi teoretice.

Lățimea părții carosabile w	Numărul benzilor teoretice de circulație	Lățimea unei benzi teoretice de circulație	Lățimea zonei rămase
$w < 5,4 \text{ m}$	$n_l = 1$	3 m	$w - 3 \text{ m}$
$5,4 \text{ m} \leq w < 6 \text{ m}$	$n_l = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = \text{Int}\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_l$
NOTĂ – De exemplu, pentru o parte carosabilă cu lățimea de 11 m, $n_1 = \text{Int}\left(\frac{w}{3}\right) = 3$, și lățimea zonei rămase este: $11 - 3 \times 3 = 2 \text{ m}$.			

Numărul, poziția și numerotarea benzilor teoretice încărcate, se realizează astfel incat efectele convoaielor de calcul să fie cele mai defavorabile in scopul unei verificări individuale.



w =Lățimea părții carosabile

w_1 =Lățimea unei benzi teoretice de circulație

1=Banda teoretică nr.1

2=Banda teoretică nr.2

3=Banda teoretică nr.3

4=Zona rămasă

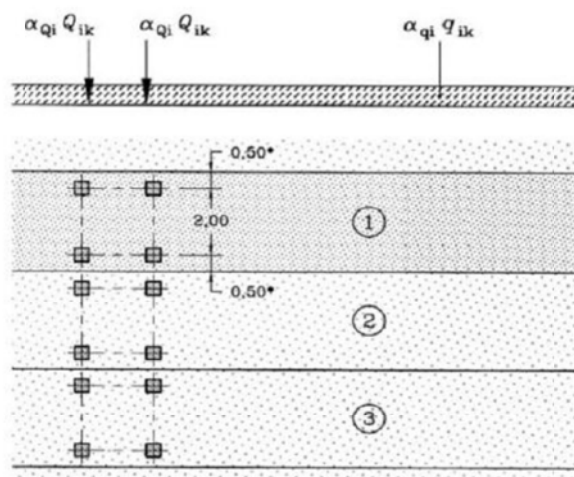
2.2.2.1.1 Convoiul de calcul 1 (LM1) :

Acest convoi este alcătuit din încărcări concentrate și încărcări uniform distribuite ce acoperă cele mai multe efecte din traficul alcătuit din camioane și automobile, fiind utilizat atât pentru verificări locale cât și globale.

Încărcările concentrate sunt reprezentate de sistemul tandem TS alcătuit din 2 osii cu greutatea " $\alpha_Q Q_k$ " pe fiecare osie.

Notă: atât pentru verificări locale cât și generale se va aplica un singur sistem tandem pe fiecare bandă de circulație cu precizarea că poziționarea acestuia se va face centrat pe axul fiecărei benzi, în cazul verificărilor globale, și în poziția cea mai defavorabilă în cazul celor locale, fără însă a depăși limita de 0.5 m între două sisteme de pe două benzi adiacente.

Încărcările uniform distribuite sunt reprezentate de sistemul UDL cu greutatea " $\alpha_{qq} q_k$ " pe metru pătrat;



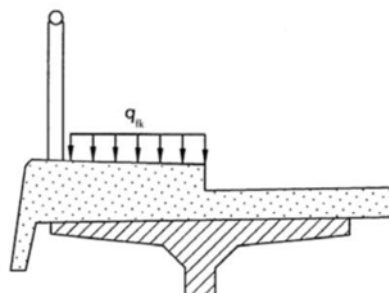
Coeficienții de corecție pentru încărcările uniform distribuite și concentrate " α_q " și " α_Q " se iau egali cu 1 (conform secțiunea 4-4.3.2(3)-SR EN 1991-2:2004/NB:2006).

Poziția	Sistemul tandem TS	Sistemul UDL
	Încărcarea pe osie Q_{ik} (kN)	q_{ik} (sau q_{rk}) (kN/m ²)
Banda numărul 1	300	9,0
Banda numărul 2	200	2,5
Banda numărul 3	100	2,5
Alte benzi	0	2,5
Zona rămasă (q_{rk})	0	2,5

Nota: Valorile încărcărilor prezentate in tabel includ amplificarea dinamică.

Încărcarea uniform distribuită

Pentru podurile de șosea care susțin trotuare pentru pietoni sau piste pentru cicliști, se definește o forță uniform distribuită q_{fk} .



Valoarea $q_{fk}=3\text{kN/mp}$ pentru gruparea gr1a.

2.2.2.1.2 Forțe orizontale

Forța de franare (SR EN 1991-2:2005)

Forța de franare " Q_{lk} " este o forță ce acționează în sens longitudinal podului la nivelul suprafeței de rulare a părții carosabile.

Valoarea caracteristică Q_{lk} este limitată la 900 kN pentru întreaga lățime a podului, și trebuie calculată ca fracțiune din totalul sarcinilor verticale maxime corespunzătoare convoiului LM1, aplicabile benzii nr.1 după cum urmează:

$$Q_{lk} = 0.60\alpha_{Q1} (2Q_{1k}) + 0.10 \alpha_{q1} q_{1k} w_1 L \leq 900 (\text{kN})$$

L = lungimea tablierului sau a părții considerate în calcul.

Pentru cazurile curente când lungimea de încărcare $L > 1,2\text{m}$ forța de franare este egală cu $Q_{lk} = 360 + 2,7L \leq 900 (\text{kN})$.

Limita inferioară a forței de franare la podurile de șosea este de 300kN.

$L_{cale} =$	153.3	m	$w_1 =$	3	m
$Q_{1k} =$	300	kN	$\alpha_{Q1} =$	1	
$q_{1k} =$	9	kN/m ²	$\alpha_{q1} =$	1	
$Q_{lk} =$	773.91	kN	< 900	kN	
$Q_{lk} =$	773.91	kN			
Nr Grinzi	4				
$Q_{lk} / \text{grinda} =$	193.4775	kN/m - forța uniform distribuită pe lungimea grinzii			

2.2.2.2 Acțiuni termice (SR EN 1991-1-5:2005), temperatură ($T(-)$ și $T(+)$)

Valoarea caracteristică a componentei de temperatură uniformă este asociată fie contracției, fie dilatării fiind notată cu $\Delta_{TN, \text{contracție}}$ sau cu $\Delta_{TN, \text{dilatare}}$ pentru fiecare caz în parte.

$$\Delta_{TN, \text{contracție}} = T_0 - T_{e, \text{min}} \quad - \text{temperatura, } T(-)$$

$$\Delta_{TN, \text{dilatare}} = T_{e, \text{max}} - T_0 \quad - \text{temperatura, } T(+)$$

T_0 - reprezintă temperatura inițială, se utilizează valoarea recomandată 10°C (SR EN 1991-1-5:2004/NA:2008).

$T_{e.min}$ și $T_{e.max}$ reprezintă componentele de temperatură uniformă minimă și maximă.

$T_{e.min} = -32^{\circ}\text{C}$ și $T_{e.max} = 42^{\circ}\text{C}$ – conf. SR EN 1991-1-5:2004/NA:2008, tabel NA.1(RO).

2.2.3 Actiuni seismice (S)

Amplasamentul structurii este caracterizat de acceleratia maxima a terenului $a_g=0.20g$ si perioada de colt $T_c=0.7s$.

Pentru calculul forței seismice se respectă prevederile:

- SR EN 1998-1 – Proiectarea structurilor pentru rezistența la cutremur. Reguli generale, acțiuni seismice și reguli pentru clădiri;
- SR EN 1998-2 – Proiectarea structurilor pentru rezistența la cutremur. Poduri;
- SR EN 1998-2/NA – Proiectarea structurilor pentru rezistența la cutremur. Poduri. Anexa națională.
- P100/2013 – Cod de proiectare seismică.

Conform SE EN 1998-2/NA – clasa de importanță este "III" și $\gamma_I=1,3$ - coeficient de amplificare a forței seismice.

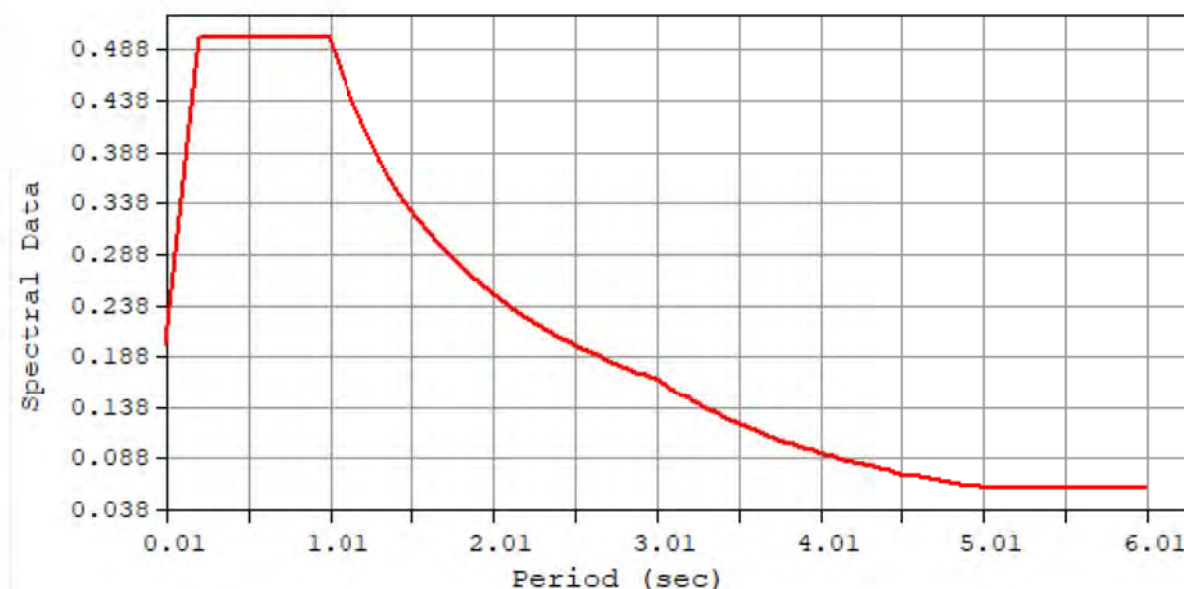


Fig. 3 Spectru de raspuns seismic orizontal

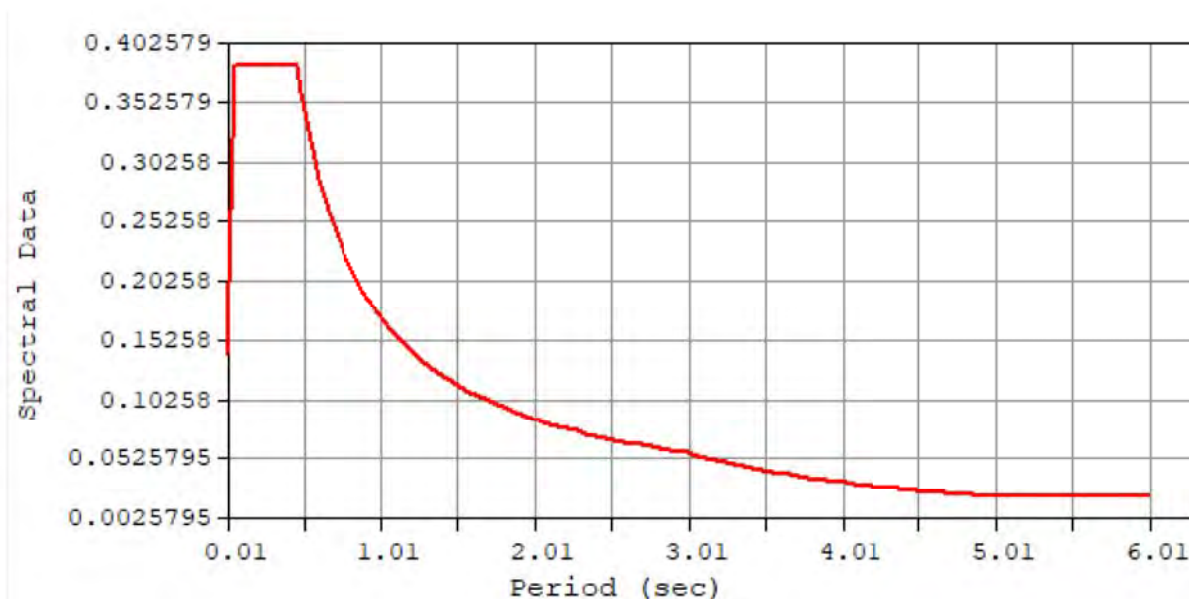


Fig. 4 Spectru de raspuns seismic vertical

3 COMBINAREA ACȚIUNILOR

Conform SR EN 1990 , Anexa A2 combinațiile sunt următoarele:

- Combinarea fundamentală din acțiuni persistente (permanente) și tranzitorii;
- Combinarea acțiunilor pentru situații accidentale;
- Combinarea acțiunilor pentru calcul seismic;

Coeficientii care intervin la combinarea acțiunilor pentru starea limită ultimă sunt:

$g_1=1,35$ – pentru acțiuni permanente (greutate proprie elemente structurale, împingere pământ);

$g_2=1,35$ – pentru acțiunea din convoi LM1 și încărcarea uniform distribuită pe trotuar;

Ψ_{i1} -0.75 pentru gruparea 1a – sistem TS tandem;

Ψ_{i2} -0.40 pentru gruparea 1a – sistem UDL forțe uniform distribuite

Ψ_{i3} -0.40 pentru gruparea 1a – încărcarea uniform distribuită pe trotuare

DESIGN TYPE : Concrete Design

LIST OF LOAD COMBINATIONS

NUM	NAME	ACTIVE	TYPE	LOADCASE(FACTOR) +	LOADCASE(FACTOR) +	LOADCASE(FACTOR)
1	cLCB1	Strength/Stress	Add			

		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	ML-caract(1.350)	
<hr/>				
2	cLCB2	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	ML-caract(1.350)	
<hr/>				
3	cLCB3	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+		Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+		Pamant/Radier(1.000) +	ML-caract(1.350)	
<hr/>				
4	cLCB4	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+		Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+		Pamant/Radier(1.000) +	ML-caract(1.350)	
<hr/>				
5	cLCB5	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	Temp (+)(1.500)	
<hr/>				
6	cLCB6	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	Temp (+)(-1.500)	
<hr/>				
7	cLCB7	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	Temp (-)(1.500)	
<hr/>				
8	cLCB8	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	Temp (-)(-1.500)	
<hr/>				
9	cLCB9	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+		Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+		Pamant/Radier(1.350) +	Temp (+)(1.500)	
<hr/>				
10	cLCB10	Strength/Stress	Add	
		Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)

+	Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+	Pamant/Radier(1.350) +	Temp (+)(-1.500)	
<hr/>			
11	cLCB11 Strength/Stress	Add	
	Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+	Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+	Pamant/Radier(1.350) +	Temp (-)(-1.500)	
<hr/>			
12	cLCB12 Strength/Stress	Add	
	Self Weight(1.350) +	Barrier(1.350) +	Additional Load1(1.350)
+	Wearing Surface(1.350) +	Utility1(1.350) +	Impingere pamant(1.350)
+	Pamant/Radier(1.350) +	Temp (-)(-1.500)	
<hr/>			
13	cLCB13 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (+)(1.500)	
<hr/>			
14	cLCB14 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (+)(-1.500)	
<hr/>			
15	cLCB15 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (-)(1.500)	
<hr/>			
16	cLCB16 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (-)(-1.500)	
<hr/>			
17	cLCB17 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (+)(1.500)	
<hr/>			
18	cLCB18 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+	Pamant/Radier(1.000) +	Temp (+)(-1.500)	
<hr/>			
19	cLCB19 Strength/Stress	Add	
	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+	Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)

	+	Pamant/Radier(1.000) +	Temp (-)(1.500)	
20	cLCB20	Strength/Stress	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	Temp (-)(-1.500)	
21	cLCB21	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	ML-caract(1.000)	
22	cLCB22	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	ML-caract(1.000) + Temp (+)(0.600)	
23	cLCB23	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	ML-caract(1.000) + Temp (+)(-0.600)	
24	cLCB24	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	ML-caract(1.000) + Temp (-)(0.600)	
25	cLCB25	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	ML-caract(1.000) + Temp (-)(-0.600)	
26	cLCB26	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	Temp (+)(1.000)	
27	cLCB27	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	Temp (+)(-1.000)	
28	cLCB28	Serviceability	Add	
		Self Weight(1.000) +	Barrier(1.000) + Additional Load1(1.000)	
	+	Wearing Surface(1.000) +	Utility1(1.000) + Impingere pamant(1.000)	
	+	Pamant/Radier(1.000) +	Temp (-)(1.000)	

29	cLCB29	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	Temp (-)(-1.000)	
30	cLCB30	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	ML-frecv(1.000) +	Temp (+)(0.500)
31	cLCB31	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	ML-frecv(1.000) +	Temp (+)(-0.500)
32	cLCB32	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	ML-frecv(1.000) +	Temp (-)(0.500)
33	cLCB33	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	ML-frecv(1.000) +	Temp (-)(-0.500)
34	cLCB34	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	Temp (+)(0.600)	
35	cLCB35	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	Temp (+)(-0.600)	
36	cLCB36	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	Temp (-)(0.600)	
37	cLCB37	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
				+ Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
				+ Pamant/Radier(1.000) +	Temp (-)(-0.600)	

38	cLCB38	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Pamant/Radier(1.000) +	Temp (+)(0.500)	
<hr/>						
39	cLCB39	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Pamant/Radier(1.000) +	Temp (+)(-0.500)	
<hr/>						
40	cLCB40	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Pamant/Radier(1.000) +	Temp (-)(0.500)	
<hr/>						
41	cLCB41	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Pamant/Radier(1.000) +	Temp (-)(-0.500)	
<hr/>						
42	cLCB42	Serviceability	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Pamant/Radier(1.000)		
<hr/>						
43	Seism_X	Strength/Stress	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Temp (+)(0.500) +	Pamant/Radier(1.000) +	Rs_X(1.000)
+				Rs_Y(0.300) +	ML-caract(0.300)	
<hr/>						
44	Seism_Y	Strength/Stress	Add	Self Weight(1.000) +	Barrier(1.000) +	Additional Load1(1.000)
+				Wearing Surface(1.000) +	Utility1(1.000) +	Impingere pamant(1.000)
+				Temp (+)(0.500) +	Pamant/Radier(1.000) +	Rs_X(0.300)
+				Rs_Y(1.000) +	ML-caract(0.300)	
<hr/>						

4 ANALIZA STRUCTURALA

4.1 Modelul structural

Analiza globală a fost realizată cu ajutorul programului de analiză element finit MIDAS Civil 2020. Suprastructura a fost modelată cu elemente tip bară legate cu link-uri elastice. Culeele se consideră încastrate la nivelul radierului. Analiza seismică - calculul modal cu spectre de răspuns.

4.1.1 Caracteristicile materialelor utilizate:

Tabel 1 Caracteristici materiale

ID	Name	Type	Standard	DB	Density (kN/m ³)
1	C50/60	Concrete	EN04 (RC)	C50/60	2.5000e+001
2	C35/45	Concrete	EN04 (RC)	C35/45	2.5000e+001
3	Y1860S7 (15.2mm)	Steel	EN05-PS (S)	Y1860S7 (15.2mm)	7.6980e+001
5	C30/37	Concrete	EN04 (RC)	C30/37	2.5000e+001
6	Dummy Material	User Defined	None	C35/45	0.0000e+000

Geometria structurii

Modelarea suprastructurii, elevatiei a fost realizata cu elemente de tip grinda (frame) pentru modelarea tablierului.

4.2 Centralizarea rezultatelor

4.2.1 Verificarea capacității portante ultime a piloților forăți

4.2.1.1 Piloți Culee

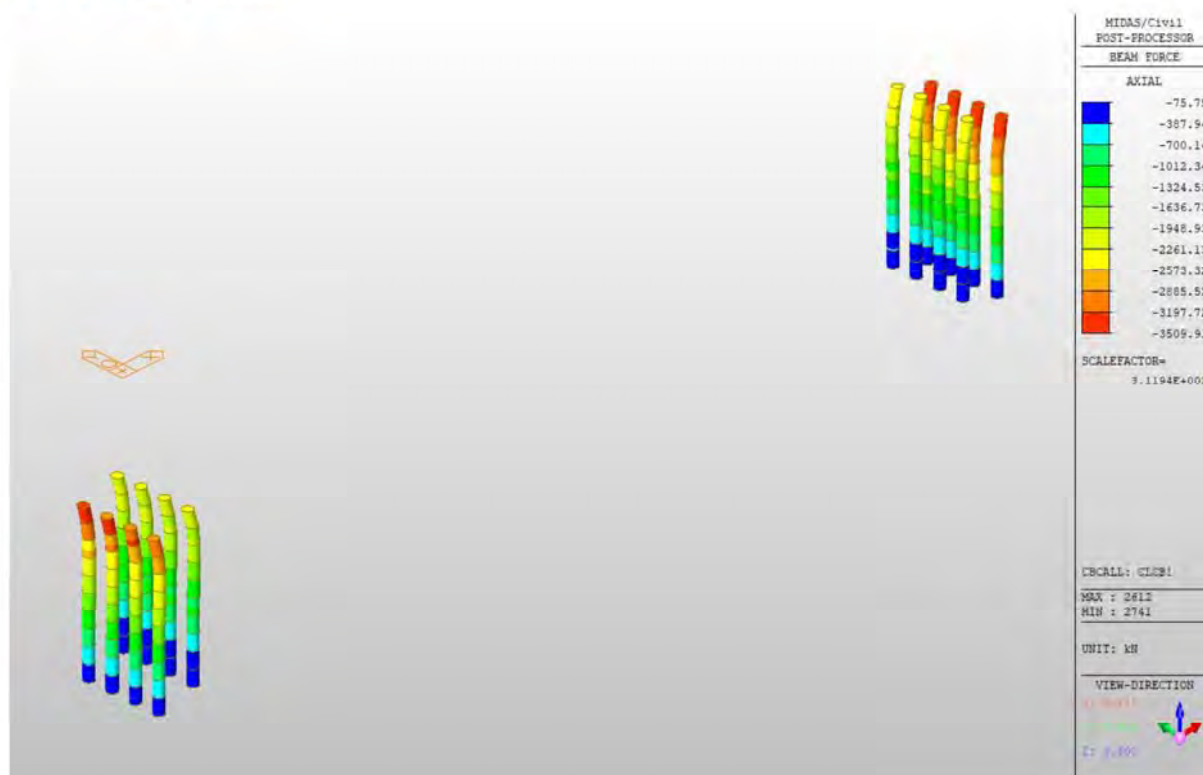


Fig. 5 Solicitări axiale maxime la piloții din culee

Fcd= 3510 kN – valoarea de calcul a încărcării axiale de compresiune asupra unui pilot corespunzător stării limite ultime – Gruparea cLCB1

Rcd= 4439 kN – capacitatea portantă ultimă la compresiune a pilotului, determinată cu programul de calcul GeoStru, anexat breviarului.

Fcd < Rcd – Adevărat

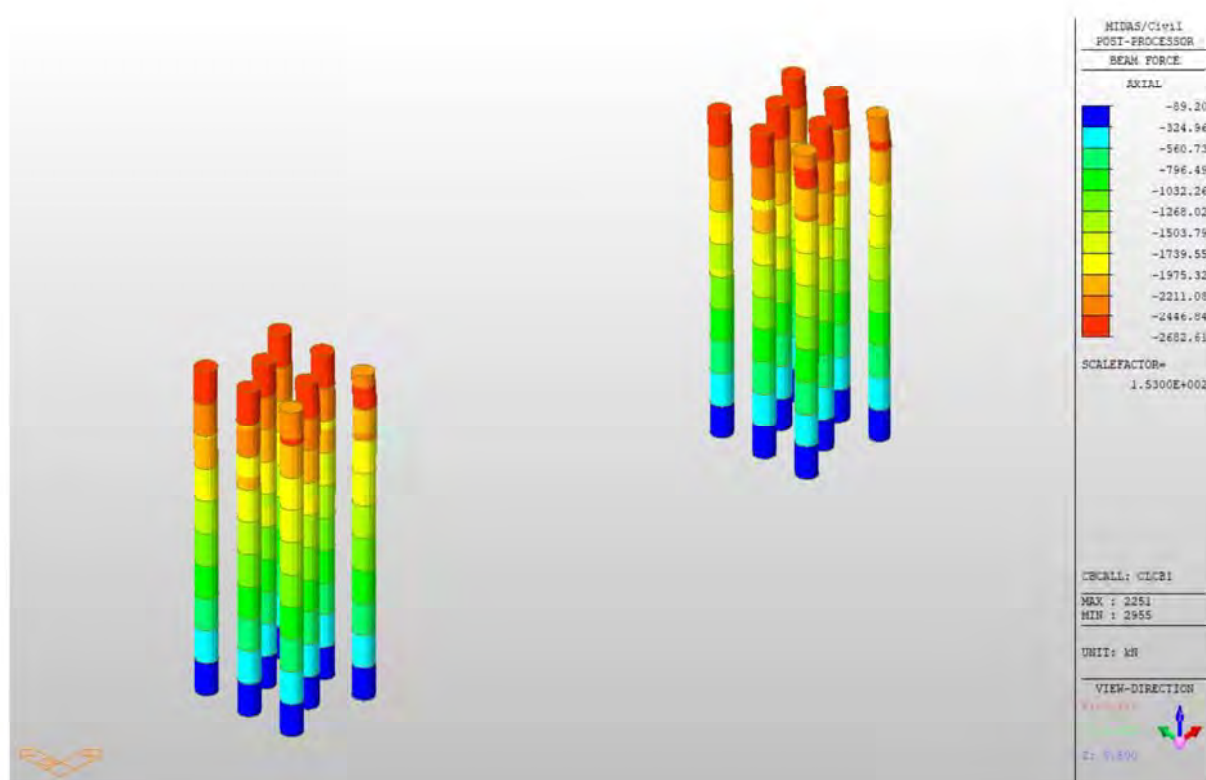


Fig. 6 Solicitări axiale maxime la piloții din pile

Fcd= 2683 kN – valoarea de calcul a încărcării axiale de compresiunea asupra unui pilot corespunzător stării limite ultime – Gruparea CLCB1

Rcd= 4439 kN – capacitatea portantă ultimă la compresiune a pilotului, determinată cu programul de calcul GeoStru, anexat breviarului.

Fcd < Rcd – Adevărat

4.2.2 Dimensionare armătură piloți la moment încovoiator și forță tăietoare

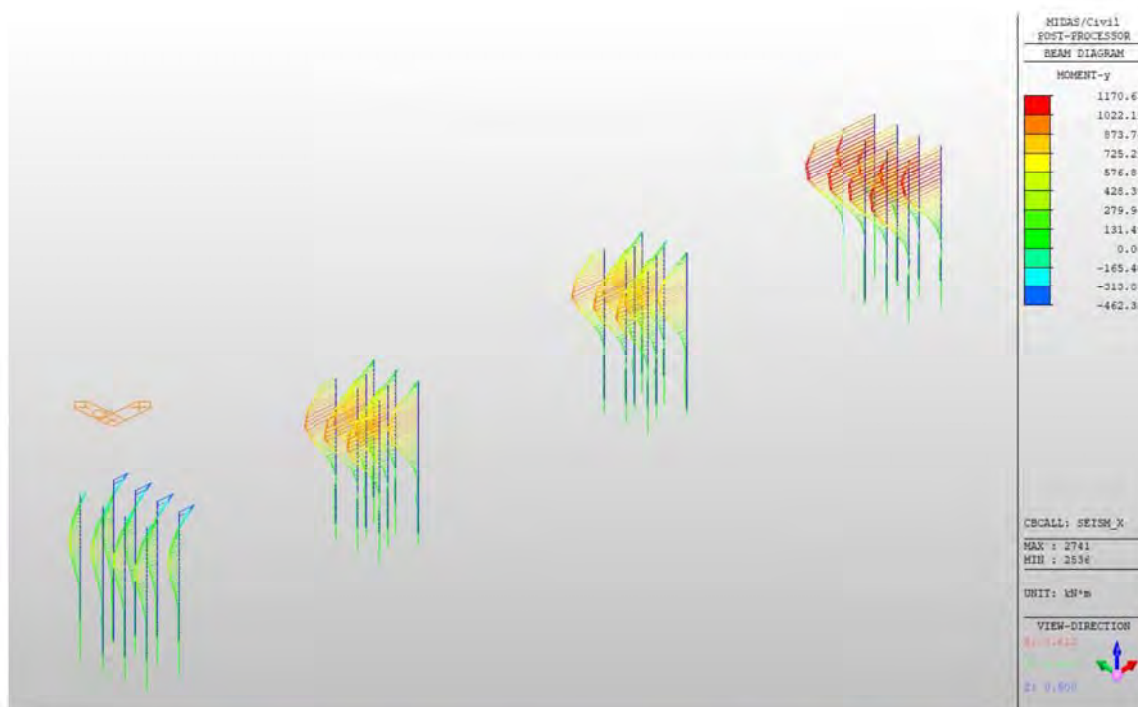


Fig. 7 Diagrama de moment încovoiator maxim în piloți din combinația seismică

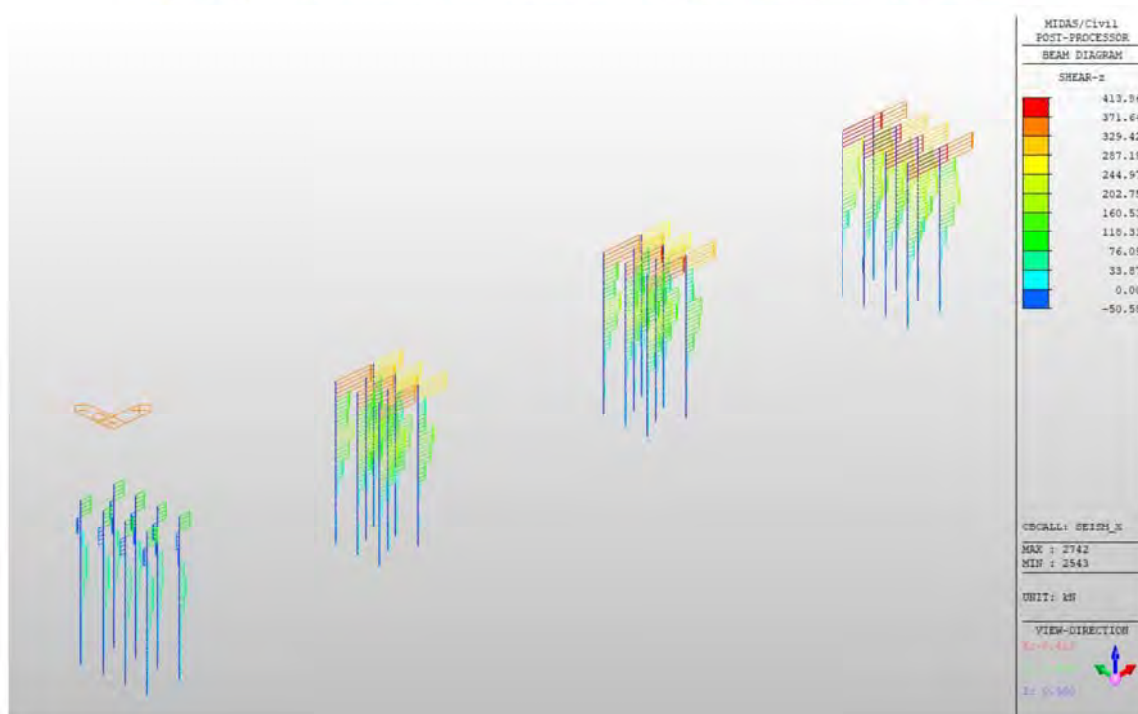


Fig. 8 Diagrama de forță tăietoare maximă din combinația seismică

PILOT**MIDAS/Civil****RC Column Checking Result**

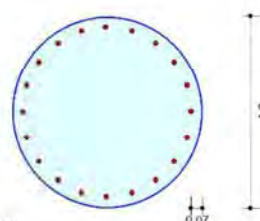
Company		Project Title	
Author	Atti	File Name	C:\...\Str 01 - 4 grinzi 36.mcb

1. Design Condition

Design Code Eurocode2-2.05
 Unit System kN, m
 Member Number 2750 (PM), 2742 (Shear)
 Material Data $f_{ck} = 30000$, $f_{yk} = 500000$, $f_{yw} = 500000$ KPa
 Column Height 1.89 m
 Section Property Pilot_culee (No : 10)
 Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	20-P20	--	--

Total Rebar Area $A_{st} = 0.0062832 \text{ m}^2$ ($R_{hst} = 0.0056$)

**2. Applied Loads**

Load Combination 33+ AT (I) Point

$N_{Ed} = 510.564 \text{ kN}$, $M_{Edy} = 1137.87$, $M_{Edz} = 82.8254$, $M_{Ed} = 1140.88 \text{ kN-m}$

3. Axial Forces and Moments Resistance Check

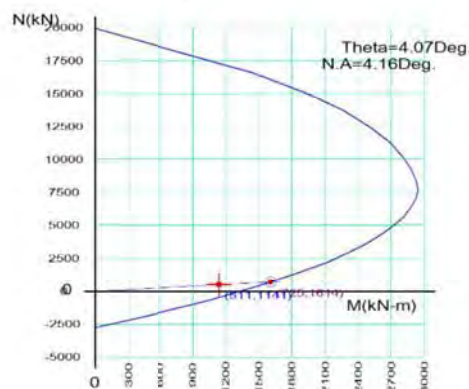
Concentric Max. Axial Load $N_{Rdmax} = 19939.6 \text{ kN}$

Axial Load Ratio $N_{Ed}/N_{Rd} = 510.564 / 724.960 = 0.704 < 1.000$ O.K

Moment Ratio $M_{Edy}/M_{Rdy} = 1137.87 / 1610.40 = 0.707 < 1.000$ O.K

$M_{Edz}/M_{Rdz} = 82.8254 / 114.622 = 0.723 < 1.000$ O.K

$M_{Ed}/M_{Rd} = 1140.88 / 1614.48 = 0.707 < 1.000$ O.K

4. P-M Interaction Diagram

$N_{Rd}(\text{kN})$	$M_{Rd}(\text{kN-m})$
19939.59	0.00
15124.76	1921.67
12426.36	2523.56
10200.70	2811.12
8436.06	2931.82
7115.58	2933.12
6101.88	2864.63
5186.40	2755.85
4197.23	2590.72
2983.70	2321.41
1421.01	1860.79
-587.30	1076.46
-2731.83	0.00

5. Shear Force Resistance Check

Applied Shear Resistance $V_{Ed} = 412.489 \text{ kN}$ (Load Combination 33-)

Shear Strength by Conc $V_{Rdc} = 414.373 \text{ kN}$

Shear Resistance by Rebar $V_{Rds} = 967.274 \text{ kN}$ (2.0-P10 @50)

Shear Ratio $V_{Ed}/V_{Rd} = 412.489 / 414.373 = 0.995 < 1.000$ O.K

4.2.3 DIMENSIONARE ARMĂTURĂ RADIER ȘI ELEVATIE CULEE

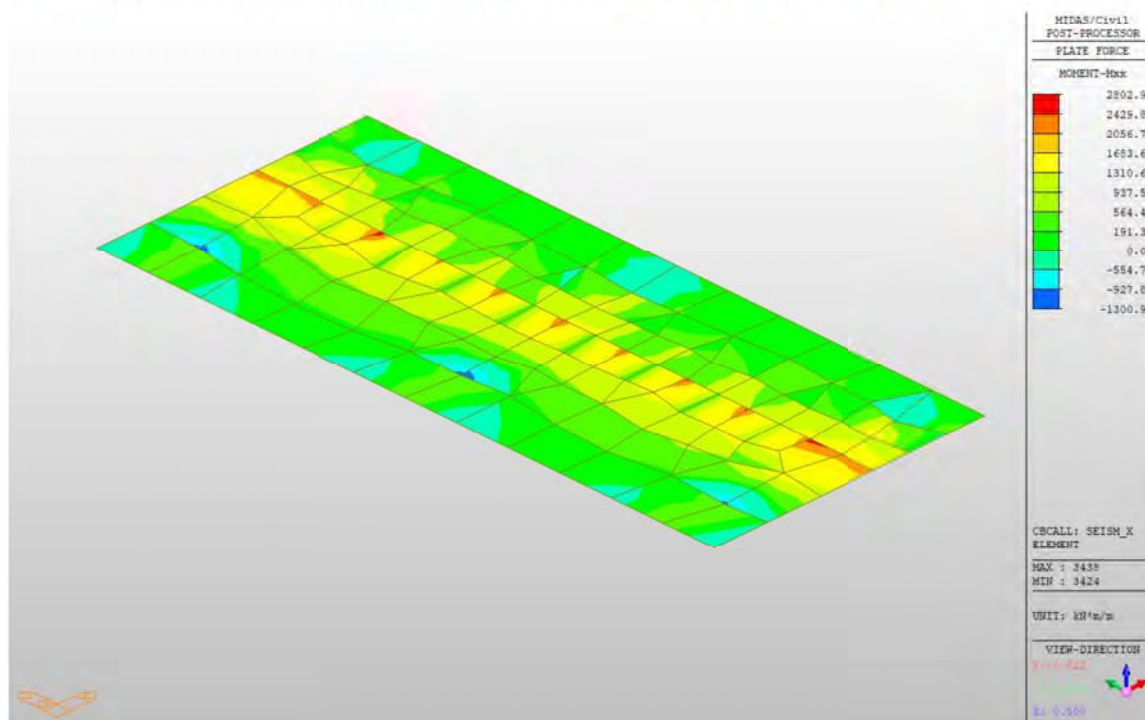


Fig. 9 Moment încovoietor maxim în radier culee

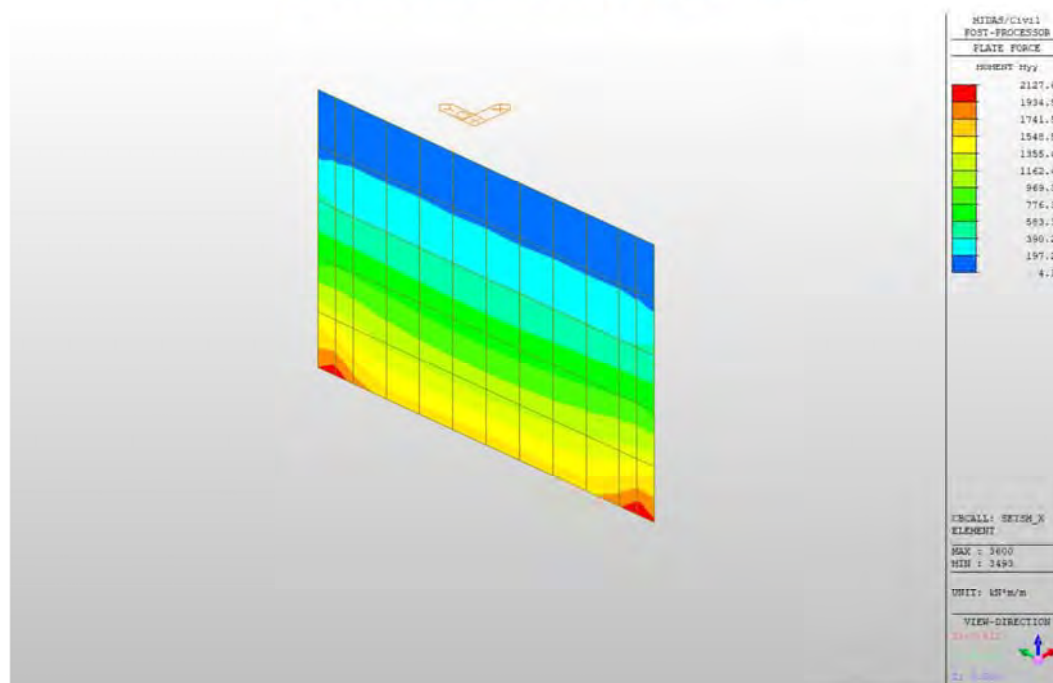


Fig. 10 Moment încovoietor în stâlpul culeei

RADIER CULEE

MIDAS/Civil

Plate Beam Check Result Dialog



Company		Project Title	
Author	Atti	File Name	C:\...\Str4 - 5 grinzi in sect.mcb

1. Design Condition

Design Type : Plate Beam (1D)
 Sub-Domain : radier
 Design Code : Eurocode2-2.05
 Unit System : kN, m, / m
 Material Data : fck = 30000, fyk = 500000, fyw = 500000 KPa
 Thickness : 1.6 m

2. Section Diagram



Element No : 3438

Rebar Pattern	Top(Negative)	Bottom(Positive)
Layer 1	P25@0.15	P32@0.15
Total Rebar Area Ast = 0.00863413 m^2/m		
Using Stirrups Spacing : 4.0-P20 @200		

3. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
Mu	1986.44	2802.99
Element No.	3239	3438
Load Combination	cLCB1	Seism_X
Mr	2141.41	3448.23
Check Ratio (Mu/Mr)	0.9276	0.8126
Using Rebar(As)	0.0033	0.0054

4. Shear Capacity

Element No. : 3417
 Load Combination : cLCB1
 Applied Shear Force V_{Ed} = 551.208
 Shear Strength (Out of plane) V_{Rdc} = 551.437 V_{Rds} = 0.00000
 Shear Ratio V_{Ed}/V_{Rd} 551.208 / 551.437 = 1.000 < 1.000 O.K

ELEVAȚIE CULEE

MIDAS/Civil **Plate Column Check Result Dialog**


MIDAS	Company		Project Title	
	Author	Atti	File Name	C:\...\Str4 - 5 grinzi in sect.mcb

1. Design Condition

Design Type : Plate Column (1D)
 Sub-Domain : elev-culee
 Design Code : Eurocode2-2.05
 Unit System : kN, m, / m
 Material Data : fck = 30000, fyk = 500000, fyw = 500000 KPa
 Rebar Pattern

	Top(Negative)	Bottom(Positive)
Layer 1	P20@0.15	P25@0.15

Total Rebar Area Ast = 0.00536687 m²/m
 Using Stirrups Spacing 2.0-P12 @150



2. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
M_Ed	1325.27	2127.62
Element No.	3552	3600
Load Combination	cLCB1	Seism_X
M_Rd	1567.34	2426.81
Check Ratio (M_Ed/M_Rd)	0.8456	0.8767
Using Rebar(As)	0.0021	0.0033

3. Axial Force Resistance Check

Element No. 3552
 Load Combination cLCB1
 Applied Axial Force Strength N_Ed = 2174.01
 Axial Force Strength N_Rd = 32842.2
 Axial Force Ratio $N_{Ed}/N_{Rd} = 2174.01 / 32842.2 = 0.066 < 1.000$ O.K

4. Shear Capacity

Element No. 3552
 Load Combination Seism_X
 Applied Shear Force V_Ed = 458.042
 Shear Strength (Out of plane) V_Rdc = 519.255 V_Rds = 0.00000
 Shear Ratio $V_{Ed}/V_{Rd} = 458.042 / 519.255 = 0.882 < 1.000$ O.K

4.2.4 DIMENSIONARE ARMĂTURĂ RADIER ȘI STÂLP PILĂ

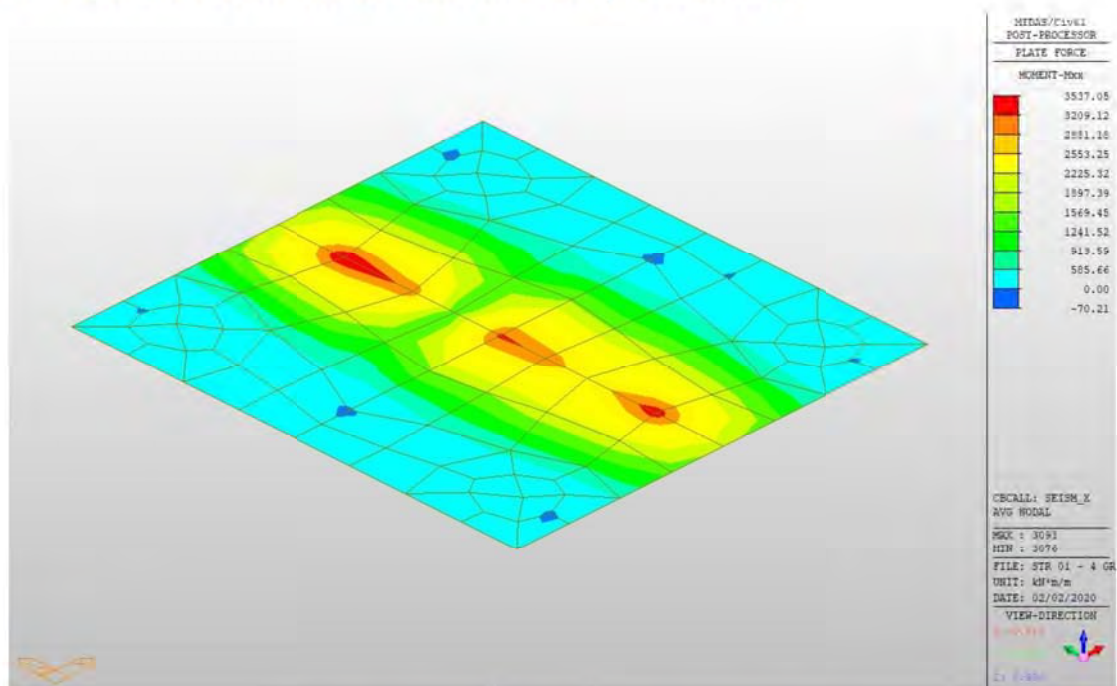


Fig. 11 Moment încovoiitor maxim în radierul pilei



Fig. 12 Moment încovoiitor maxim în stâlpul pilei

RADIER PILĂ

MIDAS/Civil

Plate Beam Check Result Dialog



Company		Project Title	
Author	Atti	File Name	C:\... \Str 01 - 4 grinzi 36.mcb

1. Design Condition

Design Type	Plate Beam (1D)
Sub-Domain	Radier_pila
Design Code	Eurocode2-2:05
Unit System	kN, m, / m
Material Data	fck = 30000, fyk = 500000, fyw = 500000 KPa
Thickness	1.6 m

2. Section Diagram



Element No : 3092

Rebar Pattern	Top(Negative)	Bottom(Positive)
Layer 1	P25@0.15	P32@0.15
Total Rebar Area	Ast = 0.00863413 m ² /m	
Using Stirrups Spacing	2.0-P25 @200	

3. Bending Moment Capacity

	Top(Negative)	Bottom(Positive)
Mu	590.34	3459.14
Element No.	3035	3092
Load Combination	cLCB1	Seism_X
Mr	2141.41	3448.28
Check Ratio (Mu/Mr)	0.2757	1.0032
Using Rebar(As)	0.0033	0.0054

4. Shear Capacity

Element No.	2261
Load Combination	cLCB1
Applied Shear Force	V_Ed = 551.278
Shear Strength (Out of plane)	V_Rdc = 551.437
Shear Ratio	V_Ed/V_Rd = 551.278 / 551.437 = 1.000 < 1.000 O.K

STÂLP PILĂ

MIDAS/Civil

RC Column Checking Result
Company
Author

Atti

Project Title**File Name**

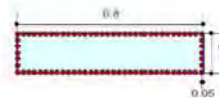
C:\...\Str 01 - 4 grinzi 36.mcb

1. Design Condition

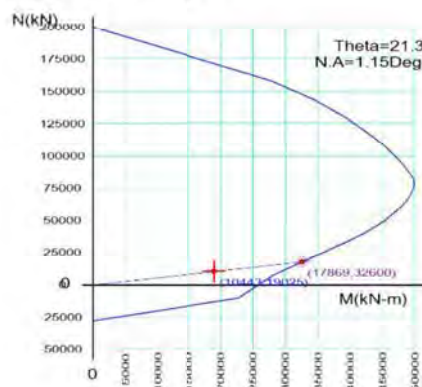
Design Code Eurocode2-2.05
 Unit System kN, m
 Member Number 2947 (PM), 2948 (Shear)
 Material Data $f_{ck} = 30000$, $f_{yk} = 500000$, $f_{yw} = 500000$ KPa
 Column Height 0.78 m
 Section Property Elev_pla (No : 5)

Rebar Pattern

	Pos 1	Pos 2	Pos 3
Layer 1	34-P32	5-P32	—

Total Rebar Area Ast = 0.0627315 m² (RhoSt = 0.0062)**2. Applied Loads**

Load Combination 33+ AT (J) Point

 $N_{Ed} = 10442.8$ kN, $M_{Edy} = 17757.4$, $M_{Edz} = 6827.09$, $M_{Ed} = 19024.6$ kN-m
3. Axial Forces and Moments Resistance CheckConcentric Max. Axial Load $N_{Rdmax} = 2.0e+005$ kNAxial Load Ratio $N_{Ed}/N_{Rd} = 10442.8 / 17869.2 = 0.584 < 1.000$ O.KMoment Ratio $M_{Edy}/M_{Rdy} = 17757.4 / 30355.4 = 0.585 < 1.000$ O.K $M_{Edz}/M_{Rdz} = 6827.09 / 11887.2 = 0.574 < 1.000$ O.K $M_{Ed}/M_{Rd} = 19024.6 / 32599.9 = 0.584 < 1.000$ O.K**4. P-M Interaction Diagram**

N_{Rd} (kN)	M_{Rd} (kN-m)
199608.13	0.00
143829.19	34336.26
118549.70	42739.50
98527.16	47297.42
82765.88	49874.20
72468.38	49642.01
64959.23	48647.93
57988.52	47320.48
50320.00	45397.94
40618.70	42311.38
27547.32	36999.87
7636.67	28249.49
-27274.57	0.00

5. Shear Force Resistance CheckApplied Shear Resistance $V_{Edy} = 1773.50$, $V_{Edz} = 655.745$ kN (Load Combination 34+)Shear Resistance (Y-Direction) $V_{Rdcy} = 2415.91$, $V_{Rdsy} = 5174.55$ kN (4.0-P10 @100)Shear Resistance (Z-Direction) $V_{Rdcz} = 3865.32$, $V_{Rdsz} = 1586.94$ kN (4.0-P10 @100)Shear Ratio $V_{Ed}/V_{Rd} = 1773.50 / 2415.91 = 0.734 < 1.000$ O.K

4.2.5 Eforturi in grinzi

Grinzile care intra in alcatuirea tablierului si ale caror caracteristici geometrice au fost descrise mai sus, au fost calculate tinandu-se seama de etapele de executie si de influenta fenomenelor reologice (contractie si curgere lenta).

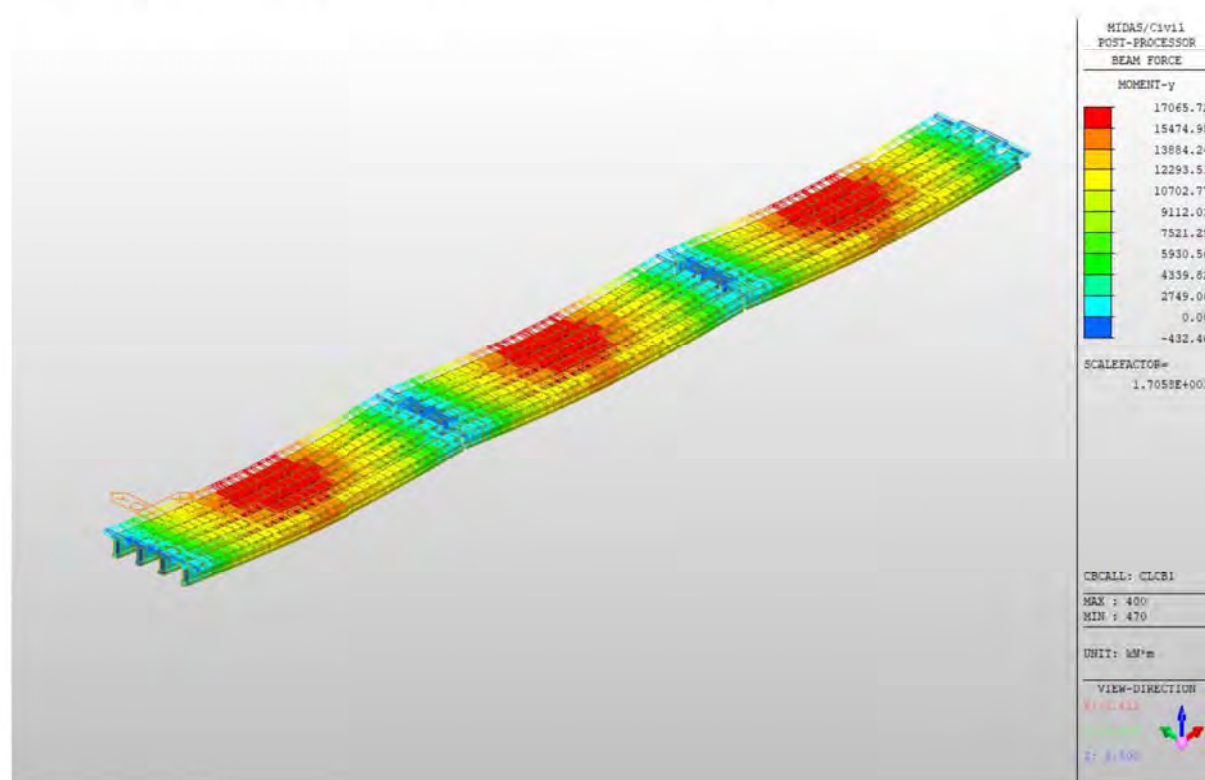


Fig. 13 Diagrama moment incovoietor grinzi

Dimensionarea grinzilor precomprimate s-a detaliat în breviare dinstincte pentru fiecare tip de grindă în parte.

5 ANEXA

PILOTI DE FUNDATIE

NORME DE REFERINTA

Pentru fazele de calcul se face referire la normele:

Eurocodul 7: Proiectare geotehnică – Partea 1: Reguli generale.

Eurocodul 8: Indicații de proiect pentru rezistența seismică a structurilor - Partea 5: Fundatii, structuri de sprijin, și spectre geotehnice.

Sarcina Limita Verticala

Sarcina limită verticală a fost calculată cu formule statistice, care o exprimă în funcție de geometria coloanei consolidate, caracteristicile terenului și interfata pilot-teren. Astfel, întrucât realizarea unui pilot, bătut sau forat, modifică întotdeauna caracteristicile terenului în jurul acestuia, se propune considerarea unui unghi de rezistență la tăiere egal cu:

$$\begin{aligned}\phi' &= \frac{3}{4} \phi + 10^\circ && \text{pentru piloti bătuti} \\ \phi' &= \phi - 3^\circ && \text{pentru piloti forati}\end{aligned}$$

unde ϕ este unghiul de rezistență la tăiere înainte de executie. În continuare vom indica cu ϕ parametrul de rezistență ales.

La sfârșitul calculului, sarcina limită Q_{lim} este divizată în mod conventional în două cote, rezistența la vârf Q_p și rezistența laterală Q_l .

Rezistența Unitară La Vârf

Formula lui Terzaghi

Soluția propusă de Terzaghi consideră că terenul existent deasupra adâncimii la care a ajuns vârful pilotului poate fi înlocuit de o suprasarcină echivalentă cu tensiunea verticală efectivă (neglijând faptul că interacțiunea dintre pilot și fundație ar putea modifica această valoare) și conduce analiza la problema capacității portante a unei fundații superficiale.

Formula lui Terzaghi poate fi scrisă:

$$Q_p = c \times N_c \times s_c + \gamma \times L \times N_q + 0.5 \times \gamma \times D \times N_\gamma \times s_\gamma$$

unde:

$$N_q = \frac{a^2}{2 \cos^2(45^\circ + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2) \tan \phi}$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = \frac{\tan \phi}{2} \left(\frac{K_{p\gamma}}{\cos^2 \phi} - 1 \right)$$

Metoda lui Berezantzev

În principiu *Berezantzev* face referire la o suprafață de alunecare "alla Terzaghi" care se opreste pe planul de sprijin (vârful pilotului); totuși acesta consideră că cilindrul de teren coaxial pilotului are diametrul egal cu extensia în secțiune a suprafeței de alunecare, este în parte "sustinut" prin acțiunea tangentială de către terenul rămas de-a lungul suprafeței laterale. Acesta dă o valoare a presiunii la baza inferioară a lui γD , și mai mică cu cât acest efect de "siloz" este marcant, adică cu cât mai mare este raportul D/B ; de acesta ține cont coeficientul N_q , care este deci funcție descrescătoare a lui D/B .

Rezistența unitară Q_p la vârf, pentru cazul terenului cu forfecarea (ϕ) și coeziunea (c), este dată de expresia:

$$Q_p = c \times N_c + \gamma \times L \times N_q$$

Indicând cu:

γ greutatea unității volumice a terenului;
 L lungimea coloanei;
 N_c e N_q sunt factorii capacității portante afectați de efectul formă (circulară);

Metoda lui Vesic

Vesic a asimilat problema rupturii în jurul vârfului pilotului și aceea a expansiunii unei cavități cilindrice în mediu elastico-plastic, în așa fel încât să se țină cont și de compresibilitatea mediului.

După *Vesic* coeficienții capacității portante N_q și N_c se pot calcula după cum urmează:

$$N_q = \frac{3}{3 - \sin \phi} \left\{ \exp \left[\left(\frac{\pi}{2} - \phi \right) \tan \phi \right] \tan^2 \left(45 + \frac{\phi}{2} \right) I_{rr}^{(4 \sin \phi) / [3(1 + \sin \phi)]} \right\}$$

Indicele de rigiditate redus I_{rr} în expresia precedentă este calculat plecând de la deformarea volumică ε_v .

Indicele de rigiditate I_r se calculează utilizând modulul de elasticitate elastică tangentială G' și rezistența la tăiere a terenului s .

Când avem de-a face cu condiții nedrenate sau solul se găsește într-o stare de densă, termenul ε_v poate fi considerat egal cu zero și se obține $I_{rr} = I_r$.

Este posibilă estimarea lui I_r cu următoarele valori:

TEREN	I_r
Nisip	75-150
Praf	50-75
Argilă	150-250

Termenul N_c al capacității portante este calculat:

$$N_c = (N_q - 1) \cot \phi \quad (a)$$

Când $\phi = 0$ (condiții nedrenate)

$$N_c = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

Metoda lui Janbu

Janbu calculează N_q (cu unghiul ψ) după cum urmează:

$$N_q = \left(\tan \phi + \sqrt{1 + \tan^2 \phi} \right)^2 \exp(2\psi \tan \phi)$$

N_c se poate calcula de la (a) când $\phi > 0$.

Per $\phi = 0$ se foloseste $N_c = 5.74$

Formula lui Hansen

Formula lui Hansen este valabilă pentru orice raport D/B , deci pentru fundatii de suprafată, dar si pentru cel profunde, acelasi autor a introdus coeficienti pentru o mai bună interpretare a comportamentului real al fundatiei, fără acestia, sarcina limită ar fi prea mult mărită odată cu adâncimea.

Pentru valori $L/D > 1$:

$$d_c = 1 + 0.4 \tan^{-1} \frac{L}{D}$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 \tan^{-1} \frac{L}{D}$$

În cazul $\phi = 0$

D/B	0	1	1.1	2	5	10	20	100
d'_c	0	0.40	0.33	0.44	0.55	0.59	0.61	0.62

În factorii următori, exeresiile cu acest semn (*) sunt egale cu $\phi = 0$.

Factor de formă:

$$s'_c = 0.2 \frac{D}{L}$$

$$s_c = 1 + \frac{N_q}{N_c} \frac{D}{L}$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_q = 1 + \frac{D}{L} \tan \phi$$

$$s_\gamma = 1 - 0.4 \frac{D}{L}$$

Factor de adâncime:

$$d'_c = 0.4k$$

$$d_c = 1 + 0.4k$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)k$$

$$d_\gamma = 1 \text{ per qualsiasi } k$$

$$k = \tan^{-1} \frac{L}{D} \text{ se } \frac{L}{D} > 1$$

Rezistenta Trunchiului

Metoda utilizată pentru calcularea capacității portante laterale este metoda α , propusă de Tomlinson (1971); rezistenta laterală este calculată după cum urmează:

$$Q_1 = (\alpha c + \sigma K \tan \delta) \cdot A_l \cdot f_w$$

A_l = suprafata laterală a pilotului;

f_w = factor de corectie legat de conicitatea trunchiului pilotului, sau reducerea diametrului pilotului;

c = valoarea medie a coeziunii (sau a rezistentei la tăiere în conditii nedrenate);

σ = presiunea verticală eficace a terenului;

K = coeficientul de alunecare orizontală, depinde de tehnologia de executie a pilotului si de starea precedentă de densitate, care se calculează după cum urmează:

Pentru piloti bătuti

$$K = 1 - \tan^2 \phi$$

Sau, în cazul specific, este posibilă desemnarea valorilor propuse în tabel:

Pilot	K	
	Teren afânat	Teren dens
Otel	0.5	1
Beton pref.	1	2
Lemn	1	3

Pentru piloti forati:

$$K = 1 - \sin \phi$$

δ = forfecarea pilot-teren în functie de asprimea suprafetei pilotului;

Pentru piloti bătuti

$$\delta = 3/4 \tan \phi$$

Pentru piloti forati

$$\delta = \tan \phi$$

α = coeficientul de adeziune găsit astfel:

Piloti forati:

$$\alpha = \frac{100 + c^2}{100 + 7c^2}$$

Caquot – Kerisel

Meyerhof – Murdock (1963)

$$\alpha = 1 - 0.1 \cdot c \text{ pentru } c < 5 \text{ t/m}^2$$

$$\alpha = 0.525 - 0.005 \cdot c \text{ pentru } c \geq 5 \text{ t/m}^2$$

Whitaker – Cooke (1966)

$$\alpha = 0.9 \text{ pentru } c < 2.5 \text{ t/m}^2$$

$$\alpha = 0.8 \text{ pentru } 2.5 \leq c < 5 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 5 \leq c \leq 7.5 \text{ t/m}^2$$

$$\alpha = 0.9 \text{ pentru } c > 7.5 \text{ t/m}^2$$

Woodward (1961)

$$\alpha = 0.9 \text{ pentru } c < 4 \text{ t/m}^2$$

$$\alpha = 0.6 \text{ pentru } 4 \leq c < 8 \text{ t/m}^2$$

$$\alpha = 0.5 \text{ pentru } 8 \leq c < 12 \text{ t/m}^2$$

$$\alpha = 0.4 \text{ pentru } 12 \leq c \leq 20 \text{ t/m}^2$$

$$\alpha = 0.20 \text{ pentru } c > 20 \text{ t/m}^2$$

Piloti bătuti

Coeficientul α pentru pilot bătut	
$2.5 \leq c < 5 \text{ t/m}^2$	$\alpha = 1.00$
$5 \leq c < 10$	$\alpha = 0.70$

$10 \leq c < 15$	$\alpha = 0.50$
$15 \leq c < 20$	$\alpha = 0.40$
$c \geq 20$	$\alpha = 0.30$

Forfeacrea Negativa

Când un pilot este bătut sau trece printr-un strat de material compresibil înainte de a se fi terminat procesul de consolidare, terenul se va misca în raport cu pilotul făcând să apară eforturi de forfecare între pilot și teren care duc la așa-zisul fenomen de forfecare negativă. Efectul forfecării negative este acela de a crește sarcina axială asupra pilotului, având ca și consecință creșterea tasărilor. Forța care ia naștere ca rezultat al forfecării negative este estimată ca fiind egală cu componenta activă a rezistenței laterale (v. Rezistența Trunchiului) de-a lungul suprafeței laterale la contactul cu stratul în care se generează astfel de fenomen, dar din partea opusă forfecării negative. Rezultatul astfel determinat nu este dedusă din sarcina limită, ci din aceea de exercițiu.

Factorul De Corectie În Condiții Seismice

Criteriul lui Vesic

Vesic apreciază că pentru a ține cont de fenomenul de creștere în calcul a capacității portante este suficientă reducerea cu 2° a unghiului de forfecare a straturilor de fundație. Limita acestei sugestii este faptul că nu ține cont de intensitatea solicitării seismice (exprimată prin parametrul accelerației seismice orizontale maxime). Acest criteriu pare să găsească confirmarea în observațiile făcute în prezenta diverselor fenomene seismice.

Criteriul lui Sano

Autorul propune reducerea unghiului de forfecare a straturilor portante cu o cantitate dată în ecuația:

$$D_p = \arctg\left(\frac{A_{max}}{\sqrt{2}}\right)$$

unde A_{max} reprezintă accelerația seismică orizontală maximă.

Acest criteriu, față de cel al lui **Vesic**, reprezintă avantajul de a lua în considerare și intensitatea solicitării seismice. Experiența demonstrează totuși că aplicarea fără apelul la critică a acestei realități poate conduce la valori prea mici ale lui **Qlim**.

Corecțiile lui **Sano** și **Vesic** se aplică exclusiv pentru terenuri necoezive dense. Este greșită aplicarea lor la terenurile afânate sau cu densitate medie, unde vibrațiile seismice produc fenomenul opus celui de dilatare, cu creșterea gradului de densitate și a unghiului de forfecare.

TASARI

Tasarea verticală a fost calculată cu metoda lui Davis-Poulos, în funcție de care pilotul este considerat rigid (nedeformabil) înecat într-un mediu elastic, semispatiu sau strat de grosime finită.

Se ipotizează că interacțiunea pilot-teren este constantă la fâșii de-a lungul a n suprafețe cilindrice în care se împarte suprafața laterală a pilotului.

Tasarea suprafeței generice i ca efect la sarcina limită transmisă de la pilot la teren de-a lungul suprafeței cu nr. j poate fi exprimat:

$$W_{i,j} = (\tau_j / E) \times B \times I_{i,j}$$

Unde:

τ_j = Creșterea tensiunii în funcție de punctul mediu al fâșiei;

E = Modulul elastic al terenului;

B = Diametrul pilotului;

$I_{i,j}$ = Coeficientul de influență.

Tasarea compresivă se obține însumând $W_{i,j}$ cu toate ariile j .

SARCINA LIMITĂ ORIZONTALĂ

Sarcina limită orizontală a fost calculată după teoria dezvoltată de Broms, care presupune că comportamentul interfeței pilot-teren este rigidă perfect elastică, și astfel rezistența terenului să fluctueze cu orice valoare, nu nulă,

a deplasării și să rămână constantă la creșterea deplasării.

Se presupune că comportamentul flexionar al pilotului este de tip rigid, perfect elastic, trebuie să spunem că rotațiile elastice ale pilotului sunt neglijabile până când momentul încovoietor ajunge la valoarea M_y de plasticizare.

Pentru terenuri coesive Broms propune adoptarea unei reacții a terenului constantă cu adâncimea egală cu:

$$p = 9 \times c_u \times B$$

cu reacția nulă până la adâncimea de 1.5 d; indicând cu:

c_u = Coeziunea nedrenată;

B = Diametrul pilotului;

p = Reacția terenului pentru unitatea de lungime a pilotului.

Pentru terenurile necoezive se presupune că rezistența variază liniar în funcție de adâncime după formula:

$$p = 3K_p \gamma z B$$

indicând cu:

p = Reacția terenului pentru unitatea de lungime a pilotului;

K_p = Coeficientul de împingere pasivă;

γ = Greutatea volumică a terenului;

z = Adâncimea;

B = Diametrul pilotului

PILOT ÎN CONDITII DE EXERCITIU

Analiza pilotului în condiții de exercițiu: **Metoda elementelor finite.**

Metoda elementelor finite modelează pilotul de fundație supus la sarcini transversale, în mod realist întrucât face uz, fie de împingeri cât și de rotații în noduri pentru definirea linierei elastice a pilotului, deci reprezintă metoda cea mai rațională și eficientă disponibilă actualmente pentru analiza acestor tipuri de structuri.

În ceea ce urmează se face apel la teoria metodelor, indicând cu \mathbf{P} matricea forțelor nodale externe, cu \mathbf{F} aceea a forțelor interne și cu \mathbf{A} matricea coeficienților de influență care, pentru echilibrul între forțele externe și interne, leagă primele două în funcție de formula:

$$\mathbf{P} = \mathbf{A}\mathbf{F}$$

Deplasările interne \mathbf{e} (împingerile și rotațiile) ale elementului noduli generic sunt legate de deplasările externe \mathbf{X} (împingerile și rotațiile) aplicate la noduri, ale următoarei formule:

$$\mathbf{e} = \mathbf{B}\mathbf{X}$$

unde matricea \mathbf{B} s-a demonstrat a fi transpunerea matricei \mathbf{A} .

De altă parte, forțele interne \mathbf{F} sunt legate de deplasările interne \mathbf{e} ale următoarei expresii:

$$\mathbf{F} = \mathbf{S}\mathbf{e}$$

Aplicând substituirile de rigoare se obține:

$$\mathbf{F} = \mathbf{S}\mathbf{A}^T\mathbf{X}$$

Și deci:

$$\mathbf{P} = \mathbf{A}\mathbf{F} = \mathbf{A}\mathbf{S}\mathbf{A}^T\mathbf{X}$$

Deci, calculând inversa matricei $\mathbf{A}\mathbf{S}\mathbf{A}^T$ se obține expresia deplasărilor externe \mathbf{X} :

$$\mathbf{X} = (\mathbf{A} \mathbf{S} \mathbf{A}^T)^{-1} \mathbf{P}$$

Notati, deci, că cu ajutorul deplasărilor \mathbf{X} este posibilă aflarea forțelor interne \mathbf{F} necesare pentru proiectul structurii.

Matricea $\mathbf{A} \mathbf{S} \mathbf{A}^T$ este cunoscută ca și matrice de rigiditate globală deoarece caracterizează legătura dintre deplasări și forțele externe nodale.

Metoda elementelor finite are, printre altele, avantajul că dă posibilitatea de a lua în considerare, ca și condiții la contur, rotații și deplasări cunoscute.

Rotările nodale care schematizează terenul sunt considerate forțe globale legate de modulul de reacție și de zona de influență a nodului. În cadrul soluției elementelor finite pentru piloti supuși la sarcini transversale, modulul de reacție este considerat în forma:

$$k_s = A_s + B_s Z^n$$

sau, nevrînd să creștem nelimitat k_s cu adâncimea, în forma:

$$k_s = A_s + B_s \tan^{-1}(Z/B)$$

în care Z reprezintă adâncimea iar B reprezintă diametrul pilotului.

Valorile lui A_s și $B_s Z^n$ sunt obținute din expresia capacității portante (Bowles) cu factori corectori s_i , d_i , și i_i egali cu 1:

$$k_s = q_{ult}/\Delta H = C(cN_c + 0.5\gamma B N_\gamma)$$

$$B_s Z^n = C(\gamma N_q Z^1)$$

Unde $C = 40$ este obținut în corespondență cu o tasare maximă de 25 mm.

Date generale...

Diametru varf	1.20 m
Lungime	16.00 m
Tip	Forat
Capacitatea de varf calculată cu:	Terzaghi
Adâncime panza freatică de la nivelul solului	3.90 m
Tip beton	2
Tip otel	1

Arhiva materiale

Beton

Nr.	Clasa Beton	f _{ck} ,cubic [Kg/cm ²]	E _c [Kg/cm ²]	f _{ck} [Kg/cm ²]	f _{cd} [Kg/cm ²]	f _{ctd} [Kg/cm ²]	f _{ctm} [Kg/cm ²]
1	C20/25	250	299600	200	113.3	10.1	22.1
2	C25/30	300	314750	250	141.6	11.4	25.6
3	C28/35	350	323080	280	158.6	12.6	27.6
4	C40/50	500	352200	400	226.6	16.3	35

Oteluri:

Nr.	Steel Class	E _s [Kg/cm ²]	f _{yk} [Kg/cm ²]	f _{yd} [Kg/cm ²]	f _{tk} [Kg/cm ²]	f _{td} [Kg/cm ²]	ep _{tk}	ep _{d_ult}	B1*B2 in.	B1*B2 fin.
1	B450C	2000000	4500	3913	4500	3913	.075	.0675	1	0.5
2	B450C*	2000000	4500	3913	5400	4500	.075	.0675	1	0.5
3	B450C**	2000000	4500	3913	4582	3985	.012	.01	1	0.5
4	S235H	2141370	2447.28	2128.11	3670.92	2128.11	0.012	0.01	1	0.5
5	S275H	2141370	2855.16	2482.97	4384.71	2482.97	0.012	0.01	1	0.5

Varianta de ocolire Timisoare Sud - Pod km 24+725 peste canalul Bega

6	S355H	2141370	3670,92	3191.66	5200.47	3670.92	0.012	0.01	1	0.5
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Stratigrafie

Nr.: Numarul stratului Hs: Grosimea stratului Fi: Unghi de frecare c: Coeziune Alfa: Coef. aderenta frecari laterale. Vs: Viteza unde de forfecare.

Strat. 1

Nr.	Hs	Greutate volumica [kg/m³]	Greutate volumica saturata [kg/m³]	c [kg/cm²]	Fi (°)	Frecare negativa	Alfa	Modul elasticitate [kg/cm²]	Vs [m/s]	Descriere litologica
1	4.50	2000.00	1950.00	0.04	20.00	Nu	1.00	40.00	0	argilă nisipoasă
2	0.30	2100.00	1950.00	0.06	25.00	Nu	0.00	60.00	0	
3	0.20	2100.00	1950.00	0.06	25.00	Nu	0.00	60.00	0	
4	0.70	1850.00	1825.00	0.00	32.00	Nu	0.00	300.00	0	
5	0.80	1900.00	1875.00	0.00	36.00	Nu	0.00	500.00	0	
6	4.50	2100.00	2050.00	0.06	25.00	Nu	0.00	60.00	0	
7	2.00	1900.00	1875.00	0.00	36.00	Nu	0.00	700.00	0	
8	3.10	1950.00	1900.00	2.00	40.00	Nu	0.00	300.00	0	
9	2.00	1800.00	1750.00	0.00	28.00	Nu	0.00	100.00	0	
10	1.40	2100.00	2075.00	0.06	25.00	Nu	0.00	60.00	0	

Sacina limita

Stratigrafie	Nq	Nc	Fi/C strat varf pilot (°)/[kg/cm²]	Greutate pilot [kg]	Sarcina limita varf [kg]	Sarcina limita laterala [kg]	Sarcina limita [kg]	Frecare negativa [kg]	Sarcina limita orizontala [kg]
A1+M1+R 3	81.26	95.66	40/2.00	45238.94	3937251.0 0	465261.30	4357274.0 0	--	--

REZISTENTA DE PROIECTARE SARCINI AXIALE

Rezistenta caracteristica sarcini axiale. Nume combinatie:				A1+M1+R3
Numar de verticale inspectate				1
Factor de corelatie pentru verticale inspectate mediu (xi3)				1.70
Factor de corelatie pentru vertical inspectate minim (xi4)				1.70
	Rc, Min [kg]	Rc, Media [kg]	Rc, Max [kg]	
Baza	3937251.00	3937251.00	3937251.00	
Lateral	465261.30	465261.30	465261.30	
Total	4357274.00	4357274.00	4357274.00	
Coeficient partial rezistenta caracteristica				R3
Baza				1.35
Lateral				1.15
Rezistenta de proiectare baza				1715578.00 kg
Rezistenta de proiectare laterala				237985.30 kg
Rezistenta de proiectare				1908324.00 kg

Tasare (Fleming 1992)

Lungime	16.00 m
Diametru capat	1.20 m
Diametru varf	1.20 m
Tronson frecare laterala nula	0.00 m
Modul elasticitate sectiune	3.09E+07 kN/mq
Punct de aplicare a rezultantei rezistentei active	0.45
Factor de flexibilitate teren/pilot	1.00E-03
Sarcina aplicata	1910.00 kN
Sarcina limita laterala	2803.40 kN
Sarcina limita de varf	22712.49 kN

Modul elasticitate teren corespndent	3.92E+03 kN/mq
Scurtarea elastica	0.44 mm
Tasare rigida	2.49 mm
Tasare totala	2.93 mm



intocmit ing. Attila Takacs

A handwritten signature in blue ink, appearing to be "Attila Takacs", written over the printed name.



Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=21.00m, h=0.95m

Element Number	119
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

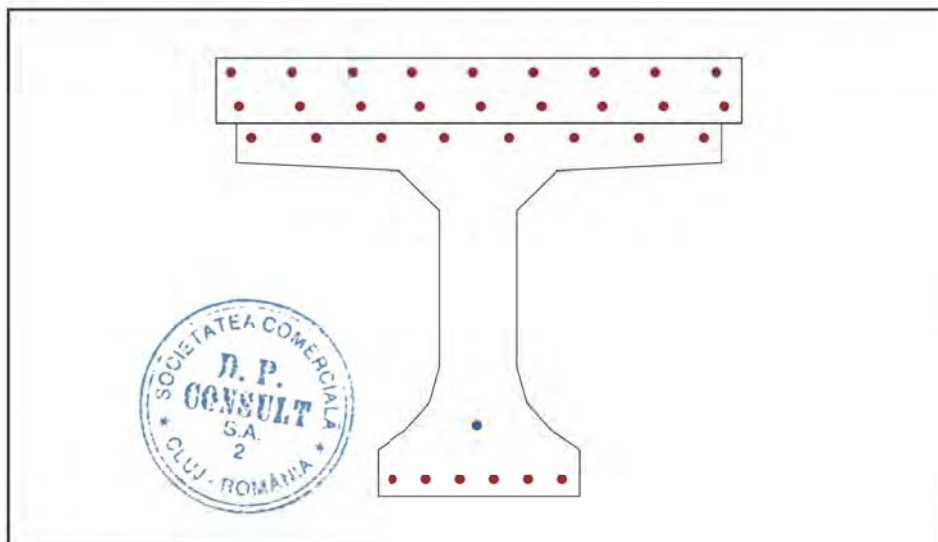
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

α_{cc} = 0.850 (for the Compressive strength)

α_{ct} = 1.000 (for the Tensile strength)

1.2 Sectional Information

Section Information	EqvInt sect(tdn, reinfmnt) (Girder)	EqvInt sect (After Comp) (Girder + Slab)
A (mm ²)	385063.151	596154.092
I_y (mm ⁴)	44903402436.405	79035158316.327
y_{st} (mm)	-	410.340
y_{sb} (mm)	-	240.340
y_t (mm)	411.522	240.340
y_b (mm)	538.478	709.660
Z_{st} (mm ³)	-	95564700.326
Z_{sb} (mm ³)	-	143832037.130
Z_t (mm ³)	30717871.974	143832037.130
Z_b (mm ³)	73722892.737	78921150.883



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$f_{ck(g)}$ = 50.000 MPa

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{ck}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{ck}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Tendon-Span1	940.8	3640.000	1863.260	1569.060	195000.000

2.Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 3038.975 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

4) Calculate the axial force in concrete, steel, and tendon.

5) Check if the resultant force of cross-section is zero.

6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	560.0	9367.567	1278.522	295.043	4966.416	2.02341
2	280.0	6219.167	1150.819	295.043	4966.416	1.40075
3	140.0	2887.733	559.882	688.435	4966.416	0.60967
4	210.0	4331.600	848.286	295.043	4966.416	0.98450
5	245.0	5267.167	1021.162	295.043	4966.416	1.19517
6	227.5	4791.167	941.373	295.043	4966.416	1.08953
7	218.8	4553.167	896.691	295.043	4966.416	1.03581
8	214.4	4434.167	872.983	295.043	4966.416	1.00868
9	212.2	4376.721	860.762	295.043	4966.416	0.99544
10	213.3	4404.417	866.903	295.043	4966.416	1.00187
11	212.7	4389.542	863.840	295.043	4966.416	0.99846
12	213.0	4396.979	865.374	295.043	4966.416	1.00017

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p

(EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 13.812 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 4383.167 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 865.374 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 295.043 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 4966.416 \text{ kN} && : \text{Tension Force in Tendon}
 \end{aligned}$$

$$\begin{aligned}
 \text{where, } a &= \lambda \cdot x = 170.406 \text{ mm} \\
 x &= 213.008 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 487.500 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 221000.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 2940.600 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 678.600 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1072.078	0.01196	0.00550	0.01746	1364.400	3640.000	4966.416
$F_p = \sum (F_{pi}) =$							4966.416

where, StressEff = $\epsilon_{p(0)i} \cdot E_p$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 4509.330 \text{ kN} \cdot \text{m} \geq M_{Ed} = 3038.975 \text{ kN} \cdot \text{m}
 \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MIN

$M_{Ed} = 0.000 \text{ kN} \cdot \text{m}$

- factor λ , and factor η

$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

Girder : $f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$

Slab : $f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

Girder : $f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

Slab : $f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

4) Calculate the axial force in concrete, steel, and tendon.

5) Check if the resultant force of cross-section is zero.

6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	560.0	4087.650	295.043	1278.522	0.000	3.42794
2	280.0	2789.150	295.043	1278.522	0.000	2.41231
3	140.0	1586.667	295.043	1278.522	4598.514	0.32018
4	210.0	2288.608	295.043	1278.522	0.000	2.02081
5	175.0	1967.467	295.043	1278.522	3962.424	0.43170
6	192.5	2135.812	295.043	1278.522	0.000	1.90130
7	183.8	2053.583	295.043	1278.522	0.000	1.83699
8	179.4	2011.011	295.043	1278.522	0.000	1.80369
9	177.2	1989.360	295.043	1278.522	3931.012	0.43850
10	178.3	2000.216	295.043	1278.522	3915.595	0.44190
11	178.8	2005.621	295.043	1278.522	3907.957	0.44359
12	179.1	2008.318	295.043	1278.522	3904.156	0.44443
13	179.2	2009.665	295.043	1278.522	0.000	1.80264
14	179.2	2008.991	295.043	1278.522	3903.207	0.44465
15	179.2	2009.328	295.043	1278.522	3902.733	0.44475
16	179.2	2009.496	295.043	1278.522	3902.496	0.44480
17	179.2	2009.581	295.043	1278.522	3902.378	0.44483
18	179.2	2009.623	295.043	1278.522	0.000	1.80260

19	179.2	2009.602	295.043	1278.522	0.000	1.80259
20	179.2	2009.581	295.043	1278.522	3902.378	0.44483

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 2009.581 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 295.043 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1278.522 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 3902.378 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 143.385 \text{ mm} \\
 x &= 179.230 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 70926.372 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 678.600 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 2940.600 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 1303.523 \text{ kN} \cdot \text{m} \geq M_{Ed} = 0.000 \text{ kN} \cdot \text{m} \quad \text{OK}
 \end{aligned}$$

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{cLCB1} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{FZ-MAX} \\
 N_{Ed} &= -3765.824 \text{ kN} \\
 V_{Ed} &= 245.059 \text{ kN} \\
 M_{Ed} &= 2834.381 \text{ kN} \cdot \text{m} \\
 V_p &= 0.000 \text{ kN}
 \end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

$$\begin{aligned}
 \text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
 \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
 \end{aligned}$$

- Check whether the section is cracked in bending.

$$\begin{aligned}
 f_b &= 4.614 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa} \\
 & \quad \text{(The section is cracked.)} \\
 \text{where, } f_b &: \text{Flexural tensile stress} && \text{(Tension is taken as positive.)} \\
 f_{ctd} &= \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c && : \text{Design tensile strength of concrete.}
 \end{aligned}$$

- Design value for the shear resistance $V_{Rd,c}$

$$\begin{aligned}
 V_{Rd,min} &= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 281.305 \text{ kN} \\
 V_{Rd,c} &= [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 284.161 \text{ kN} \geq V_{Rd,min} \\
 &= 284.161 \text{ kN} \geq V_{Ed} = 245.059 \text{ kN} \quad \therefore \text{Shear reinforcement is not required} \\
 \text{where, } C_{Rd,c} &= 0.18 / \gamma_c = 0.120 \\
 v_{min} &= 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.423 \\
 k_1 &= 0.15 \\
 k &= 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.430 \\
 \rho_1 &= A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003 \\
 A_{sl} &= 678.600 \text{ mm}^2 \\
 b_w &= 190.0 \text{ mm} \\
 d_p &= 1080.0 \text{ mm} \\
 \sigma_{cp} &= N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 6.317 \text{ MPa} \\
 N_{Ed} &= 3755.824 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 596154.092 \text{ mm}^2
 \end{aligned}$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
 \rho_{w,min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
 \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00595 \geq \rho_{w,min} = 0.00113 \quad \text{OK} \\
 \text{where, } \rho_{w,min} &\text{ is the minimum ratio of shear reinforcement} \\
 A_{sw} &= 226.200 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}
 \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
 S_{l,max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 809.997 \text{ mm} \\
 s &= 200.000 \text{ mm} \leq S_{l,max} = 809.997 \text{ mm} \quad \text{OK} \\
 \text{where, } s &\text{ is the spacing of the stirrups} \\
 d &= 1079.996 \text{ mm} \quad : \text{effective depth}
 \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
 \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 122.530 \text{ kN} \\
 (M_{Rd} - M_{Ed}) / z &= 1954.653 \text{ kN} \\
 \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
 \text{where, } M_{Rd} &= 4509.330 \text{ kN} \quad : \text{The moment resistance} \\
 z &= 856.904 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis}
 \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
 v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.238 \text{ MPa} \\
 \text{where, } \beta &= 1.0 \quad : \text{Conservatively adopted to take } \beta = 1.0 \\
 V_{Ed} &= 245.059 \text{ kN} \quad : \text{Transverse shear force} \\
 z &= 856.904 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis} \\
 b_i &= 1200.000 \text{ mm} \quad : \text{Width of interface}
 \end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
 v_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.448 \text{ MPa} \\
 v_{Rd,lim} &= 0.5 v f_{cd} = 8.000 \text{ MPa} \\
 v_{Edi} &= 0.238 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd,lim}] = 1.448 \text{ MPa} \quad \text{OK} \\
 \text{where, } f_{ctd} &= 1.900 \text{ MPa} \quad : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} \quad \text{of concrete between girder and slab} \\
 c &= 0.400 \quad : \text{factors which depend on the roughness of the interface}
 \end{aligned}$$

μ	=	0.700	: factors which depend on the roughness of the interface
σ_n	=	w_s / A_l	= 0.005 MPa
ρ	=	A_s / A_l	= 0.002
w_s	=	5.525 kN	: Weight of slab
A_l	=	1200000.000 mm ²	: Area of joint (1m for unit length)
A_s	=	2700.000 mm ²	: Area of reinforcement crossing the interface
f_{yd}	=	434.783 MPa	: Strength of shear connector
α	=	90.000 °	: Angle of reinforcement crossing the interface (45° ≤ α ≤ 90°)

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name :	cLCB1
Design Situations :	Persistent & Transient
Load Combination Type :	FZ-MIN
N_{Ed}	= -3776.870 kN
V_{Ed}	= -191.829 kN
M_{Ed}	= 2513.980 kN · m
V_p	= 0.000 kN

- Design strength of concrete

Girder :	$f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c$	= 33.333 MPa
Slab :	$f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c$	= 23.333 MPa

- Design strength of Reinforcement

Girder :	$f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar}$	= 434.783 MPa
Slab :	$f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar}$	= 434.783 MPa

- Check whether the section is cracked in bending.

f_b	=	1.336 MPa	<	f_{ctd}	=	1.900 MPa
(The section is uncracked.)						
where, f_b	:	Flexural tensile stress	(Tension is taken as positive.)			
f_{ctd}	=	$\alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$:	Design tensile strength of concrete.		

- Total stress at composite centroid from actions on the girder alone

$\sigma_{tot} = \sigma_1 + \sigma_2$	=	8.806 MPa	(DESIGNER'S GUIDE TO EN 1992-2, 6.2.7)
where,			

σ_1	=	$(M_{non} \cdot y') / I_g$	=	4.016 MPa
σ_2	=	$P / A_g + (P \cdot e \cdot y') / I_g$	=	4.790 MPa
σ_1	:	stress at the composite centroid due to the moment acting on the girder alone		
σ_2	:	stress at the composite centroid due to prestress		

M_{non}	=	1053.474 kN · m	: Bending Moment in girder alone
y_{non}	=	538.478 mm	: Height to Centroid in girder alone
y_{com}	=	709.660 mm	: Height to Centroid in Composite section
y'	=	$y_{com} - y_{non}$	= 171.182 mm
A_g	=	385063.151 mm ²	: Area Girder alone
I_g	=	4.4903E+10 mm ⁴	: Moment of inertia in Girder alone
P	=	3902.364 kN	: A factored prestressing force after all losses

$$P \cdot e = -1401.914 \text{ kN} \cdot \text{m} \quad : \text{Moment of prestressing force}$$

- Shear stress in girder alone at the height of the composite centroid

$$\tau_s = (V_{c1} / b_w) (Q_n / I_g) = 0.015 \text{ MPa}$$

where,

$$\begin{aligned} V_{c1} &= 4.631 \text{ kN} & : \text{Shear force on the precast girder alone} \\ Q_n &= 2.7186\text{E}+07 \text{ mm}^3 & : \text{First moment of Area in centroid of Girder alone} \\ I_g &= 4.4903\text{E}+10 \text{ mm}^4 & : \text{Moment of inertia in Girder alone} \\ b_w &= 190.000 \text{ mm} & : \text{Girder width} \end{aligned}$$

- Additional shear force acting on the composite section

$$V_{c2} = (I_c \cdot b_w / Q_c) \cdot \{ \sqrt{((f_{ctd})^2 + f_{ctd} \cdot \sigma_{tot})} - \tau_s \} = 681.304 \text{ kN}$$

where,

$$\begin{aligned} Q_c &= 9.9086\text{E}+07 \text{ mm}^3 & : \text{First moment of Area in centroid of Composite section} \\ I_c &= 7.9035\text{E}+10 \text{ mm}^4 & : \text{Moment of inertia in Composite section} \\ b_w &= 190.000 \text{ mm} & : \text{Girder width} \end{aligned}$$

- Total shear resistance of the composite section (regions Un-cracked in bending)

$$V_{Rd,c} = V_{c1} + V_{c2} = 685.936 \text{ kN} > V_{Ed} = -191.829 \text{ kN} \quad \text{OK}$$

∴ No shear reinforcement is required

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned} \rho_{w, \min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\ \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00595 \geq \rho_{w, \min} = 0.00113 \quad \text{OK} \\ \text{where, } \rho_{w, \min} &\text{ is the minimum ratio of shear reinforcement} \\ A_{sw} &= 226.200 \text{ mm}^2 & : \text{Area of diagonal reinforcement} \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned} S_{l, \max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 809.997 \text{ mm} \\ s &= 200.000 \text{ mm} \leq S_{l, \max} = 809.997 \text{ mm} \quad \text{OK} \\ \text{where, } s &\text{ is the spacing of the stirrups} \\ d &= 1079.996 \text{ mm} & : \text{effective depth} \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1;2004, 6.2.3.(7))

$$\begin{aligned} \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 95.915 \text{ kN} \\ (M_{Rd} - M_{Ed}) / z &= 2328.558 \text{ kN} \\ \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\ \text{where, } M_{Rd} &= 4509.330 \text{ kN} & : \text{The moment resistance} \\ z &= 856.904 \text{ mm} & : \text{Lever arm from the ultimate bending resistance analysis} \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1;2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned} V_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.187 \text{ MPa} \\ \text{where, } \beta &= 1.0 & : \text{Conservatively adopted to take } \beta = 1.0 \\ V_{Ed} &= -191.829 \text{ kN} & : \text{Transverse shear force} \\ z &= 856.904 \text{ mm} & : \text{Lever arm from the ultimate bending resistance analysis} \\ b_i &= 1200.000 \text{ mm} & : \text{Width of interface} \end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
 V_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) &= & 1.448 \text{ MPa} \\
 V_{Rd,lim} &= 0.5 v f_{cd} &= & 8.000 \text{ MPa} \\
 V_{Edi} &= 0.187 \text{ MPa} \leq V_{Rd} = \min[V_{Rdi}, V_{Rd,lim}] &= & 1.448 \text{ MPa} \quad \text{OK}
 \end{aligned}$$

where,

$$\begin{aligned}
 f_{ctd} &= 1.900 \text{ MPa} && \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} && \text{of concrete between girder and slab} \\
 c &= 0.400 && \text{: factors which depend on the roughness of the interface} \\
 \mu &= 0.700 && \text{: factors which depend on the roughness of the interface} \\
 \sigma_n &= w_s / A_i = 0.005 \text{ MPa} \\
 \rho &= A_s / A_i = 0.002 \\
 w_s &= 5.525 \text{ kN} && \text{: Weight of slab} \\
 A_i &= 1200000.000 \text{ mm}^2 && \text{: Area of joint (1m for unit length)} \\
 A_s &= 2700.000 \text{ mm}^2 && \text{: Area of reinforcement crossing the interface} \\
 f_{yd} &= 434.783 \text{ MPa} && \text{: Strength of shear connector} \\
 \alpha &= 90.000^\circ && \text{: Angle of reinforcement crossing the interface} \\
 &&& (45^\circ \leq \alpha \leq 90^\circ)
 \end{aligned}$$

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

$$\begin{aligned}
 \text{Load Combination Name :} & \text{ cLCB1} \\
 \text{Design Situations :} & \text{ Persistent \& Transient} \\
 \text{Load Combination Type :} & \text{ FZ-MAX} \\
 N_{Ed} &= -3765.824 \text{ kN} \\
 V_{Ed} &= 245.059 \text{ kN} \\
 T_{Ed} &= 2.140 \text{ kN} \cdot \text{m} \\
 M_{Ed} &= 2834.381 \text{ kN} \cdot \text{m}
 \end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u &= & 61.4 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} &= & 80.0 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) &= & 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) &= & 19.3 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} &= & 0.454 \text{ kN} \cdot \text{m} \\
 I_{xx_i} &= 1.1876\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 5.7976\text{E}+09 \text{ mm}^4 \\
 u &= 2683.8 \text{ mm} \\
 A &= 164900 \text{ mm}^2 \\
 u_i &= 2367.0 \text{ mm} \\
 A_{ki} &= 63884 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 190.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 61.331 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.200 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 6.668 \text{ MPa} \\ N_{Ed} &= 3755.824 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 564779 \text{ mm}^2 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 64.4 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 6.9 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.246 \text{ kN} \cdot \text{m} \\ I_{xx,i} &= 6.4482\text{E}+08 \text{ mm}^4 \\ I_{xx} &= 5.7976\text{E}+09 \text{ mm}^4 \\ u &= 1180.0 \text{ mm} \\ A &= 76000 \text{ mm}^2 \\ u_i &= 860.0 \text{ mm} \\ A_{ki} &= 35200 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 190.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 54.069 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.200 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 6.668 \text{ MPa} \\ N_{Ed} &= 3755.824 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 564779 \text{ mm}^2 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 82.7 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \end{aligned}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 13.6 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.690 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 1.8065\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 5.7976\text{E}+09 \text{ mm}^4$$

$$u = 1473.3 \text{ mm}$$

$$A = 121850 \text{ mm}^2$$

$$u_i = 1153.9 \text{ mm}$$

$$A_{ki} = 67529 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin \theta \cdot \cos \theta = 77.795 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.200 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.668 \text{ MPa}$$

$$N_{Ed} = 3755.824 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 564779 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.163 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 1549.324 \text{ kN}$$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$$N_{Ed} = -3776.870 \text{ kN}$$

$$V_{Ed} = -191.829 \text{ kN}$$

$$T_{Ed} = -0.443 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 2513.980 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 61.4 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 4.0 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.094 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.1876\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 5.7976\text{E}+09 \text{ mm}^4$$

$$u = 2683.8 \text{ mm}$$

$$A = 164900 \text{ mm}^2$$

$$u_i = 2367.0 \text{ mm}$$

$$A_{ki} = 63884 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 61.361 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.201 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.687 \text{ MPa}$$

$$N_{Ed} = 3776.870 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 564779 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 64.4 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 1.4 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.051 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 6.4482\text{E}+08 \text{ mm}^4$$

$$I_{xx} = 5.7976\text{E}+09 \text{ mm}^4$$

$$u = 1180.0 \text{ mm}$$

$$A = 76000 \text{ mm}^2$$

$$u_i = 860.0 \text{ mm}$$

$$A_{ki} = 35200 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 54.095 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.201 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 6.687 \text{ MPa} \\
N_{Ed} &= 3776.870 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 564779 \text{ mm}^2
\end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 82.7 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 2.8 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.143 \text{ kN} \cdot \text{m}
\end{aligned}$$

$$I_{xx,i} = 1.8065\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 5.7976\text{E}+09 \text{ mm}^4$$

$$u = 1473.3 \text{ mm}$$

$$A = 121850 \text{ mm}^2$$

$$u_i = 1153.9 \text{ mm}$$

$$A_{ki} = 67529 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 77.833 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.201 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 6.687 \text{ MPa} \\
N_{Ed} &= 3776.870 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 564779 \text{ mm}^2
\end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.125 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 1550.048 \text{ kN}$$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MX-MIN

$$N_{Ed} = -3845.004 \text{ kN}$$

$$V_{Ed} = 33.612 \text{ kN}$$

$$T_{Ed} = -17.065 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 2303.889 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 61.4 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 154.1 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 3.617 \text{ kN} \cdot \text{m} \\ I_{xx_i} &= 1.1876\text{E}+09 \text{ mm}^4 \\ I_{xx} &= 5.7976\text{E}+09 \text{ mm}^4 \\ u &= 2683.8 \text{ mm} \\ A &= 164900 \text{ mm}^2 \\ u_i &= 2367.0 \text{ mm} \\ A_{ki} &= 63884 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 190.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned} T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 61.546 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\ \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.204 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 6.808 \text{ MPa} \\ N_{Ed} &= 3845.004 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 564779 \text{ mm}^2 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 64.4 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 55.2 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.964 \text{ kN} \cdot \text{m} \\ I_{xx_i} &= 6.4482\text{E}+08 \text{ mm}^4 \\ I_{xx} &= 5.7976\text{E}+09 \text{ mm}^4 \\ u &= 1180.0 \text{ mm} \\ A &= 76000 \text{ mm}^2 \\ u_i &= 860.0 \text{ mm} \\ A_{ki} &= 35200 \text{ mm}^2 \end{aligned}$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 54.258 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.204 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.808 \text{ MPa}$$

$$N_{Ed} = 3845.004 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 564779 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 82.7 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.0 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 108.1 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 5.502 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 1.8065\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 5.7976\text{E}+09 \text{ mm}^4$$

$$u = 1473.3 \text{ mm}$$

$$A = 121850 \text{ mm}^2$$

$$u_i = 1153.9 \text{ mm}$$

$$A_{ki} = 67529 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 190.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 78.068 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.204 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.808 \text{ MPa}$$

$$N_{Ed} = 3845.004 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 564779 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.062 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 1554.514 \text{ kN}$$

5. Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB5
Design Situations : Quasi-permanent
Load Combination Type : -
Long/Short Term : Short-term
 $N_{Ed} = -3926.223 \text{ kN}$
 $M_{Ed} = -241.688 \text{ kN} \cdot \text{m}$

- Stress check (-: compression + : tension)

$$\sigma = -6.029 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4
Design Situations : Frequent
Load Combination Type : MY-MAX
Long/Short Term : Short-term
 $N_{Ed} = -3833.646 \text{ kN}$
 $M_{Ed} = 566.080 \text{ kN} \cdot \text{m}$

- Stress check (-: compression + : tension)

$$\sigma = -6.198 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

6. Crack Width

■ Top

1 Calculate crack width, w_k

(EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4
Serviceability Load Type : Frequent
Load Combination Type : MY-MIN
Long/Short Term : Short-term
 $N_{Ed} = -3927.331 \text{ kN}$
 $M_{Ed} = -252.185 \text{ kN} \cdot \text{m}$

- Stress at top surface

$$\sigma_{top} = -5.998 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
1.78	23.71	1.79	23.71	1.78	23.71	23.71	31.38	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f_{ci}$$

$$f_{ci} = 44.82176 \text{ (MPa)}$$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
0.14	0.79	0.13	0.78	0.14	0.79	0.79	24.50	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_{ci}$

$f_{ci} = 35$ (MPa)

8.Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MIN

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
5.99	14.72	5.98	14.72	6.00	14.73	14.73	30.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f'_c$

$f'_c = 42.85714$ (MPa)



2) Slab

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
19.52	5.46	19.52	5.46	22.32	20.28	19.52	21.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f'_c$

$f'_c = 30$ (MPa)

intocmit ing. Attila Takacs



Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=21.00m, h=1.05m

Element Number	90
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

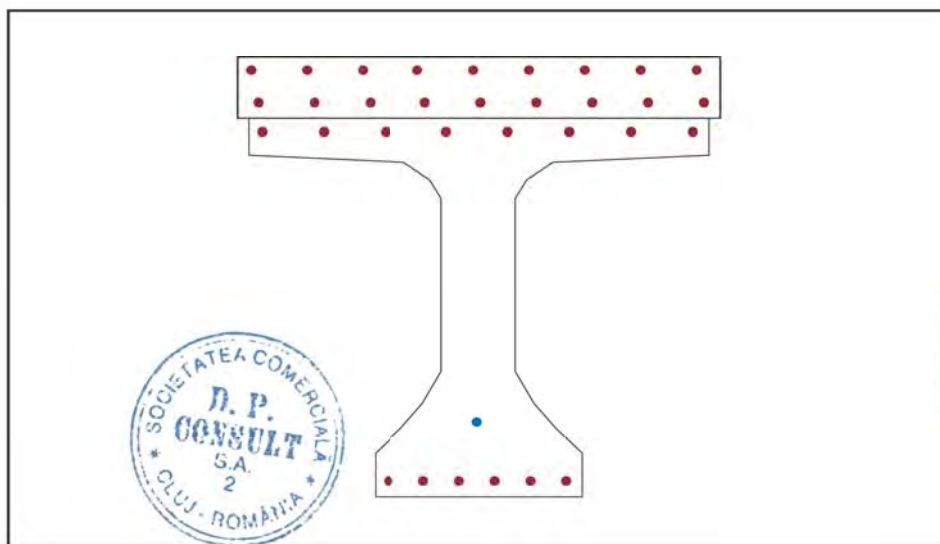
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

$\alpha_{cc} = 0.850$ (for the Compressive strength)

$\alpha_{ct} = 1.000$ (for the Tensile strength)

1.2 Sectional Information

Section Information	EquvInt sect(tdn, reinfmnt) (Girder)	EquvInt sect (After Comp) (Girder + Slab)
A (mm ²)	434728.441	645819.382
I_y (mm ⁴)	62533523197.998	107740229817.076
y_{st} (mm)	-	467.012
y_{sb} (mm)	-	297.012
y_t (mm)	475.802	297.012
y_b (mm)	574.198	752.988
Z_{st} (mm ³)	-	96381528.059
Z_{sb} (mm ³)	-	141836916.287
Z_t (mm ³)	34163957.654	141836916.287
Z_b (mm ³)	75648763.377	74145930.438



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(g)} = 50.000 \text{ MPa}$$

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Tendon-Span1	1015.0	3360.000	1863.260	1569.060	195000.000

2. Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 3406.926 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	610.0	9709.833	1278.522	295.043	4584.384	2.25198
2	305.0	6983.033	1225.221	295.043	4584.384	1.68222
3	152.5	3145.567	630.809	688.435	4584.384	0.71620
4	228.8	4839.900	947.488	295.043	4584.384	1.18608
5	190.6	3931.958	789.660	688.435	4584.384	0.89546
6	209.7	4325.154	847.423	688.435	4584.384	0.98099
7	219.2	4572.008	899.186	295.043	4584.384	1.12128
8	214.5	4438.062	873.426	295.043	4584.384	1.08855
9	212.1	4374.304	860.111	295.043	4584.384	1.07275
10	210.9	4349.729	853.341	295.043	4584.384	1.06633
11	210.3	4337.442	849.927	295.043	4584.384	1.06311
12	210.0	4331.298	848.243	688.435	4584.384	0.98231
13	210.1	4334.370	849.071	295.043	4584.384	1.06231
14	210.1	4332.834	848.642	295.043	4584.384	1.06190
15	210.0	4332.066	848.428	295.043	4584.384	1.06170
16	210.0	4331.682	848.320	295.043	4584.384	1.06160
17	210.0	4331.490	848.268	688.435	4584.384	0.98235
18	210.0	4331.586	848.293	295.043	4584.384	1.06158
19	210.0	4331.538	848.280	295.043	4584.384	1.06156
20	210.0	4331.490	848.268	688.435	4584.384	0.98235

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 4331.490 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 848.268 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 688.435 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 4584.384 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 167.997 \text{ mm} \\
 x &= 209.995 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 218394.446 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 2940.600 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 678.600 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1131.729	0.01342	0.00580	0.01922	1364.400	3360.000	4584.384
$F_p = \sum (F_{pi}) =$							4584.384

where, Stress_{Eff} = $\epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0.1K} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned} M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\ &= 4595.809 \text{ kN} \cdot \text{m} \geq M_{Ed} = 3406.926 \text{ kN} \cdot \text{m} \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MAX

$N_{Ed} = -3698.612 \text{ kN}$

$V_{Ed} = 268.133 \text{ kN}$

$M_{Ed} = 3200.596 \text{ kN} \cdot \text{m}$

$V_p = 0.000 \text{ kN}$

- Design strength of concrete

Girder : $f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa}$

Slab : $f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}$

- Design strength of Reinforcement

Girder : $f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

Slab : $f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

- Check whether the section is cracked in bending.

$f_b = 5.092 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)

$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.

- Design value for the shear resistance $V_{Rd,c}$

$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 300.701 \text{ kN}$

$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 299.948 \text{ kN} \geq V_{Rd,min}$
 $= 300.701 \text{ kN} \geq V_{Ed} = 268.133 \text{ kN} \therefore \text{Shear reinforcement is not required}$

where, $C_{Rd,c} = 0.18 / \gamma_c = 0.120$

$v_{min} = 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.415$

$k_1 = 0.15$

$k = 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.412$

$\rho_1 = A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003$

$A_{sl} = 678.600 \text{ mm}^2$

$b_w = 200.0 \text{ mm}$

$d_p = 1180.0 \text{ mm}$

$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 5.727 \text{ MPa}$

$N_{Ed} = 3698.612 \text{ kN}$ (if compression, $N_{Ed} > 0$)

$A_c = 645819.382 \text{ mm}^2$

2) Check minimum shear reinforcement

(EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w, \min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00566 \geq \rho_{w, \min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w, \min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 226.200 \text{ mm}^2 : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l, \max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 884.996 \text{ mm}$$

$$s = 200.000 \text{ mm} \leq S_{l, \max} = 884.996 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 1179.995 \text{ mm} : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 134.067 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 1572.500 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z \quad \text{OK}$$

where, $M_{Rd} = 4595.809 \text{ kN} : \text{The moment resistance}$

$$z = 887.258 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

4) Check shear at the interface between concrete cast at different times

(EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.244 \text{ MPa}$$

where, $\beta = 1.0 : \text{Conservatively adopted to take } \beta = 1.0$

$$V_{Ed} = 268.133 \text{ kN} : \text{Transverse shear force}$$

$$z = 887.258 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

$$b_i = 1240.000 \text{ mm} : \text{Width of interface}$$

- Design shear resistance at the interface

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.426 \text{ MPa}$$

$$v_{Rd, \lim} = 0.5 v f_{cd} = 8.000 \text{ MPa}$$

$$v_{Edi} = 0.244 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, \lim}] = 1.426 \text{ MPa} \quad \text{OK}$$

where, $f_{ctd} = 1.900 \text{ MPa} : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd})$

$$f_{cd} = 33.333 \text{ MPa} : \text{of concrete between girder and slab}$$

$$c = 0.400 : \text{factors which depend on the roughness of the interface}$$

$$\mu = 0.700 : \text{factors which depend on the roughness of the interface}$$

$$\sigma_n = w_s / A_i = 0.004 \text{ MPa}$$

$$\rho = A_s / A_i = 0.002$$

$$w_s = 5.525 \text{ kN} : \text{Weight of slab}$$

$$A_i = 1240000.000 \text{ mm}^2 : \text{Area of joint (1m for unit length)}$$

$$A_s = 2700.000 \text{ mm}^2 : \text{Area of reinforcement crossing the interface}$$

$$f_{yd} = 434.783 \text{ MPa} : \text{Strength of shear connector}$$

$$\alpha = 90.000^\circ : \text{Angle of reinforcement crossing the interface} \\ (45^\circ \leq \alpha \leq 90^\circ)$$

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd, c}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$$N_{Ed} = -3711.618 \text{ kN}$$

$$V_{Ed} = -205.945 \text{ kN}$$

$$M_{Ed} = 2882.886 \text{ kN} \cdot \text{m}$$

$$V_p = 0.000 \text{ kN}$$

- Design strength of concrete

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}$$

- Design strength of Reinforcement

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Check whether the section is cracked in bending.

$$f_b = 2.622 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)

$$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c \quad : \text{Design tensile strength of concrete.}$$

- Design value for the shear resistance $V_{Rd,c}$

$$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 301.414 \text{ kN}$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 300.661 \text{ kN} \geq V_{Rd,min}$$

$= 301.414 \text{ kN} \geq V_{Ed} = -205.945 \text{ kN} \quad \therefore \text{Shear reinforcement is not required}$

$$\text{where, } C_{Rd,c} = 0.18 / \gamma_c = 0.120$$

$$v_{min} = 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.415$$

$$k_1 = 0.15$$

$$k = 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.412$$

$$\rho_1 = A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003$$

$$A_{sl} = 678.600 \text{ mm}^2$$

$$b_w = 200.0 \text{ mm}$$

$$d_p = 1180.0 \text{ mm}$$

$$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 5.747 \text{ MPa}$$

$$N_{Ed} = 3711.618 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 645819.382 \text{ mm}^2$$

2) Check minimum shear reinforcement

(EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w,min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00566 \geq \rho_{w,min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w,min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 226.200 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l,max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 884.996 \text{ mm}$$

$$s = 200.000 \text{ mm} \leq S_{l,max} = 884.996 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 1179.995 \text{ mm} \quad : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1;2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 102.973 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 1930.580 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z$$

OK

where, $M_{Rd} = 4595.809 \text{ kN}$: The moment resistance

$z = 887.258 \text{ mm}$: Lever arm from the ultimate bending resistance analysis

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1;2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.187 \text{ MPa}$$

where, $\beta = 1.0$: Conservatively adopted to take $\beta = 1.0$

$V_{Ed} = -205.945 \text{ kN}$: Transverse shear force

$z = 887.258 \text{ mm}$: Lever arm from the ultimate bending resistance analysis

$b_i = 1240.000 \text{ mm}$: Width of interface

- Design shear resistance at the interface

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.426 \text{ MPa}$$

$$v_{Rd,lim} = 0.5 v f_{cd} = 8.000 \text{ MPa}$$

$$v_{Edi} = 0.187 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd,lim}] = 1.426 \text{ MPa} \quad \text{OK}$$

where, $f_{ctd} = 1.900 \text{ MPa}$: Minimum design tensile strength(f_{ctd}) and design strength(f_{cd})

$f_{cd} = 33.333 \text{ MPa}$ of concrete between girder and slab

$c = 0.400$: factors which depend on the roughness of the interface

$\mu = 0.700$: factors which depend on the roughness of the interface

$\sigma_n = w_s / A_i = 0.004 \text{ MPa}$

$\rho = A_s / A_i = 0.002$

$w_s = 5.525 \text{ kN}$: Weight of slab

$A_i = 1240000.000 \text{ mm}^2$: Area of joint (1m for unit length)

$A_s = 2700.000 \text{ mm}^2$: Area of reinforcement crossing the interface

$f_{yd} = 434.783 \text{ MPa}$: Strength of shear connector

$\alpha = 90.000^\circ$: Angle of reinforcement crossing the interface
($45^\circ \leq \alpha \leq 90^\circ$)

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MAX

$N_{Ed} = -3698.612 \text{ kN}$

$V_{Ed} = 268.133 \text{ kN}$

$T_{Ed} = 7.608 \text{ kN} \cdot \text{m}$

$M_{Ed} = 3200.596 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u &= & 60.8 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} &= & 80.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) &= & 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) &= & 57.0 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} &= & 1.298 \text{ kN} \cdot \text{m} \\
 I_{xx,i} &= 1.1760\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\
 u &= 2769.6 \text{ mm} \\
 A &= 168400 \text{ mm}^2 \\
 u_i &= 2446.9 \text{ mm} \\
 A_{ki} &= 64071 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
 T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta &= & 60.495 \text{ kN} \cdot \text{m} \geq T_{Ed,i} && \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) &= & 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} &= & 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c &= & 6.008 \text{ MPa} \\
 N_{Ed} &= 3698.612 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 615629 \text{ mm}^2
 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u &= & 70.6 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} &= & 80.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) &= & 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) &= & 26.1 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} &= & 1.048 \text{ kN} \cdot \text{m} \\
 I_{xx,i} &= 9.4938\text{E}+08 \text{ mm}^4 \\
 I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\
 u &= 1360.0 \text{ mm} \\
 A &= 96000 \text{ mm}^2 \\
 u_i &= 1040.0 \text{ mm} \\
 A_{ki} &= 48000 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 72.514 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.008 \text{ MPa}$$

$$N_{Ed} = 3698.612 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 615629 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 93.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 53.2 \text{ mm}^2/\text{mm}$$

where,

$$T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 3.093 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 2.8021\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 7.4262\text{E}+09 \text{ mm}^4$$

$$u = 1588.3 \text{ mm}$$

$$A = 149200 \text{ mm}^2$$

$$u_i = 1238.3 \text{ mm}$$

$$A_{ki} = 82820 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 93.837 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.008 \text{ MPa}$$

$$N_{Ed} = 3698.612 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 615629 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.178 \leq 1.0 \quad \text{OK}$$

where,

$$V_{Rd,max} = 1663.517 \text{ kN}$$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$$N_{Ed} = -3711.618 \text{ kN}$$

$$\begin{aligned}
 V_{Ed} &= -205.945 \text{ kN} \\
 T_{Ed} &= -3.976 \text{ kN} \cdot \text{m} \\
 M_{Ed} &= 2882.886 \text{ kN} \cdot \text{m}
 \end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 60.8 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\
 A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 29.8 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.678 \text{ kN} \cdot \text{m} \\
 I_{xx_i} &= 1.1760\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\
 u &= 2769.6 \text{ mm} \\
 A &= 168400 \text{ mm}^2 \\
 u_i &= 2446.9 \text{ mm} \\
 A_{ki} &= 64071 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
 T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 60.527 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.181 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 6.029 \text{ MPa} \\
 N_{Ed} &= 3711.618 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 615629 \text{ mm}^2
 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 70.6 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\
 A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 13.6 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.548 \text{ kN} \cdot \text{m} \\
 I_{xx_i} &= 9.4938\text{E}+08 \text{ mm}^4 \\
 I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4
 \end{aligned}$$

$$\begin{aligned}
 u &= 1360.0 \text{ mm} \\
 A &= 96000 \text{ mm}^2 \\
 u_i &= 1040.0 \text{ mm} \\
 A_{ki} &= 48000 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 72.553 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.181 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 6.029 \text{ MPa} \\
 N_{Ed} &= 3711.618 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 615629 \text{ mm}^2
 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 93.9 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 27.8 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.616 \text{ kN} \cdot \text{m}
 \end{aligned}$$

$$\begin{aligned}
 I_{xx,i} &= 2.8021\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\
 u &= 1588.3 \text{ mm} \\
 A &= 149200 \text{ mm}^2 \\
 u_i &= 1238.3 \text{ mm} \\
 A_{ki} &= 82820 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 93.888 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.181 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 6.029 \text{ MPa} \\
 N_{Ed} &= 3711.618 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 615629 \text{ mm}^2
 \end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.133 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 1664.375 \text{ kN}$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB1
 Design Situations : Persistent & Transient
 Load Combination Type : MX-MAX
 $N_{Ed} = -3698.362 \text{ kN}$
 $V_{Ed} = 154.212 \text{ kN}$
 $T_{Ed} = 21.994 \text{ kN} \cdot \text{m}$
 $M_{Ed} = 2986.120 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 60.8 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 164.8 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 3.753 \text{ kN} \cdot \text{m} \\ I_{xx_i} &= 1.1760\text{E}+09 \text{ mm}^4 \\ I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\ u &= 2769.6 \text{ mm} \\ A &= 168400 \text{ mm}^2 \\ u_i &= 2446.9 \text{ mm} \\ A_{ki} &= 64071 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 60.494 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$
 $\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$
 $\sigma_{cp} = N_{Ed} / A_c = 6.007 \text{ MPa}$
 $N_{Ed} = 3698.362 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$

$$A_c = 615629 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 70.6 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 75.5 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 3.030 \text{ kN} \cdot \text{m} \\ I_{xx,i} &= 9.4938\text{E}+08 \text{ mm}^4 \\ I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\ u &= 1360.0 \text{ mm} \\ A &= 96000 \text{ mm}^2 \\ u_i &= 1040.0 \text{ mm} \\ A_{ki} &= 48000 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned} T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 72.513 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\ \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 6.007 \text{ MPa} \\ N_{Ed} &= 3698.362 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 615629 \text{ mm}^2 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 93.9 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 80.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 153.8 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 8.943 \text{ kN} \cdot \text{m} \\ I_{xx,i} &= 2.8021\text{E}+09 \text{ mm}^4 \\ I_{xx} &= 7.4262\text{E}+09 \text{ mm}^4 \\ u &= 1588.3 \text{ mm} \\ A &= 149200 \text{ mm}^2 \\ u_i &= 1238.3 \text{ mm} \\ A_{ki} &= 82820 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 93.837 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.180 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.007 \text{ MPa}$$

$$N_{Ed} = 3698.362 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 615629 \text{ mm}^2$$

- Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.141 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 1663.501 \text{ kN}$

5. Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB5

Design Situations : Quasi-permanent

Load Combination Type : -

Long/Short Term : Short-term

$$N_{Ed} = -3831.224 \text{ kN}$$

$$M_{Ed} = -182.978 \text{ kN} \cdot \text{m}$$

- Stress check (-: compression + : tension)

$$\sigma = -5.744 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4

Design Situations : Frequent

Load Combination Type : MY-MAX

Long/Short Term : Short-term

$$N_{Ed} = -3752.458 \text{ kN}$$

$$M_{Ed} = 754.444 \text{ kN} \cdot \text{m}$$

- Stress check (-: compression + : tension)

$$\sigma = -4.315 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

6. Crack Width

■ Top

- 1 Calculate crack width, w_k (EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4

Serviceability Load Type : Frequent

Load Combination Type : MY-MIN

Long/Short Term : Short-term

$$N_{Ed} = -3831.706 \text{ kN}$$

$$M_{Ed} = -195.653 \text{ kN} \cdot \text{m}$$

- Stress at top surface

$$\sigma_{top} = -5.708 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
2.23	18.69	2.20	18.68	2.26	18.71	18.71	31.38	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f_{ci}$$

$$f_{ci} = 44.82176 \text{ (MPa)}$$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
0.27	0.72	0.26	0.71	0.28	0.72	0.72	24.50	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f_{ci}$$

$$f_{ci} = 35 \text{ (MPa)}$$

8. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MIN

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
5.70	11.80	5.68	11.79	5.72	11.81	11.81	30.00	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f_c$$

$$f_c = 42.85714 \text{ (MPa)}$$

2) Slab

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
20.88	5.36	20.87	5.36	20.85	19.07	20.88	21.00	OK

For post-tensioned elements $k_6 = 0.70$

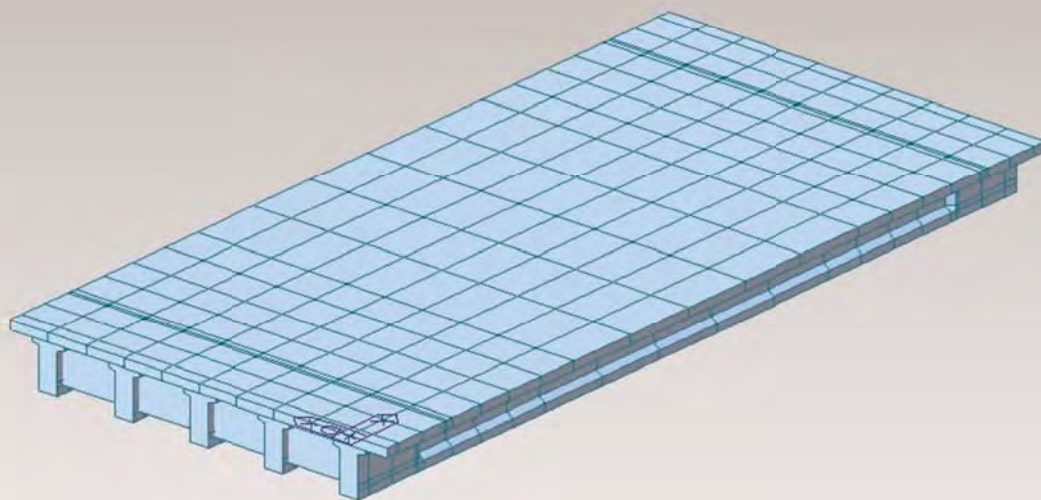
$$ALW = \text{Limiting stress} = k_6 \cdot f_c$$

$$f_c = 30 \text{ (MPa)}$$

intocmit ing. Attila Takacs

Proiectare si executie "Varianta de Ocolire Timisoara Sud"

Grinda L=23.50 m, h=1.30 m



Modelare 3D



Element Number	57
Position Information	J

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

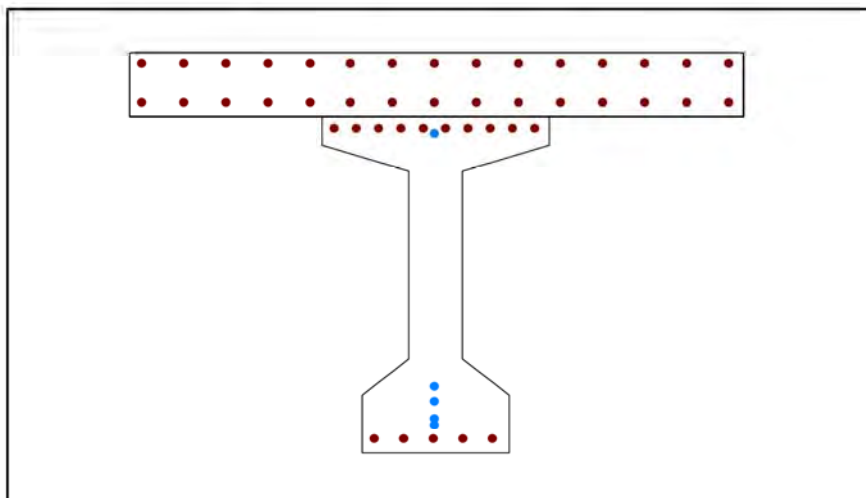
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

$\alpha_{cc} = 0.850$ (for the Compressive strength)

$\alpha_{ct} = 1.000$ (for the Tensile strength)

1.2 Sectional Information

Section Information	Equvnt sect(tdn, reinfmnt) (Girder)	Equvnt sect (After Comp) (Girder + Slab)
A (mm ²)	492854.073	1033598.779
I_y (mm ⁴)	104369343655.819	261205791132.254
y_{st} (mm)	-	493.525
y_{sb} (mm)	-	243.525
y_t (mm)	647.859	243.525
y_b (mm)	652.141	1056.475
Z_{st} (mm ³)	-	256414797.101
Z_{sb} (mm ³)	-	344510247.512
Z_t (mm ³)	28244707.208	344510247.512
Z_b (mm ³)	43649540.037	438005000.400



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$f_{ck(g)} = 50.000$ MPa

$E_{c(g)} = 37277.000$ MPa

$f_{cm} = f_{ck} + 8 = 58.000$ MPa

$f_{ctm} = 0.30 \cdot f_{ck}^{(2/3)} = 4.064$ MPa ($f_{ck} \leq C50/60$)

$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845$ MPa

- Reinforcement Material Information

$f_{yk(g)} = 500.000$ MPa , $E_{s(g)} = 200000.000$ MPa

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$\begin{aligned}
 f_{ck(s)} &= 35.000 \text{ MPa} \\
 E_{c(s)} &= 33282.000 \text{ MPa} \\
 f_{cm} &= f_{ck} + 8 = 43.000 \text{ MPa} \\
 f_{ctm} &= 0.30 \cdot f_{ck}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60) \\
 f_{ctk;0.05} &= 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}
 \end{aligned}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	S_CABLE 3-Copy	1420.000	840.000	1863.260	1569.060	195000.000
2	S_CABLE 5-Copy	320.000	280.000	1863.260	1569.060	195000.000
3	S_CABLE 4-Copy	1445.000	1680.000	1863.260	1569.060	195000.000
4	S_CABLE 1-Copy	1295.000	560.000	1863.260	1569.060	195000.000
5	S_CABLE 2-Copy	1353.000	840.000	1863.260	1569.060	195000.000

2.Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : SLU
 Design Situations : Persistent & Transient
 Load Combination Type : FX-MAX
 M_{Ed} = 6173.410 kN · m

- factor λ, and factor η

$$\begin{aligned}
 \lambda_{(g)} &= 0.800 \quad (f_{ck} \leq 50 \text{ MPa}) \\
 \lambda_{(s)} &= 0.800 \quad (f_{ck} \leq 50 \text{ MPa}) \\
 \eta_{(g)} &= 1.000 \quad (f_{ck} \leq 50 \text{ MPa}) \\
 \eta_{(s)} &= 1.000 \quad (f_{ck} \leq 50 \text{ MPa})
 \end{aligned}$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\begin{aligned}
 \text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
 \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
 \end{aligned}$$

- Calculate Neutral Axis

- 1) Assume neutral axis depth.
- 2) Calculate the strain of steel and tendon.
- 3) Calculate the stress of steel and tendon.
- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	775.000	16447.500	1966.957	288.478	5348.448	3.26675

2	387.500	12849.167	1787.741	288.478	5348.448	2.59661
3	193.750	7070.583	737.609	1517.826	5730.480	1.07724
4	96.875	3535.292	574.669	1517.826	5730.480	0.56702
5	145.313	5302.937	737.609	1517.826	5730.480	0.83337
6	169.531	6186.760	737.609	1517.826	5730.480	0.95531
7	181.641	6628.672	737.609	1517.826	5730.480	1.01628
8	175.586	6407.716	737.609	1517.826	5730.480	0.98579
9	178.613	6518.194	737.609	1517.826	5730.480	1.00103
10	177.100	6462.955	737.609	1517.826	5730.480	0.99341
11	177.856	6490.575	737.609	1517.826	5730.480	0.99722
12	178.235	6504.384	737.609	1517.826	5730.480	0.99913

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p

(EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && \text{: Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 6504.384 \text{ kN} && \text{: Compression Force for Slab} \\
 F_s &= f_s \cdot A_s = 737.609 \text{ kN} && \text{: Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1517.826 \text{ kN} && \text{: Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 5730.480 \text{ kN} && \text{: Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 142.588 \text{ mm} \\
 x &= 178.235 \text{ mm} && \text{: neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 327952.148 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for slab part} \\
 A_s &= 1696.500 \text{ mm}^2 && \text{: Compression reinforcement areas} \\
 A_s &= 3491.000 \text{ mm}^2 && \text{: Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
S_CABLE 3-Copy	1337.320	0.02438	0.00686	0.03124	1364.400	840.000	1146.096
S_CABLE 5-Copy	1380.280	0.00278	0.00708	0.00986	1364.400	280.000	382.032
S_CABLE 4-Copy	1339.105	0.02488	0.00687	0.03174	1364.400	1680.000	2292.192
S_CABLE 1-Copy	1341.864	0.02193	0.00688	0.02881	1364.400	560.000	764.064
S_CABLE 2-Copy	1340.055	0.02307	0.00687	0.02994	1364.400	840.000	1146.096
$F_p = \sum (F_{pi}) =$							5730.480

where, $\text{StressEff} = \epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0.1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 7828.098 \text{ kN} \cdot \text{m} \geq M_{Ed} = 6173.410 \text{ kN} \cdot \text{m}
 \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB9

Design Situations : Persistent & Transient

Load Combination Type : FX-MIN

$$M_{Ed} = -1341.973 \text{ kN} \cdot \text{m}$$

- factor λ , and factor η

$$\begin{aligned}\lambda_{(g)} &= 0.800 & (f_{ck} \leq 50 \text{ MPa}) \\ \lambda_{(s)} &= 0.800 & (f_{ck} \leq 50 \text{ MPa}) \\ \eta_{(g)} &= 1.000 & (f_{ck} \leq 50 \text{ MPa}) \\ \eta_{(s)} &= 1.000 & (f_{ck} \leq 50 \text{ MPa})\end{aligned}$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\begin{aligned}\text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa} \\ \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}\end{aligned}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\begin{aligned}\text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\ \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}\end{aligned}$$

- Calculate Neutral Axis

- 1) Assume neutral axis depth.
- 2) Calculate the strain of steel and tendon.
- 3) Calculate the stress of steel and tendon.
- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	775.000	6438.750	288.478	1966.957	382.032	2.86388
2	387.500	4574.948	288.478	1966.957	382.032	2.07043
3	193.750	2415.417	288.478	1966.957	2281.359	0.63646
4	290.625	3621.133	288.478	1966.957	382.032	1.66438
5	242.188	3019.271	288.478	1966.957	1146.096	1.06254
6	217.969	2717.344	288.478	1966.957	1146.096	0.96555
7	230.078	2868.307	288.478	1966.957	1146.096	1.01405
8	224.023	2792.826	288.478	1966.957	1146.096	0.98980
9	227.051	2830.566	288.478	1966.957	1146.096	1.00192
10	225.537	2811.696	288.478	1966.957	1146.096	0.99586
11	226.294	2821.131	288.478	1966.957	1146.096	0.99889
12	226.672	2825.849	288.478	1966.957	1146.096	1.00041

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p

(EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 2825.849 \text{ kN} & : \text{Compression Force for Girder} \\ F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN} & : \text{Compression Force for Slab} \\ F_{s'} &= f_{s'} \cdot A_{s'} = 288.478 \text{ kN} & : \text{Compression reinforcement Force} \\ F_s &= f_s \cdot A_s = 1966.957 \text{ kN} & : \text{Tension Force in reinforcement} \\ F_p &= \sum F_{pi} = 1146.096 \text{ kN} & : \text{Tension Force in Tendon}\end{aligned}$$

$$\text{where, } a = \lambda \cdot x = 181.338 \text{ mm}$$

$$\begin{aligned}x &= 226.672 \text{ mm} & : \text{neutral axis depth} \\ A_{ca(g)} &= 99735.840 \text{ mm}^2 & : \text{Area of rectangular con'c stress block for girder part} \\ A_{ca(s)} &= 0.000 \text{ mm}^2 & : \text{Area of rectangular con'c stress block for slab part} \\ A_{s'} &= 663.500 \text{ mm}^2 & : \text{Compression reinforcement areas} \\ A_s &= 4524.000 \text{ mm}^2 & : \text{Tension reinforcement areas}\end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
-----	-----------------------------	------------------------	--------------------	-----------------	----------------	-----------------------------	---------------

S_CABLE 5-Copy	1380.280	0.01549	0.00708	0.02257	1364.400	280.000	382.032
S_CABLE 1-Copy	1341.864	0.00044	0.00688	0.00732	1364.400	560.000	764.064
					$F_p = \sum (F_{pi}) =$		1146.096

where, $\text{StressEff} = \epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned} M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\ &= 3111.264 \text{ kN} \cdot \text{m} \geq M_{Ed} = -1341.973 \text{ kN} \cdot \text{m} \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name : cLCB2

Design Situations : Persistent & Transient

Load Combination Type : FX-MAX

$$N_{Ed} = -11336.898 \text{ kN}$$

$$V_{Ed} = 399.345 \text{ kN}$$

$$M_{Ed} = 4715.858 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}$$

- Design strength of Reinforcement

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Checking shear resistance of the regions cracked or uncracked in bending

$$f_b = -9.897 \text{ MPa} < f_{ctd} = 1.900 \text{ MPa}$$

(Check the shear strength for noncrack section.)

where, f_b : Flexural tensile stress (Tension takes positive)

$f_{ctd} = \alpha_{ct} \cdot f_{dk,0.05} / \gamma_c$: Design tensile strength of concrete.

- Total stress at composite centroid from actions on the girder alone

$$\sigma_{tot} = \sigma_1 + \sigma_2 = 8.440 \text{ MPa} \quad (\text{DESIGNER'S GUIDE TO EN 1992-2, 6.2.7})$$

where,

$$\sigma_1 = (M_{non} \cdot y') / I_g = 6.320 \text{ MPa}$$

$$\sigma_2 = P / A_g + (P \cdot e \cdot y') / I_g = 2.120 \text{ MPa}$$

σ_1 : stress at the composite centroid due to the moment acting on the girder alone

σ_2 : stress at the composite centroid due to prestress

$$M_{non} = 1631.265 \text{ kN} \cdot \text{m} \quad : \text{Bending Moment in girder alone}$$

$$y_{non} = 652.141 \text{ mm} \quad : \text{Height to Centroid in girder alone}$$

$$y_{com} = 1056.475 \text{ mm} \quad : \text{Height to Centroid in Composite section}$$

$$\begin{aligned}
 y' &= y_{com} - y_{non} = 404.334 \text{ mm} \\
 A_g &= 492854.073 \text{ mm}^2 && : \text{Area Girder alone} \\
 I_g &= 1.0437E+11 \text{ mm}^4 && : \text{Moment of inertia in Girder alone} \\
 P &= 5636.614 \text{ kN} && : \text{A factored prestressing force after all losses} \\
 P \cdot e &= -2404.875 \text{ kN} \cdot \text{m} && : \text{Moment of prestressing force}
 \end{aligned}$$

- Shear stress in girder alone at the height of the composite centroid

$$\tau_s = (V_{c1} / b_w) (Q_n / I_g) = 0.013 \text{ MPa}$$

where,

$$\begin{aligned}
 V_{c1} &= 11.915 \text{ kN} && : \text{Shear force on the precast girder alone} \\
 Q_n &= 2.2666E+07 \text{ mm}^3 && : \text{First moment of Area in centroid of Girder alone} \\
 I_g &= 1.0437E+11 \text{ mm}^4 && : \text{Moment of inertia in Girder alone} \\
 b_w &= 200.000 \text{ mm} && : \text{Girder width}
 \end{aligned}$$

- Additional shear force acting on the composite section

$$V_{c2} = (I_c \cdot b_w / Q_c) \cdot \{ \sqrt{((f_{ctd})^2 + f_{ctd} \cdot \sigma_{tot})} - \tau_s \} = 984.277 \text{ kN}$$

where,

$$\begin{aligned}
 Q_c &= 2.3457E+08 \text{ mm}^3 && : \text{First moment of Area in centroid of Composite section} \\
 I_c &= 2.6121E+11 \text{ mm}^4 && : \text{Moment of inertia in Composite section} \\
 b_w &= 200.000 \text{ mm} && : \text{Girder width}
 \end{aligned}$$

- Total shear resistance of the composite section (regions Un-cracked in bending)

$$V_{Rd,c} = V_{c1} + V_{c2} = 996.192 \text{ kN} > V_{Ed} = 399.345 \text{ kN} \quad \text{OK}$$

∴ No shear reinforcement is required

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
 \rho_{w, \min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
 \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00283 \geq \rho_{w, \min} = 0.00113 \quad \text{OK} \\
 \text{where, } \rho_{w, \min} &\text{ is the minimum ratio of shear reinforcement} \\
 A_{sw} &= 113.000 \text{ mm}^2 && : \text{Area of diagonal reinforcement}
 \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
 S_{l, \max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1125.035 \text{ mm} \\
 s &= 200.000 \text{ mm} \leq S_{l, \max} = 1125.035 \text{ mm} \quad \text{OK} \\
 \text{where, } s &\text{ is the spacing of the stirrups} \\
 d &= 1500.046 \text{ mm} && : \text{effective depth}
 \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
 \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 199.672 \text{ kN} \\
 (M_{Rd} - M_{Ed}) / z &= 2881.730 \text{ kN} \\
 \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
 \text{where, } M_{Rd} &= 7828.098 \text{ kN} && : \text{The moment resistance} \\
 z &= 1079.990 \text{ mm} && : \text{Lever arm from the ultimate bending resistance analysis}
 \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
 v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.435 \text{ MPa} \\
 \text{where, } \beta &= 1.0 && : \text{Conservatively adopted to take } \beta = 1.0 \\
 V_{Ed} &= 399.345 \text{ kN} && : \text{Transverse shear force} \\
 z &= 1079.990 \text{ mm} && : \text{Lever arm from the ultimate bending resistance analysis} \\
 b_i &= 850.000 \text{ mm} && : \text{Width of interface}
 \end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
 V_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) &= & 1.419 \text{ MPa} \\
 V_{Rd,lim} &= 0.5 v f_{cd} &= & 8.000 \text{ MPa} \\
 V_{Edi} &= 0.435 \text{ MPa} \leq V_{Rd} = \min[V_{Rdi}, V_{Rd,lim}] &= & 1.419 \text{ MPa} \quad \text{OK} \\
 \text{where, } f_{ctd} &= 1.900 \text{ MPa} && \text{: Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} && \text{: of concrete between girder and slab} \\
 c &= 0.400 && \text{: factors which depend on the roughness of the interface} \\
 \mu &= 0.700 && \text{: factors which depend on the roughness of the interface} \\
 \sigma_n &= w_s / A_i = 0.016 \text{ MPa} \\
 \rho &= A_s / A_i = 0.002 \\
 w_s &= 13.533 \text{ kN} && \text{: Weight of slab} \\
 A_i &= 850000.000 \text{ mm}^2 && \text{: Area of joint (1m for unit length)} \\
 A_s &= 1809.600 \text{ mm}^2 && \text{: Area of reinforcement crossing the interface} \\
 f_{yd} &= 434.783 \text{ MPa} && \text{: Strength of shear connector} \\
 \alpha &= 90.000^\circ && \text{: Angle of reinforcement crossing the interface} \\
 &&& (45^\circ \leq \alpha \leq 90^\circ)
 \end{aligned}$$

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{SLU} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{FX-MIN} \\
 N_{Ed} &= -5720.106 \text{ kN} \\
 V_{Ed} &= -206.399 \text{ kN} \\
 M_{Ed} &= 3249.613 \text{ kN} \cdot \text{m}
 \end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

$$\begin{aligned}
 \text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
 \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
 \end{aligned}$$

- Checking shear resistance of the regions cracked or uncracked in bending

$$\begin{aligned}
 f_b &= -13.629 \text{ MPa} < f_{ctd} = 1.900 \text{ MPa} \\
 &&& \text{(Check the shear strength for noncrack section.)} \\
 \text{where, } f_b &: \text{Flexural tensile stress} && \text{(Tension takes positive)} \\
 f_{ctd} &= \alpha_{ct} \cdot f_{dk,0.05} / \gamma_c && \text{: Design tensile strength of concrete.}
 \end{aligned}$$

- Total stress at composite centroid from actions on the girder alone

$$\begin{aligned}
 \sigma_{tot} &= \sigma_1 + \sigma_2 = 8.440 \text{ MPa} && \text{(DESIGNER'S GUIDE TO EN 1992-2, 6.2.7)} \\
 \text{where, } \sigma_1 &= (M_{non} \cdot y') / I_g = 6.320 \text{ MPa} \\
 \sigma_2 &= P / A_g + (P \cdot e \cdot y') / I_g = 2.120 \text{ MPa} \\
 \sigma_1 &: \text{stress at the composite centroid due to the moment acting on the girder alone} \\
 \sigma_2 &: \text{stress at the composite centroid due to prestress}
 \end{aligned}$$

$$\begin{aligned}
 M_{non} &= 1631.265 \text{ kN} \cdot \text{m} && \text{: Bending Moment in girder alone} \\
 y_{non} &= 652.141 \text{ mm} && \text{: Height to Centroid in girder alone} \\
 y_{com} &= 1056.475 \text{ mm} && \text{: Height to Centroid in Composite section} \\
 y' &= y_{com} - y_{non} = 404.334 \text{ mm} \\
 A_g &= 492854.073 \text{ mm}^2 && \text{: Area Girder alone}
 \end{aligned}$$

$$\begin{aligned}
 I_g &= 1.0437\text{E}+11 \text{ mm}^4 && \text{: Moment of inertia in Girder alone} \\
 P &= 5636.614 \text{ kN} && \text{: A factored prestressing force after all losses} \\
 P \cdot e &= -2404.875 \text{ kN} \cdot \text{m} && \text{: Moment of prestressing force}
 \end{aligned}$$

- Shear stress in girder alone at the height of the composite centroid

$$\tau_s = (V_{c1} / b_w) (Q_n / I_g) = 0.013 \text{ MPa}$$

where,

$$\begin{aligned}
 V_{c1} &= 11.915 \text{ kN} && \text{: Shear force on the precast girder alone} \\
 Q_n &= 2.2666\text{E}+07 \text{ mm}^3 && \text{: First moment of Area in centroid of Girder alone} \\
 I_g &= 1.0437\text{E}+11 \text{ mm}^4 && \text{: Moment of inertia in Girder alone} \\
 b_w &= 200.000 \text{ mm} && \text{: Girder width}
 \end{aligned}$$

- Additional shear force acting on the composite section

$$V_{c2} = (I_c \cdot b_w / Q_c) \cdot \{\sqrt{(f_{ctd})^2 + f_{ctd} \cdot \sigma_{tot}} - \tau_s\} = 984.277 \text{ kN}$$

where,

$$\begin{aligned}
 Q_c &= 2.3457\text{E}+08 \text{ mm}^3 && \text{: First moment of Area in centroid of Composite section} \\
 I_c &= 2.6121\text{E}+11 \text{ mm}^4 && \text{: Moment of inertia in Composite section} \\
 b_w &= 200.000 \text{ mm} && \text{: Girder width}
 \end{aligned}$$

- Total shear resistance of the composite section (regions Un-cracked in bending)

$$V_{Rd,c} = V_{c1} + V_{c2} = 996.192 \text{ kN} > V_{Ed} = -206.399 \text{ kN} \quad \text{OK}$$

\therefore No shear reinforcement is required

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
 \rho_{w, \min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
 \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00283 \geq \rho_{w, \min} = 0.00113 \quad \text{OK} \\
 \text{where, } \rho_{w, \min} &\text{ is the minimum ratio of shear reinforcement} \\
 A_{sw} &= 113.000 \text{ mm}^2 && \text{: Area of diagonal reinforcement}
 \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
 S_{l, \max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1125.035 \text{ mm} \\
 s &= 200.000 \text{ mm} \leq S_{l, \max} = 1125.035 \text{ mm} \quad \text{OK} \\
 \text{where, } s &\text{ is the spacing of the stirrups} \\
 d &= 1500.046 \text{ mm} && \text{: effective depth}
 \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
 \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 103.199 \text{ kN} \\
 (M_{Rd} - M_{Ed}) / z &= 4239.378 \text{ kN} \\
 \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
 \text{where, } M_{Rd} &= 7828.098 \text{ kN} && \text{: The moment resistance} \\
 z &= 1079.990 \text{ mm} && \text{: Lever arm from the ultimate bending resistance analysis}
 \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
 v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.225 \text{ MPa} \\
 \text{where, } \beta &= 1.0 && \text{: Conservatively adopted to take } \beta = 1.0 \\
 V_{Ed} &= -206.399 \text{ kN} && \text{: Transverse shear force} \\
 z &= 1079.990 \text{ mm} && \text{: Lever arm from the ultimate bending resistance analysis} \\
 b_i &= 850.000 \text{ mm} && \text{: Width of interface}
 \end{aligned}$$

- Design shear resistance at the interface

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.419 \text{ MPa}$$

$$\begin{aligned}
 V_{Rd,lim} &= 0.5 v f_{cd} &= & 8.000 \text{ MPa} \\
 V_{Edi} &= 0.225 \text{ MPa} \leq V_{Rd} = \min[V_{Rdi}, V_{Rd,lim}] &= & 1.419 \text{ MPa} \quad \text{OK} \\
 \text{where, } f_{ctd} &= 1.900 \text{ MPa} && \text{: Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} && \text{of concrete between girder and slab} \\
 c &= 0.400 && \text{: factors which depend on the roughness of the interface} \\
 \mu &= 0.700 && \text{: factors which depend on the roughness of the interface} \\
 \sigma_n &= w_s / A_i = 0.016 \text{ MPa} \\
 \rho &= A_s / A_i = 0.002 \\
 w_s &= 13.533 \text{ kN} && \text{: Weight of slab} \\
 A_i &= 850000.000 \text{ mm}^2 && \text{: Area of joint (1m for unit length)} \\
 A_s &= 1809.600 \text{ mm}^2 && \text{: Area of reinforcement crossing the interface} \\
 f_{yd} &= 434.783 \text{ MPa} && \text{: Strength of shear connector} \\
 \alpha &= 90.000^\circ && \text{: Angle of reinforcement crossing the interface} \\
 &&& (45^\circ \leq \alpha \leq 90^\circ)
 \end{aligned}$$

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name : cLCB2
 Design Situations : Persistent & Transient
 Load Combination Type : MZ-MAX
 $N_{Ed} = -11336.898 \text{ kN}$
 $V_{Ed} = 399.345 \text{ kN}$
 $T_{Ed} = 12.073 \text{ kN} \cdot \text{m}$
 $M_{Ed} = 4715.858 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 74.869 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.008 \text{ mm}^2 / \text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 14.781 \text{ mm}^2 / \text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.962 \text{ kN} \cdot \text{m} \\
 I_{xx_i} &= 1.4790\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 1.4729\text{E}+10 \text{ mm}^4 \\
 u &= 1950.074 \text{ mm} \\
 A &= 146000.000 \text{ mm}^2 \\
 u_i &= 1950.074 \text{ mm} \\
 A_{ki} &= 146000.000 \text{ mm}^2
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 218.617 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 11.418 \text{ MPa} \\
N_{Ed} &= 11336.898 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892.198 \text{ mm}^2
\end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 78.378 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.008 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 15.340 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.045 \text{ kN} \cdot \text{m}
\end{aligned}$$

$$I_{xx,i} = 1.6069\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1850.000 \text{ mm}$$

$$A = 145000.000 \text{ mm}^2$$

$$u_i = 1850.000 \text{ mm}$$

$$A_{ki} = 145000.000 \text{ mm}^2$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 227.297 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 11.418 \text{ MPa} \\
N_{Ed} &= 11336.898 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892.198 \text{ mm}^2
\end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 106.934 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.018 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 29.735 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 2.765 \text{ kN} \cdot \text{m}
\end{aligned}$$

$$I_{xx,i} = 4.2496\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1648.219 \text{ mm}$$

$$A = 176250.000 \text{ mm}^2$$

$$u_i = 1648.219 \text{ mm}$$

$$A_{ki} = 176250.000 \text{ mm}^2$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 376.941 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 11.418 \text{ MPa} \\
N_{Ed} &= 11336.898 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892.198 \text{ mm}^2
\end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \Sigma T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.189 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 2159.980 \text{ kN}$

■ Minimum Shear Force

1 Design Load

$$\begin{aligned}
\text{Load Combination Name : } & \text{SLU} \\
\text{Design Situations : } & \text{Persistent \& Transient} \\
\text{Load Combination Type : } & \text{MZ-MIN} \\
N_{Ed} &= -5720.106 \text{ kN} \\
V_{Ed} &= -206.399 \text{ kN} \\
T_{Ed} &= -14.638 \text{ kN} \cdot \text{m} \\
M_{Ed} &= 3249.613 \text{ kN} \cdot \text{m}
\end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 74.869 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.009 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 17.922 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.167 \text{ kN} \cdot \text{m} \\
I_{xx_i} &= 1.4790\text{E}+09 \text{ mm}^4 \\
I_{xx} &= 1.4729\text{E}+10 \text{ mm}^4 \\
u &= 1950.074 \text{ mm} \\
A &= 146000.000 \text{ mm}^2 \\
u_i &= 1950.074 \text{ mm} \\
A_{ki} &= 146000.000 \text{ mm}^2
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 205.121 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned}
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.173 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 5.761 \text{ MPa} \\
N_{Ed} &= 5720.106 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892.198 \text{ mm}^2
\end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 78.378 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.010 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 18.600 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,j} / I_{xx} = 1.268 \text{ kN} \cdot \text{m} \\
 I_{xx,j} &= 1.6069\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 1.4729\text{E}+10 \text{ mm}^4 \\
 u &= 1850.000 \text{ mm} \\
 A &= 145000.000 \text{ mm}^2 \\
 u_i &= 1850.000 \text{ mm} \\
 A_{ki} &= 145000.000 \text{ mm}^2
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$\begin{aligned}
 T_{Rd,max,i} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 213.265 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.173 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 5.761 \text{ MPa} \\
 N_{Ed} &= 5720.106 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 992892.198 \text{ mm}^2
 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 106.934 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.022 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 36.054 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,j} / I_{xx} = 3.353 \text{ kN} \cdot \text{m} \\
 I_{xx,j} &= 4.2496\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 1.4729\text{E}+10 \text{ mm}^4 \\
 u &= 1648.219 \text{ mm} \\
 A &= 176250.000 \text{ mm}^2 \\
 u_i &= 1648.219 \text{ mm} \\
 A_{ki} &= 176250.000 \text{ mm}^2
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$\begin{aligned}
 T_{Rd,max,i} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 353.671 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.173 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 5.761 \text{ MPa} \\
 N_{Ed} &= 5720.106 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 992892.198 \text{ mm}^2
 \end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \Sigma T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.108 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 2014.872 \text{ kN}$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB2

Design Situations : Persistent & Transient

Load Combination Type : MZ-MIN

$$N_{Ed} = -11464.359 \text{ kN}$$

$$V_{Ed} = -153.387 \text{ kN}$$

$$T_{Ed} = -17.055 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 1792.061 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 74.869 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.011 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 20.880 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.359 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.4790\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1950.074 \text{ mm}$$

$$A = 146000.000 \text{ mm}^2$$

$$u_i = 1950.074 \text{ mm}$$

$$A_{ki} = 146000.000 \text{ mm}^2$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,max,i} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin \theta \cdot \cos \theta = 218.617 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 11.546 \text{ MPa}$$

$$N_{Ed} = 11464.359 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892.198 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 78.378 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.012 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 21.670 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.477 \text{ kN} \cdot \text{m}$$

$$\begin{aligned}
 I_{xx_j} &= 1.6069E+09 \text{ mm}^4 \\
 I_{xx} &= 1.4729E+10 \text{ mm}^4 \\
 u &= 1850.000 \text{ mm} \\
 A &= 145000.000 \text{ mm}^2 \\
 u_i &= 1850.000 \text{ mm} \\
 A_{ki} &= 145000.000 \text{ mm}^2
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin\theta \cdot \cos\theta = 227.297 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 11.546 \text{ MPa}$$

$$N_{Ed} = 11464.359 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892.198 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 106.934 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.025 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 42.006 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_j} / I_{xx} = 3.906 \text{ kN} \cdot \text{m}$$

$$I_{xx_j} = 4.2496E+09 \text{ mm}^4$$

$$I_{xx} = 1.4729E+10 \text{ mm}^4$$

$$u = 1648.219 \text{ mm}$$

$$A = 176250.000 \text{ mm}^2$$

$$u_i = 1648.219 \text{ mm}$$

$$A_{ki} = 176250.000 \text{ mm}^2$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Calculate maximum torsion resistance, $T_{Rd,max}$

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_{ef,i} \cdot \sin\theta \cdot \cos\theta = 376.941 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1.250 \quad (\text{for } 0.25 \cdot f_{cd} < \sigma_{cp} \leq 0.5 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 11.546 \text{ MPa}$$

$$N_{Ed} = 11464.359 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892.198 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.077 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 2159.980 \text{ kN}$$

5.Decompression Check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB26
 Design Situations : Quasi-permanent
 Load Combination Type : -
 Long/Short Term : Short-term
 $N_{Ed} = -8915.522 \text{ kN}$
 $M_{Ed} = -1884.304 \text{ kN} \cdot \text{m}$

- Stress check (-: compression + : tension)

$$\sigma = -6.314 \text{ MPa} \leq \sigma_a = 0.000$$

OK

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB23
 Design Situations : Quasi-permanent
 Load Combination Type : FX-MAX
 Long/Short Term : Short-term
 $N_{Ed} = -8849.430 \text{ kN}$
 $M_{Ed} = -1862.898 \text{ kN} \cdot \text{m}$

- Stress check (-: compression + : tension)

$$\sigma = -20.640 \text{ MPa} \leq \sigma_a = 0.000$$

OK

6. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : CS1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
1.26	22.14	1.26	22.14	1.26	22.14	22.14	35.00	OK

For post-tensioned elements, $k_8 = 0.70$

ALW = Limiting stress = $k_8 \cdot f_{ci}$

$f_{ci} = 50 \text{ (MPa)}$

2) Slab

■ Compression

- Critical Stage Name : CS4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
2.94	1.45	3.07	1.58	2.81	1.32	3.07	24.50	OK

For post-tensioned elements, $k_8 = 0.70$

ALW = Limiting stress = $k_8 \cdot f_{ci}$

$f_{ci} = 35 \text{ (MPa)}$

7. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB24

- Load Combination Type : FX-MIN

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
6.34	30.52	6.48	30.62	6.19	30.43	29.62	30.00	OK

For post-tensioned elements, $k_8 = 0.70$

ALW = Limiting stress = $k_8 \cdot f_c$

$$f_c = 42.857143 \text{ (MPa)}$$

2) Slab

■ Compression

- Load Combination Name : cLCB23

- Load Combination Type : FX-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
5.94	-4.09	19.80	10.64	20.21	18.70	20.21	21.00	OK

For post-tensioned elements, $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 30 \text{ (MPa)}$



intocmit ing. Attila Takacs



Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=27.00m, h=1.30m

Element Number	97
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

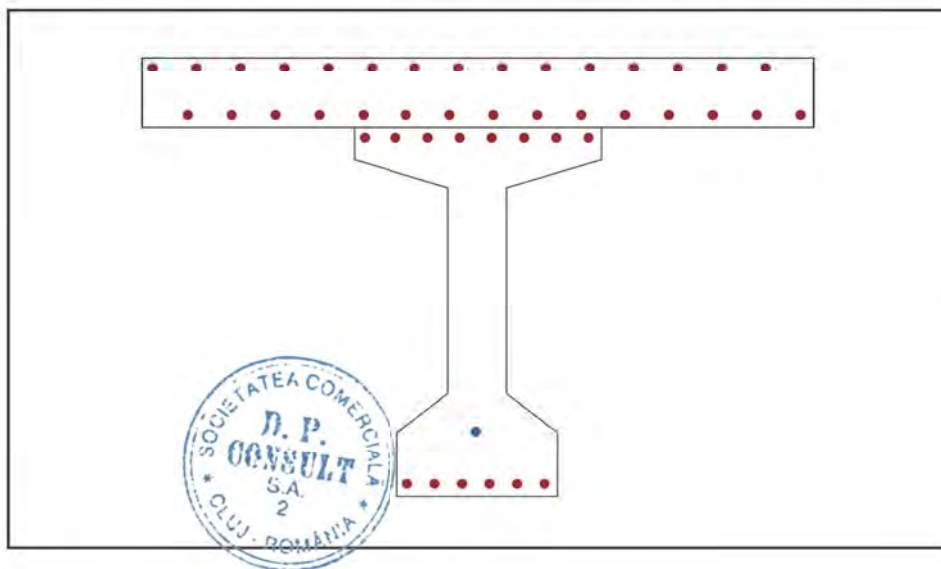
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

α_{cc} = 0.850 (for the Compressive strength)

α_{ct} = 1.000 (for the Tensile strength)

1.2 Sectional Information

Section Information	EqvInt sect(tdn. reinfmnt) (Girder)	EqvInt sect (After Comp) (Girder + Slab)
A (mm ²)	499071.785	1039816.491
I_y (mm ⁴)	106103572735.916	264454175725.296
y_{st} (mm)	-	501.092
y_{sb} (mm)	-	251.092
y_t (mm)	648.999	251.092
y_b (mm)	651.001	1048.908
Z_{st} (mm ³)	-	254867209.396
Z_{sb} (mm ³)	-	341529668.502
Z_t (mm ³)	29191003.783	341529668.502
Z_b (mm ³)	45113369.483	444610227.826



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(g)} = 50.000 \text{ MPa}$$

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Tendon-Span1	1325.6	5040.000	1860.000	1600.000	195000.000

2. Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 7730.649 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	775.0	16447.500	2174.557	346.174	7012.174	2.53074
2	387.5	12849.167	1892.274	346.174	7012.174	2.00336
3	193.8	7070.583	737.609	1783.122	7012.174	0.88777
4	290.6	10605.875	1356.020	346.174	7012.174	1.62562
5	242.2	8838.229	1149.280	1045.513	7012.174	1.23950
6	218.0	7954.406	1013.078	1045.513	7012.174	1.11291
7	205.9	7512.495	737.609	1783.122	7012.174	0.93801
8	211.9	7733.451	974.163	1045.513	7012.174	1.08066
9	208.9	7622.973	737.609	1783.122	7012.174	0.95057
10	210.4	7678.212	964.084	1045.513	7012.174	1.07255
11	209.6	7650.592	737.609	1783.122	7012.174	0.95371
12	210.0	7664.402	961.542	1045.513	7012.174	1.07052
13	209.8	7657.497	737.609	1783.122	7012.174	0.95450
14	209.9	7660.949	737.609	1783.122	7012.174	0.95489
15	210.0	7662.676	737.609	1783.122	7012.174	0.95509
16	210.0	7663.539	961.383	1045.513	7012.174	1.07040
17	210.0	7663.107	737.609	1783.122	7012.174	0.95514
18	210.0	7663.323	737.609	1783.122	7012.174	0.95516
19	210.0	7663.431	737.609	1783.122	7012.174	0.95517
20	210.0	7663.431	737.609	1783.122	7012.174	0.95517

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && \text{: Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 7663.431 \text{ kN} && \text{: Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 737.609 \text{ kN} && \text{: Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1783.122 \text{ kN} && \text{: Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 7012.174 \text{ kN} && \text{: Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 167.997 \text{ mm} \\
 x &= 209.995 \text{ mm} && \text{: neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 386391.472 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 3393.000 \text{ mm}^2 && \text{: Compression reinforcement areas} \\
 A_s &= 2404.680 \text{ mm}^2 && \text{: Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1126.150	0.01859	0.00578	0.02437	1391.304	5040.000	7012.174
$F_p = \sum (F_{pi}) =$							7012.174

where, Stress_{Eff} = $\epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0.1K} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned} M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\ &= 9375.783 \text{ kN} \cdot \text{m} \geq M_{Ed} = 7730.649 \text{ kN} \cdot \text{m} \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_{pi} is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MIN

$$M_{Ed} = 0.000 \text{ kN} \cdot \text{m}$$

- factor λ , and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

4) Calculate the axial force in concrete, steel, and tendon.

5) Check if the resultant force of cross-section is zero.

6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	775.0	6438.750	346.174	2174.557	0.000	3.12014
2	387.5	4574.948	346.174	2174.557	0.000	2.26305
3	193.8	2415.417	346.174	2174.557	6220.738	0.32895
4	290.6	3621.133	346.174	2174.557	0.000	1.82442
5	242.2	3019.271	346.174	2174.557	0.000	1.54765
6	218.0	2717.344	346.174	2174.557	5777.989	0.38522
7	230.1	2868.307	346.174	2174.557	0.000	1.47822
8	224.0	2792.826	346.174	2174.557	5682.259	0.39953
9	227.1	2830.566	346.174	2174.557	0.000	1.46087

10	225.5	2811.696	346.174	2174.557	0.000	1.45219
11	224.8	2802.261	346.174	2174.557	0.000	1.44785
12	224.4	2797.543	346.174	2174.557	5676.448	0.40042
13	224.6	2799.902	346.174	2174.557	0.000	1.44677
14	224.5	2798.723	346.174	2174.557	0.000	1.44622
15	224.4	2798.133	346.174	2174.557	0.000	1.44595
16	224.4	2797.838	346.174	2174.557	5676.085	0.40048
17	224.4	2797.985	346.174	2174.557	5675.904	0.40051
18	224.4	2798.059	346.174	2174.557	5675.813	0.40052
19	224.4	2798.096	346.174	2174.557	0.000	1.44594
20	224.4	2798.078	346.174	2174.557	0.000	1.44593

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 2798.078 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 346.174 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 2174.557 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 0.000 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 179.556 \text{ mm} \\
 x &= 224.445 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 98755.678 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 796.200 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 5001.480 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 2935.967 \text{ kN} \cdot \text{m} \geq M_{Ed} = 0.000 \text{ kN} \cdot \text{m} \quad \text{OK}
 \end{aligned}$$

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{cLCB1} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{FZ-MAX} \\
 N_{Ed} &= -5507.405 \text{ kN} \\
 V_{Ed} &= 350.319 \text{ kN} \\
 M_{Ed} &= 7070.037 \text{ kN} \cdot \text{m} \\
 V_p &= 0.000 \text{ kN}
 \end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab} : f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Check whether the section is cracked in bending.

$$f_b = 9.254 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)
 $f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.

- Design value for the shear resistance $V_{Rd,c}$

$$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 358.999 \text{ kN}$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 356.706 \text{ kN} \geq V_{Rd,min}$$

$$= 358.999 \text{ kN} \geq V_{Ed} = 350.319 \text{ kN} \quad \therefore \text{Shear reinforcement is not required}$$

where,

$$C_{Rd,c} = 0.18 / \gamma_c = 0.120$$

$$v_{min} = 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.394$$

$$k_1 = 0.15$$

$$k = 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.364$$

$$\rho_1 = A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003$$

$$A_{sl} = 796.200 \text{ mm}^2$$

$$b_w = 200.0 \text{ mm}$$

$$d_p = 1510.0 \text{ mm}$$

$$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 5.297 \text{ MPa}$$

$$N_{Ed} = 5507.405 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1039816.491 \text{ mm}^2$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w,min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01005 \geq \rho_{w,min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w,min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 402.000 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l,max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 1132.535 \text{ mm}$$

$$s = 200.000 \text{ mm} \leq S_{l,max} = 1132.535 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 1510.046 \text{ mm} \quad : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 175.159 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 2121.074 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z \quad \text{OK}$$

where, $M_{Rd} = 9375.783 \text{ kN}$: The moment resistance
 $z = 1087.065 \text{ mm}$: Lever arm from the ultimate bending resistance analysis

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.379 \text{ MPa}$$

where, $\beta = 1.0$: Conservatively adopted to take $\beta = 1.0$
 $V_{Ed} = 350.319 \text{ kN}$: Transverse shear force
 $z = 1087.065 \text{ mm}$: Lever arm from the ultimate bending resistance analysis

$b_i = 850.000 \text{ mm}$: Width of interface

- Design shear resistance at the interface

$$V_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.773 \text{ MPa}$$

$$V_{Rd,lim} = 0.5 v f_{cd} = 8.000 \text{ MPa}$$

$$V_{Edi} = 0.379 \text{ MPa} \leq V_{Rd} = \min[V_{Rdi}, V_{Rd,lim}] = 1.773 \text{ MPa} \quad \text{OK}$$

where, $f_{ctd} = 1.900 \text{ MPa}$: Minimum design tensile strength(f_{ctd}) and design strength(f_{cd})

$f_{cd} = 33.333 \text{ MPa}$ of concrete between girder and slab

$c = 0.400$: factors which depend on the roughness of the interface

$\mu = 0.700$: factors which depend on the roughness of the interface

$\sigma_n = w_s / A_i = 0.017 \text{ MPa}$

$\rho = A_s / A_i = 0.003$

$w_s = 14.375 \text{ kN}$: Weight of slab

$A_i = 850000.000 \text{ mm}^2$: Area of joint (1m for unit length)

$A_s = 2795.000 \text{ mm}^2$: Area of reinforcement crossing the interface

$f_{yd} = 434.783 \text{ MPa}$: Strength of shear connector

$\alpha = 90.000^\circ$: Angle of reinforcement crossing the interface
($45^\circ \leq \alpha \leq 90^\circ$)

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$N_{Ed} = -5542.266 \text{ kN}$

$V_{Ed} = -261.532 \text{ kN}$

$M_{Ed} = 6511.625 \text{ kN} \cdot \text{m}$

$V_p = 0.000 \text{ kN}$

- Design strength of concrete

Girder : $f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa}$

Slab : $f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}$

- Design strength of Reinforcement

Girder : $f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

Slab : $f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

- Check whether the section is cracked in bending.

$$f_b = 6.756 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)

$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.

- Design value for the shear resistance $V_{Rd,c}$

$$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 360.517 \text{ kN}$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 358.225 \text{ kN} \geq V_{Rd,min}$$

$$= 360.517 \text{ kN} \geq V_{Ed} = -261.532 \text{ kN} \quad \therefore \text{Shear reinforcement is not required}$$

where, $C_{Rd,c} = 0.18 / \gamma_c = 0.120$

$$v_{min} = 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.394$$

$$\begin{aligned}
k_1 &= 0.15 \\
k &= 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.364 \\
\rho_1 &= A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003 \\
A_{sl} &= 796.200 \text{ mm}^2 \\
b_w &= 200.0 \text{ mm} \\
d_p &= 1510.0 \text{ mm} \\
\sigma_{cp} &= N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 5.330 \text{ MPa} \\
N_{Ed} &= 5542.266 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 1039816.491 \text{ mm}^2
\end{aligned}$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
\rho_{w, \min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
\rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01005 \geq \rho_{w, \min} = 0.00113 \quad \text{OK} \\
\text{where, } \rho_{w, \min} &\text{ is the minimum ratio of shear reinforcement} \\
A_{sw} &= 402.000 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}
\end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
S_{l, \max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1132.535 \text{ mm} \\
s &= 200.000 \text{ mm} \leq S_{l, \max} = 1132.535 \text{ mm} \quad \text{OK} \\
\text{where, } s &\text{ is the spacing of the stirrups} \\
d &= 1510.046 \text{ mm} \quad : \text{effective depth}
\end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
\Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 130.766 \text{ kN} \\
(M_{Rd} - M_{Ed}) / z &= 2634.762 \text{ kN} \\
\Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
\text{where, } M_{Rd} &= 9375.783 \text{ kN} \quad : \text{The moment resistance} \\
z &= 1087.065 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis}
\end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.283 \text{ MPa} \\
\text{where, } \beta &= 1.0 \quad : \text{Conservatively adopted to take } \beta = 1.0 \\
V_{Ed} &= -261.532 \text{ kN} \quad : \text{Transverse shear force} \\
z &= 1087.065 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis} \\
b_i &= 850.000 \text{ mm} \quad : \text{Width of interface}
\end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
v_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.773 \text{ MPa} \\
v_{Rd, \lim} &= 0.5 v f_{cd} = 8.000 \text{ MPa} \\
v_{Edi} &= 0.283 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, \lim}] = 1.773 \text{ MPa} \quad \text{OK} \\
\text{where, } f_{ctd} &= 1.900 \text{ MPa} \quad : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
f_{cd} &= 33.333 \text{ MPa} \quad : \text{of concrete between girder and slab} \\
c &= 0.400 \quad : \text{factors which depend on the roughness of the interface} \\
\mu &= 0.700 \quad : \text{factors which depend on the roughness of the interface} \\
\sigma_n &= w_s / A_i = 0.017 \text{ MPa} \\
\rho &= A_s / A_i = 0.003 \\
w_s &= 14.375 \text{ kN} \quad : \text{Weight of slab} \\
A_i &= 850000.000 \text{ mm}^2 \quad : \text{Area of joint (1m for unit length)}
\end{aligned}$$

A_s	=	2795.000 mm ²	: Area of reinforcement crossing the interface
f_{yd}	=	434.783 MPa	: Strength of shear connector
α	=	90.000 °	: Angle of reinforcement crossing the interface (45° ≤ α ≤ 90°)

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name :	cLCB1
Design Situations :	Persistent & Transient
Load Combination Type :	FZ-MAX
N_{Ed}	= -5507.405 kN
V_{Ed}	= 350.319 kN
T_{Ed}	= -8.986 kN · m
M_{Ed}	= 7070.037 kN · m

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$t_{ef,i}$	=	A / u	=	74.9 mm
t_i	=	$\max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim}$	=	106.9 mm
A_{st} / s_t	=	$T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta)$	=	0.0 mm ² /mm
A_{sl} / s_l	=	$T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd})$	=	18.3 mm ² /mm
where,	$T_{Ed,i}$	=	$T_{Ed} \cdot I_{xx_i} / I_{xx}$	= 0.716 kN · m
	I_{xx_i}	=	1.4790E+09 mm ⁴	
	I_{xx}	=	1.4729E+10 mm ⁴	
	u	=	1950.1 mm	
	A	=	146000 mm ²	
	u_i	=	1647.8 mm	
	A_{ki}	=	74042 mm ²	
	$t_{cover,i}$	=	40.0 mm	
	$t_{sect,lim}$	=	200.0 mm	

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 75.999 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.166 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.547 \text{ MPa}$$

$$N_{Ed} = 5507.405 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 78.4 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 22.2 \text{ mm}^2/\text{mm}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.778 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 1.6069\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1850.0 \text{ mm}$$

$$A = 145000 \text{ mm}^2$$

$$u_i = 2276.0 \text{ mm}$$

$$A_{ki} = 91613 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 136.778 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.166 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.547 \text{ MPa}$$

$$N_{Ed} = 5507.405 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 106.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 31.7 \text{ mm}^2/\text{mm}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 2.058 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 4.2496\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1648.2 \text{ mm}$$

$$A = 176250 \text{ mm}^2$$

$$u_i = 1331.6 \text{ mm}$$

$$A_{ki} = 99560 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 198.686 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.166 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.547 \text{ MPa}$$

$$N_{Ed} = 5507.405 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892 \text{ mm}^2$$

- Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.181 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 2015.672 \text{ kN}$$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$$N_{Ed} = -5542.266 \text{ kN}$$

$$V_{Ed} = -261.532 \text{ kN}$$

$$T_{Ed} = 12.956 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 6511.625 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 74.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 26.4 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.033 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.4790\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1950.1 \text{ mm}$$

$$A = 146000 \text{ mm}^2$$

$$u_i = 1647.8 \text{ mm}$$

$$A_{ki} = 74042 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 76.068 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.167 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 5.582 \text{ MPa} \\ N_{Ed} &= 5542.266 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 992892 \text{ mm}^2 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 78.4 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm} \\ A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 32.1 \text{ mm}^2/\text{mm} \end{aligned}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.122 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 1.6069\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1850.0 \text{ mm}$$

$$A = 145000 \text{ mm}^2$$

$$u_i = 2276.0 \text{ mm}$$

$$A_{ki} = 91613 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 136.901 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.167 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 5.582 \text{ MPa} \\ N_{Ed} &= 5542.266 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 992892 \text{ mm}^2 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 106.9 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm} \\ A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 45.6 \text{ mm}^2/\text{mm} \end{aligned}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 2.967 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 4.2496 \text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729 \text{E}+10 \text{ mm}^4$$

$$u = 1648.2 \text{ mm}$$

$$A = 176250 \text{ mm}^2$$

$$u_i = 1331.6 \text{ mm}$$

$$A_{ki} = 99560 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 198.866 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.167 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.582 \text{ MPa}$$

$$N_{Ed} = 5542.266 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.140 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 2017.421 \text{ kN}$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MX-MAX

$$N_{Ed} = -5589.634 \text{ kN}$$

$$V_{Ed} = -155.356 \text{ kN}$$

$$T_{Ed} = 20.213 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 6250.297 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 74.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 41.2 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.611 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.4790\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1950.1 \text{ mm}$$

$$A = 146000 \text{ mm}^2$$

$$u_i = 1647.8 \text{ mm}$$

$$A_{ki} = 74042 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 76.161 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.169 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.630 \text{ MPa}$$

$$N_{Ed} = 5539.634 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 992892 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 78.4 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 50.0 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.750 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.6069\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.4729\text{E}+10 \text{ mm}^4$$

$$u = 1850.0 \text{ mm}$$

$$A = 145000 \text{ mm}^2$$

$$u_i = 2276.0 \text{ mm}$$

$$A_{ki} = 91613 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 137.069 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\begin{aligned}
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.169 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 5.630 \text{ MPa} \\
N_{Ed} &= 5539.634 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892 \text{ mm}^2
\end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 106.9 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.9 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 71.2 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 4.629 \text{ kN} \cdot \text{m} \\
I_{xx,i} &= 4.2496\text{E}+09 \text{ mm}^4 \\
I_{xx} &= 1.4729\text{E}+10 \text{ mm}^4 \\
u &= 1648.2 \text{ mm} \\
A &= 176250 \text{ mm}^2 \\
u_i &= 1331.6 \text{ mm} \\
A_{ki} &= 99560 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 199.109 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.169 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 5.630 \text{ MPa} \\
N_{Ed} &= 5539.634 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 992892 \text{ mm}^2
\end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$\begin{aligned}
T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} &= 0.093 \leq 1.0 \quad \text{OK} \\
\text{where, } V_{Rd,max} &= 2019.798 \text{ kN}
\end{aligned}$$

5.Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

$$\begin{aligned}
\text{Load Combination Name :} & \quad \text{cLCB5} \\
\text{Design Situations :} & \quad \text{Quasi-permanent} \\
\text{Load Combination Type :} & \quad - \\
\text{Long/Short Term :} & \quad \text{Short-term} \\
N_{Ed} &= -5687.507 \text{ kN} \\
M_{Ed} &= 934.278 \text{ kN} \cdot \text{m}
\end{aligned}$$

- Stress check (- : compression + : tension)

$$\sigma = -10.224 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4
 Design Situations : Frequent
 Load Combination Type : MY-MAX
 Long/Short Term : Short-term
 $N_{Ed} = -5597.031 \text{ kN}$
 $M_{Ed} = 2623.341 \text{ kN} \cdot \text{m}$

- Stress check (-: compression + : tension)

$$\sigma = -2.359 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

6. Crack Width

■ Top

1 Calculate crack width, w_k (EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4
 Serviceability Load Type : Frequent
 Load Combination Type : MY-MIN
 Long/Short Term : Short-term
 $N_{Ed} = -5688.565 \text{ kN}$
 $M_{Ed} = 915.734 \text{ kN} \cdot \text{m}$

- Stress at top surface

$$\sigma_{top} = -10.207 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
2.82	22.12	2.84	22.13	2.81	22.11	22.13	31.38	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_{ci}$

$f_{ci} = 44.82176 \text{ (MPa)}$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
0.99	1.41	0.98	1.41	1.00	1.42	1.42	24.50	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_{ci}$

$f_{ci} = 35 \text{ (MPa)}$

8. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3
- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
12.67	-2.19	12.76	-2.13	12.59	-2.24	12.76	30.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 42.85714$ (MPa)



2) Slab

■ Compression

- Load Combination Name : cLCB3
- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
20.35	4.93	20.34	4.92	19.97	17.84	20.35	21.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 30$ (MPa)



intocmit ing. Attila Takacs

Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=31.50m, h=1.40m

Element Number	66
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

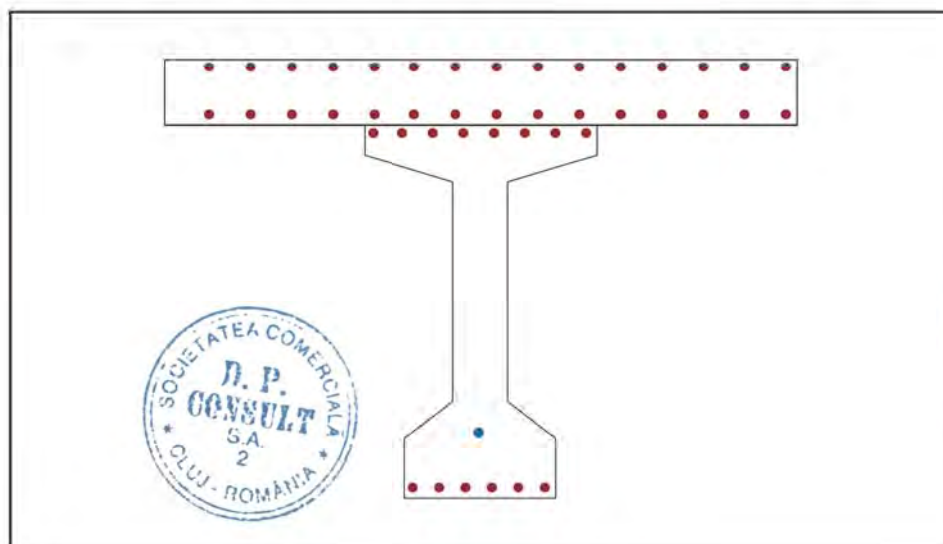
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength,

α_{cc} = 0.850 (for the Compressive strength)

α_{ct} = 1.000 (for the Tensile strength)

1.2 Sectional Information

Section Information	EqvInt sect(tdn, reinfmnt) (Girder)	EqvInt sect (After Comp) (Girder + Slab)
A (mm ²)	522625.915	1063370.621
I_y (mm ⁴)	129174063828.402	314291174568.084
y_{st} (mm)	-	537.626
y_{sb} (mm)	-	287.626
y_t (mm)	703.042	287.626
y_b (mm)	696.958	1112.374
Z_{st} (mm ³)	-	258575526.961
Z_{sb} (mm ³)	-	348068173.463
Z_t (mm ³)	29473313.342	348068173.463
Z_b (mm ³)	45549666.075	371014467.202



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(g)} = 50.000 \text{ MPa}$$

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Tendon-Span1	1408.1	5880.000	1860.000	1600.000	195000.000

2. Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 8937.016 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	825.0	16674.167	2174.557	346.174	8180.870	2.21046
2	412.5	13330.833	2003.387	346.174	8180.870	1.79830
3	206.3	7526.750	737.609	1783.122	8180.870	0.82942
4	309.4	11290.125	1504.171	346.174	8180.870	1.50044
5	257.8	9408.438	1223.534	1045.513	8180.870	1.15234
6	232.0	8467.594	1095.582	1045.513	8180.870	1.03650
7	219.1	7997.172	1020.317	1045.513	8180.870	0.97736
8	225.6	8232.383	1059.025	1045.513	8180.870	1.00705
9	222.4	8114.777	1039.951	1045.513	8180.870	0.99223
10	224.0	8173.580	1049.557	1045.513	8180.870	0.99965

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && \text{: Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 8173.580 \text{ kN} && \text{: Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 1049.557 \text{ kN} && \text{: Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1045.513 \text{ kN} && \text{: Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 8180.870 \text{ kN} && \text{: Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 179.180 \text{ mm} \\
 x &= 223.975 \text{ mm} && \text{: neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 412113.281 \text{ mm}^2 && \text{: Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 3393.000 \text{ mm}^2 && \text{: Compression reinforcement areas} \\
 A_s &= 2404.680 \text{ mm}^2 && \text{: Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1114.538	0.01850	0.00572	0.02422	1391.304	5880.000	8180.870
$F_p = \sum (F_{pi}) =$							8180.870

where, Stress_{Eff} = $\epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 11413.638 \text{ kN} \cdot \text{m} \geq M_{Ed} = 8937.016 \text{ kN} \cdot \text{m}
 \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MIN

$M_{Ed} = 0.000 \text{ kN} \cdot \text{m}$

- factor λ , and factor η

$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

Girder : $f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$

Slab : $f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

Girder : $f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

Slab : $f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

- Calculate Neutral Axis

- 1) Assume neutral axis depth.
- 2) Calculate the strain of steel and tendon.
- 3) Calculate the stress of steel and tendon.
- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	825.0	6665.417	346.174	2174.557	3717.096	1.19009
2	412.5	4752.031	346.174	2174.557	4893.808	0.72127
3	618.8	5730.417	346.174	2174.557	4109.333	0.96701
4	721.9	6197.917	346.174	2174.557	3885.197	1.07993
5	670.3	5964.167	346.174	2174.557	3988.645	1.02387
6	644.5	5847.292	346.174	2174.557	4046.575	0.99555
7	657.4	5905.729	346.174	2174.557	4017.042	1.00974
8	651.0	5876.510	346.174	2174.557	4031.662	1.00265
9	647.8	5861.901	346.174	2174.557	4039.082	0.99910

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p

(EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$F_{c(g)} = (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 5861.901 \text{ kN}$: Compression Force for Girder

$F_{c(s)} = (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN}$: Compression Force for Slab

$F_{s'} = f_s \cdot A_{s'} = 346.174 \text{ kN}$: Compression reinforcement Force

$F_s = f_s \cdot A_s = 2174.557 \text{ kN}$: Tension Force in reinforcement

$F_p = \sum F_{pi} = 4039.082 \text{ kN}$: Tension Force in Tendon

where, $a = \lambda \cdot x = 518.203 \text{ mm}$

$x = 647.754 \text{ mm}$: neutral axis depth

$A_{ca(g)} = 206890.625 \text{ mm}^2$: Area of rectangular con'c stress block for girder part

$$\begin{aligned}
 A_{ca(s)} &= 0.000 \text{ mm}^2 & : \text{Area of rectangular con'c stress block for slab part} \\
 A_s' &= 796.200 \text{ mm}^2 & : \text{Compression reinforcement areas} \\
 A_s &= 5001.480 \text{ mm}^2 & : \text{Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1114.538	-0.00219	0.00572	0.00352	686.919	5880.000	4039.082
$F_p = \sum (F_{pi}) =$							4039.082

where, $\text{StressEff} = \epsilon_{p(0)i} \cdot E_{pi}$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 6243.731 \text{ kN} \cdot \text{m} \geq M_{Ed} = 0.000 \text{ kN} \cdot \text{m}
 \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MAX

$$N_{Ed} = -6550.129 \text{ kN}$$

$$V_{Ed} = 154.810 \text{ kN}$$

$$M_{Ed} = 8066.348 \text{ kN} \cdot \text{m}$$

$$V_p = 0.000 \text{ kN}$$

- Design strength of concrete

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}$$

- Design strength of Reinforcement

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Check whether the section is cracked in bending.

$$f_b = 7.355 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)

$$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c \quad : \text{Design tensile strength of concrete.}$$

- Design value for the shear resistance $V_{Rd,c}$

$$\begin{aligned}
 V_{Rd,min} &= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 424.098 \text{ kN} \\
 V_{Rd,c} &= [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 419.483 \text{ kN} \geq V_{Rd,min} \\
 &= 424.098 \text{ kN} \geq V_{Ed} = 154.810 \text{ kN} \quad \therefore \text{Shear reinforcement is not required} \\
 \text{where, } C_{Rd,c} &= 0.18 / \gamma_c = 0.120 \\
 v_{min} &= 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.389 \\
 k_1 &= 0.15 \\
 k &= 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.352 \\
 \rho_1 &= A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.002 \\
 A_{sl} &= 796.200 \text{ mm}^2 \\
 b_w &= 200.0 \text{ mm} \\
 d_p &= 1615.0 \text{ mm} \\
 \sigma_{cp} &= N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 6.160 \text{ MPa} \\
 N_{Ed} &= 6550.129 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 1063370.621 \text{ mm}^2
 \end{aligned}$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
 \rho_{w,min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
 \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00770 \geq \rho_{w,min} = 0.00113 \quad \text{OK} \\
 \text{where, } \rho_{w,min} &\text{ is the minimum ratio of shear reinforcement} \\
 A_{sw} &= 308.000 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}
 \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
 S_{l,max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1211.258 \text{ mm} \\
 s &= 200.000 \text{ mm} \leq S_{l,max} = 1211.258 \text{ mm} \quad \text{OK} \\
 \text{where, } s &\text{ is the spacing of the stirrups} \\
 d &= 1615.010 \text{ mm} \quad : \text{effective depth}
 \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
 \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 77.405 \text{ kN} \\
 (M_{Rd} - M_{Ed}) / z &= 2705.831 \text{ kN} \\
 \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
 \text{where, } M_{Rd} &= 11413.638 \text{ kN} \quad : \text{The moment resistance} \\
 z &= 1237.065 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis}
 \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
 v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.147 \text{ MPa} \\
 \text{where, } \beta &= 1.0 \quad : \text{Conservatively adopted to take } \beta = 1.0 \\
 V_{Ed} &= 154.810 \text{ kN} \quad : \text{Transverse shear force} \\
 z &= 1237.065 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis} \\
 b_i &= 850.000 \text{ mm} \quad : \text{Width of interface}
 \end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
 v_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.897 \text{ MPa} \\
 v_{Rd,lim} &= 0.5 v f_{cd} = 8.000 \text{ MPa} \\
 v_{Edi} &= 0.147 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd,lim}] = 1.897 \text{ MPa} \quad \text{OK} \\
 \text{where, } f_{ctd} &= 1.900 \text{ MPa} \quad : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} \quad \text{of concrete between girder and slab} \\
 c &= 0.400 \quad : \text{factors which depend on the roughness of the interface}
 \end{aligned}$$

μ	=	0.700	: factors which depend on the roughness of the interface
σ_n	=	w_s / A_l	= 0.017 MPa
ρ	=	A_s / A_l	= 0.004
w_s	=	14.375 kN	: Weight of slab
A_l	=	850000.000 mm ²	: Area of joint (1m for unit length)
A_s	=	3141.600 mm ²	: Area of reinforcement crossing the interface
f_{yd}	=	434.783 MPa	: Strength of shear connector
α	=	90.000 °	: Angle of reinforcement crossing the interface (45° ≤ α ≤ 90°)

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name :	cLCB1
Design Situations :	Persistent & Transient
Load Combination Type :	FZ-MIN
N_{Ed}	= -6559.030 kN
V_{Ed}	= -110.406 kN
M_{Ed}	= 7505.631 kN · m
V_p	= 0.000 kN

- Design strength of concrete

Girder :	$f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c$	= 33.333 MPa
Slab :	$f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c$	= 23.333 MPa

- Design strength of Reinforcement

Girder :	$f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar}$	= 434.783 MPa
Slab :	$f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar}$	= 434.783 MPa

- Check whether the section is cracked in bending.

f_b	= 5.122 MPa	>	f_{ctd}	= 1.900 MPa
(The section is cracked.)				
where, f_b	: Flexural tensile stress (Tension is taken as positive.)			
f_{ctd}	= $\alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.		

- Design value for the shear resistance $V_{Rd,c}$

$V_{Rd,min}$	= $(v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p$	= 424.503 kN	
$V_{Rd,c}$	= $[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p$	= 419.889 kN	$\geq V_{Rd,min}$
	= 424.503 kN	$\geq V_{Ed}$	= -110.406 kN \therefore Shear reinforcement is not required
where, $C_{Rd,c}$	= $0.18 / \gamma_c$	= 0.120	
v_{min}	= $0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)}$	= 0.389	
k_1	= 0.15		
k	= $1 + \sqrt{(200 / d_p)} \leq 2.0$	= 1.352	
ρ_1	= $A_{sl} / (b_w \cdot d_p) \leq 0.02$	= 0.002	
A_{sl}	= 796.200 mm ²		
b_w	= 200.0 mm		
d_p	= 1615.0 mm		
σ_{cp}	= $N_{Ed} / A_c \leq 0.2 \cdot f_{cd}$	= 6.168 MPa	
N_{Ed}	= 6559.030 kN	(if compression, $N_{Ed} > 0$)	
A_c	= 1063370.621 mm ²		

2) Check minimum shear reinforcement

(EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w, \min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.00770 \geq \rho_{w, \min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w, \min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 308.000 \text{ mm}^2 : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l, \max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 1211.258 \text{ mm}$$

$$s = 200.000 \text{ mm} \leq S_{l, \max} = 1211.258 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 1615.010 \text{ mm} : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 55.203 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 3159.095 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z \quad \text{OK}$$

where, $M_{Rd} = 11413.638 \text{ kN} : \text{The moment resistance}$

$$z = 1237.065 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

4) Check shear at the interface between concrete cast at different times

(EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.105 \text{ MPa}$$

where, $\beta = 1.0 : \text{Conservatively adopted to take } \beta = 1.0$

$$V_{Ed} = -110.406 \text{ kN} : \text{Transverse shear force}$$

$$z = 1237.065 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

$$b_i = 850.000 \text{ mm} : \text{Width of interface}$$

- Design shear resistance at the interface

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.897 \text{ MPa}$$

$$v_{Rd, \lim} = 0.5 v f_{cd} = 8.000 \text{ MPa}$$

$$v_{Edi} = 0.105 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, \lim}] = 1.897 \text{ MPa} \quad \text{OK}$$

where, $f_{ctd} = 1.900 \text{ MPa} : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd})$

$$f_{cd} = 33.333 \text{ MPa} : \text{of concrete between girder and slab}$$

$$c = 0.400 : \text{factors which depend on the roughness of the interface}$$

$$\mu = 0.700 : \text{factors which depend on the roughness of the interface}$$

$$\sigma_n = w_s / A_i = 0.017 \text{ MPa}$$

$$\rho = A_s / A_i = 0.004$$

$$w_s = 14.375 \text{ kN} : \text{Weight of slab}$$

$$A_i = 850000.000 \text{ mm}^2 : \text{Area of joint (1m for unit length)}$$

$$A_s = 3141.600 \text{ mm}^2 : \text{Area of reinforcement crossing the interface}$$

$$f_{yd} = 434.783 \text{ MPa} : \text{Strength of shear connector}$$

$$\alpha = 90.000^\circ : \text{Angle of reinforcement crossing the interface}$$

($45^\circ \leq \alpha \leq 90^\circ$)

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MAX

$$N_{Ed} = -6550.129 \text{ kN}$$

$$V_{Ed} = 154.810 \text{ kN}$$

$$T_{Ed} = -7.510 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 8066.348 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 74.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 15.1 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.590 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 1.4790\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 1950.1 \text{ mm}$$

$$A = 146000 \text{ mm}^2$$

$$u_i = 1647.8 \text{ mm}$$

$$A_{ki} = 74042 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 77.797 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.467 \text{ MPa}$$

$$N_{Ed} = 6550.129 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 80.5 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 16.7 \text{ mm}^2/\text{mm}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 0.748 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 1.8749\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 2050.0 \text{ mm}$$

$$A = 165000 \text{ mm}^2$$

$$u_i = 1728.0 \text{ mm}$$

$$A_{ki} = 88978 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 136.817 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.467 \text{ MPa}$$

$$N_{Ed} = 6550.129 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 106.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 24.9 \text{ mm}^2/\text{mm}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.696 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 4.2499\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 1648.2 \text{ mm}$$

$$A = 176250 \text{ mm}^2$$

$$u_i = 1256.5 \text{ mm}$$

$$A_{ki} = 98596 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 201.418 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\begin{aligned}\sigma_{cp} &= N_{Ed} / A_c = 6.467 \text{ MPa} \\ N_{Ed} &= 6550.129 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 1012892 \text{ mm}^2\end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.072 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 2345.067 \text{ kN}$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1
Design Situations : Persistent & Transient
Load Combination Type : FZ-MIN

$$\begin{aligned}N_{Ed} &= -6559.030 \text{ kN} \\ V_{Ed} &= -110.406 \text{ kN} \\ T_{Ed} &= 16.251 \text{ kN} \cdot \text{m} \\ M_{Ed} &= 7505.631 \text{ kN} \cdot \text{m}\end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}t_{ef,i} &= A / u = 74.9 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 32.7 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.277 \text{ kN} \cdot \text{m} \\ I_{xx_i} &= 1.4790E+09 \text{ mm}^4 \\ I_{xx} &= 1.5017E+10 \text{ mm}^4 \\ u &= 1950.1 \text{ mm} \\ A &= 146000 \text{ mm}^2 \\ u_i &= 1647.8 \text{ mm} \\ A_{ki} &= 74042 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm}\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 77.814 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\begin{aligned}
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 6.476 \text{ MPa} \\
N_{Ed} &= 6559.030 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 1012892 \text{ mm}^2
\end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 80.5 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 36.2 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.619 \text{ kN} \cdot \text{m} \\
I_{xx,i} &= 1.8749\text{E}+09 \text{ mm}^4 \\
I_{xx} &= 1.5017\text{E}+10 \text{ mm}^4 \\
u &= 2050.0 \text{ mm} \\
A &= 165000 \text{ mm}^2 \\
u_i &= 1728.0 \text{ mm} \\
A_{ki} &= 88978 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 136.847 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 6.476 \text{ MPa} \\
N_{Ed} &= 6559.030 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 1012892 \text{ mm}^2
\end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 106.9 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 53.8 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 3.669 \text{ kN} \cdot \text{m} \\
I_{xx,i} &= 4.2499\text{E}+09 \text{ mm}^4 \\
I_{xx} &= 1.5017\text{E}+10 \text{ mm}^4 \\
u &= 1648.2 \text{ mm} \\
A &= 176250 \text{ mm}^2 \\
u_i &= 1256.5 \text{ mm} \\
A_{ki} &= 98596 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 201.463 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.476 \text{ MPa}$$

$$N_{Ed} = 6559.030 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.060 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 2345.564 \text{ kN}$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MX-MAX

$N_{Ed} = -6537.548 \text{ kN}$

$V_{Ed} = -60.000 \text{ kN}$

$T_{Ed} = 30.951 \text{ kN} \cdot \text{m}$

$M_{Ed} = 7661.395 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 74.9 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 62.2 \text{ mm}^2/\text{mm}$$

where,

$$T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 2.432 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.4790\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 1950.1 \text{ mm}$$

$$A = 146000 \text{ mm}^2$$

$$u_i = 1647.8 \text{ mm}$$

$$A_{ki} = 74042 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{\text{sect,lim}} = 200.0 \text{ mm}$$

$t_{\text{ef},i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{\text{Rd,maxi}} = 2 \cdot v \cdot \alpha_{\text{cw}} \cdot f_{\text{cd}} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 77.773 \text{ kN} \cdot \text{m} \geq T_{\text{Ed},i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{\text{ck}} / 250) = 0.480$$

$$\alpha_{\text{cw}} = 1 + \sigma_{\text{cp}} / f_{\text{cd}} = 1.194 \quad (\text{for } 0 < \sigma_{\text{cp}} \leq 0.25 \cdot f_{\text{cd}})$$

$$\sigma_{\text{cp}} = N_{\text{Ed}} / A_c = 6.454 \text{ MPa}$$

$$N_{\text{Ed}} = 6537.548 \text{ kN} \quad (\text{if compression, } N_{\text{Ed}} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{\text{ef},i} = A / u = 80.5 \text{ mm}$$

$$t_i = \max[t_{\text{ef},i}, 2t_{\text{cover},i}] < t_{\text{sect,lim}} = 106.8 \text{ mm}$$

$$A_{\text{st}} / s_t = T_{\text{Ed},i} / (2 \cdot A_{ki} \cdot f_{\text{yd}} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{\text{sl}} / s_l = T_{\text{Ed},i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{\text{yd}}) = 68.9 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{\text{Ed},i} = T_{\text{Ed}} \cdot I_{\text{xx},i} / I_{\text{xx}} = 3.083 \text{ kN} \cdot \text{m}$$

$$I_{\text{xx},i} = 1.8749\text{E}+09 \text{ mm}^4$$

$$I_{\text{xx}} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 2050.0 \text{ mm}$$

$$A = 165000 \text{ mm}^2$$

$$u_i = 1728.0 \text{ mm}$$

$$A_{ki} = 88978 \text{ mm}^2$$

$$t_{\text{cover},i} = 40.0 \text{ mm}$$

$$t_{\text{sect,lim}} = 200.0 \text{ mm}$$

$t_{\text{ef},i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{\text{Rd,maxi}} = 2 \cdot v \cdot \alpha_{\text{cw}} \cdot f_{\text{cd}} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 136.774 \text{ kN} \cdot \text{m} \geq T_{\text{Ed},i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{\text{ck}} / 250) = 0.480$$

$$\alpha_{\text{cw}} = 1 + \sigma_{\text{cp}} / f_{\text{cd}} = 1.194 \quad (\text{for } 0 < \sigma_{\text{cp}} \leq 0.25 \cdot f_{\text{cd}})$$

$$\sigma_{\text{cp}} = N_{\text{Ed}} / A_c = 6.454 \text{ MPa}$$

$$N_{\text{Ed}} = 6537.548 \text{ kN} \quad (\text{if compression, } N_{\text{Ed}} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{\text{ef},i} = A / u = 106.9 \text{ mm}$$

$$t_i = \max[t_{\text{ef},i}, 2t_{\text{cover},i}] < t_{\text{sect,lim}} = 106.8 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm}$$

$$A_{st}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 102.4 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 6.988 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 4.2499\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.5017\text{E}+10 \text{ mm}^4$$

$$u = 1648.2 \text{ mm}$$

$$A = 176250 \text{ mm}^2$$

$$u_i = 1256.5 \text{ mm}$$

$$A_{ki} = 98596 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 201.355 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.194 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.454 \text{ MPa}$$

$$N_{Ed} = 6537.548 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1012892 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.050 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 2344.364 \text{ kN}$$

5.Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB5

Design Situations : Quasi-permanent

Load Combination Type : -

Long/Short Term : Short-term

$$N_{Ed} = -6618.883 \text{ kN}$$

$$M_{Ed} = 1270.062 \text{ kN} \cdot \text{m}$$

- Stress check (- : compression + : tension)

$$\sigma = -11.213 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4

Design Situations : Frequent

Load Combination Type : MY-MAX

Long/Short Term : Short-term

$$N_{Ed} = -6576.177 \text{ kN}$$

$$M_{Ed} = 2872.873 \text{ kN} \cdot \text{m}$$

- Stress check (-: compression + : tension)

$$\sigma = -3.820 \text{ MPa} \leq \sigma_a = 0.000$$

OK

6. Crack Width

■ Top

1 Calculate crack width, w_k

(EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4

Serviceability Load Type : Frequent

Load Combination Type : MY-MIN

Long/Short Term : Short-term

$$N_{Ed} = -6638.126 \text{ kN}$$

$$M_{Ed} = 1249.466 \text{ kN} \cdot \text{m}$$

- Stress at top surface

$$\sigma_{top} = -11.213 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
4.36	22.52	4.40	22.54	4.31	22.49	22.54	31.38	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f'_{ci}$$

$$f'_{ci} = 44.82176 \text{ (MPa)}$$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
1.36	1.71	1.37	1.72	1.35	1.70	1.72	24.50	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f'_{ci}$$

$$f'_{ci} = 35 \text{ (MPa)}$$

8. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
13.65	-0.76	13.83	-0.64	13.47	-0.87	13.83	30.00	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f'_c$$

$$f_c = 42.85714 \text{ (MPa)}$$

2) Slab

■ Compression

- Load Combination Name : cLCB3
- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
19.14	5.04	19.15	5.05	18.34	16.50	19.15	21.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 30 \text{ (MPa)}$



intocmit ing. Attila Takacs



Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=36.00m, h=2.00m

Element Number	118
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

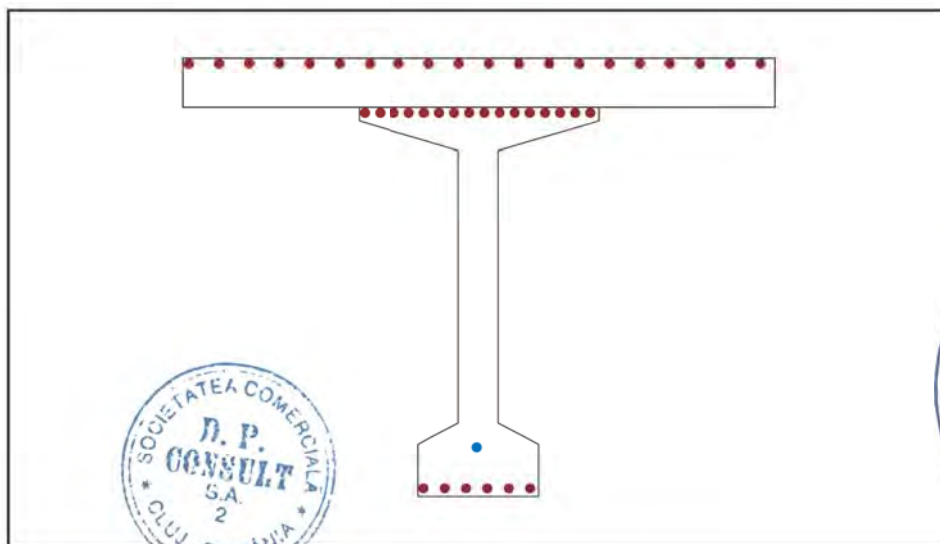
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

α_{cc} = 0.850 (for the Compressive strength)

α_{ct} = 1.000 (for the Tensile strength)

1.2 Sectional Information

Section Information	EqvInt sect(tdn, reinfmnt) (Girder)	EqvInt sect (After Comp) (Girder + Slab)
A (mm ²)	722136.155	1433365.477
I_y (mm ⁴)	385254526615.250	833700901695.910
y_{st} (mm)	-	694.825
y_{sb} (mm)	-	434.825
y_t (mm)	982.273	434.825
y_b (mm)	1017.727	1565.175
Z_{st} (mm ³)	-	479058805.593
Z_{sb} (mm ³)	-	621738563.195
Z_t (mm ³)	42342925.884	621738563.195
Z_b (mm ³)	84685851.768	481584525.177



1.3 Material Information

■ Girder

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(g)} = 50.000 \text{ MPa}$$

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Gr2	2012.0	7000.000	1860.000	1674.000	195000.000

2. Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 16777.196 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	1130.0	22969.833	2382.157	346.174	10189.565	2.40628
2	565.0	20334.456	2382.157	346.174	10189.565	2.15615
3	282.5	13222.883	983.478	1744.852	10189.565	1.19037
4	141.3	6611.442	983.478	1744.852	10189.565	0.63639
5	211.9	9917.163	983.478	1744.852	10189.565	0.91338
6	247.2	11570.023	983.478	1744.852	10189.565	1.05187
7	229.5	10743.593	983.478	1744.852	10189.565	0.98263
8	238.4	11156.808	983.478	1744.852	10189.565	1.01725
9	233.9	10950.200	983.478	1744.852	10189.565	0.99994

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 10950.200 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 983.478 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1744.852 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 10189.565 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 187.156 \text{ mm} \\
 x &= 233.945 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 552110.938 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 2262.000 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 4013.160 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Gr2	997.746	0.02660	0.00512	0.03172	1455.652	7000.000	10189.565
$F_p = \sum (F_{pi}) =$							10189.565

where, StressEff = $\epsilon_{p(0)i} \cdot E_p$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 20624.953 \text{ kN} \cdot \text{m} \geq M_{Ed} = 16777.196 \text{ kN} \cdot \text{m}
 \end{aligned}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MIN

$M_{Ed} = 0.000 \text{ kN} \cdot \text{m}$

- factor λ , and factor η

$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

Girder : $f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$

Slab : $f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

Girder : $f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

Slab : $f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$

- Calculate Neutral Axis

- 1) Assume neutral axis depth.
- 2) Calculate the strain of steel and tendon.
- 3) Calculate the stress of steel and tendon.
- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	1130.0	8806.000	346.174	2382.157	0.000	3.84197
2	565.0	6244.667	346.174	2382.157	0.000	2.76675
3	282.5	3842.000	346.174	2382.157	0.000	1.75814
4	141.3	1921.000	346.174	2382.157	10189.565	0.18034
5	211.9	2881.500	346.174	2382.157	7798.794	0.31703
6	247.2	3361.750	346.174	2382.157	6999.927	0.39521
7	264.8	3601.875	346.174	2382.157	0.000	1.65734
8	256.0	3481.813	346.174	2382.157	0.000	1.60694
9	251.6	3421.781	346.174	2382.157	0.000	1.58174
10	249.4	3391.766	346.174	2382.157	0.000	1.56914
11	248.3	3376.758	346.174	2382.157	0.000	1.56284
12	247.7	3369.254	346.174	2382.157	6989.252	0.39646
13	248.0	3373.006	346.174	2382.157	0.000	1.56127
14	247.9	3371.130	346.174	2382.157	6986.590	0.39678
15	247.9	3372.068	346.174	2382.157	6985.261	0.39693
16	248.0	3372.537	346.174	2382.157	6984.596	0.39701
17	248.0	3372.771	346.174	2382.157	6984.264	0.39705
18	248.0	3372.889	346.174	2382.157	0.000	1.56122
19	248.0	3372.830	346.174	2382.157	0.000	1.56119
20	248.0	3372.801	346.174	2382.157	0.000	1.56118

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 3372.801 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 346.174 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 2382.157 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 0.000 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 198.400 \text{ mm} \\
 x &= 248.000 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 119040.024 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 796.200 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 5478.960 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 4921.463 \text{ kN} \cdot \text{m} \geq M_{Ed} = 0.000 \text{ kN} \cdot \text{m} \quad \text{OK}
 \end{aligned}$$

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{cLCB1} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{FZ-MAX} \\
 N_{Ed} &= -6840.356 \text{ kN} \\
 V_{Ed} &= 441.358 \text{ kN} \\
 M_{Ed} &= 15467.790 \text{ kN} \cdot \text{m} \\
 V_p &= 0.000 \text{ kN}
 \end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

$$\begin{aligned}
 \text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
 \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
 \end{aligned}$$

- Check whether the section is cracked in bending.

$$\begin{aligned}
 f_b &= 11.329 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa} \\
 & && (\text{The section is cracked.}) \\
 \text{where, } f_b &: \text{Flexural tensile stress} && (\text{Tension is taken as positive.}) \\
 f_{ctd} &= \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c && : \text{Design tensile strength of concrete.}
 \end{aligned}$$

- Design value for the shear resistance $V_{Rd,c}$

$$\begin{aligned}
 V_{Rd,min} &= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 481.748 \text{ kN} \\
 V_{Rd,c} &= [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 462.633 \text{ kN} \geq V_{Rd,min}
 \end{aligned}$$

$$\begin{aligned}
 &= 481.748 \text{ kN} \geq V_{Ed} = 441.358 \text{ kN} \quad \therefore \text{Shear reinforcement is not required} \\
 \text{where, } C_{Rd,c} &= 0.18 / \gamma_c = 0.120 \\
 v_{min} &= 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.367 \\
 k_1 &= 0.15 \\
 k &= 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.300 \\
 \rho_1 &= A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.002 \\
 A_{sl} &= 796.200 \text{ mm}^2 \\
 b_w &= 200.0 \text{ mm} \\
 d_p &= 2225.0 \text{ mm} \\
 \sigma_{cp} &= N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 4.772 \text{ MPa} \\
 N_{Ed} &= 6840.356 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 1433365.477 \text{ mm}^2
 \end{aligned}$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
 \rho_{w, min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
 \rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01540 \geq \rho_{w, min} = 0.00113 \quad \text{OK} \\
 \text{where, } \rho_{w, min} &\text{ is the minimum ratio of shear reinforcement} \\
 A_{sw} &= 308.000 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}
 \end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
 S_{l, max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1668.737 \text{ mm} \\
 s &= 100.000 \text{ mm} \leq S_{l, max} = 1668.737 \text{ mm} \quad \text{OK} \\
 \text{where, } s &\text{ is the spacing of the stirrups} \\
 d &= 2224.982 \text{ mm} \quad : \text{effective depth}
 \end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
 \Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 220.679 \text{ kN} \\
 (M_{Rd} - M_{Ed}) / z &= 2984.140 \text{ kN} \\
 \Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
 \text{where, } M_{Rd} &= 20624.953 \text{ kN} \quad : \text{The moment resistance} \\
 z &= 1728.191 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis}
 \end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
 v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.213 \text{ MPa} \\
 \text{where, } \beta &= 1.0 \quad : \text{Conservatively adopted to take } \beta = 1.0 \\
 V_{Ed} &= 441.358 \text{ kN} \quad : \text{Transverse shear force} \\
 z &= 1728.191 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis} \\
 b_i &= 1200.000 \text{ mm} \quad : \text{Width of interface}
 \end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
 v_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.783 \text{ MPa} \\
 v_{Rd, lim} &= 0.5 v f_{cd} = 8.000 \text{ MPa} \\
 v_{Edi} &= 0.213 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, lim}] = 1.783 \text{ MPa} \quad \text{OK} \\
 \text{where, } f_{ctd} &= 1.900 \text{ MPa} \quad : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
 f_{cd} &= 33.333 \text{ MPa} \quad : \text{of concrete between girder and slab} \\
 c &= 0.200 \quad : \text{factors which depend on the roughness of the interface} \\
 \mu &= 0.600 \quad : \text{factors which depend on the roughness of the interface} \\
 \sigma_n &= w_s / A_i = 0.016 \text{ MPa}
 \end{aligned}$$

ρ	$= A_s / A_i$	$=$	0.005	
w_s	$=$	19.175	kN	: Weight of slab
A_i	$=$	1200000.000	mm ²	: Area of joint (1m for unit length)
A_s	$=$	6408.000	mm ²	: Area of reinforcement crossing the interface
f_{yd}	$=$	434.783	MPa	: Strength of shear connector
α	$=$	90.000	°	: Angle of reinforcement crossing the interface ($45^\circ \leq \alpha \leq 90^\circ$)

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

Load Combination Name :	cLCB1
Design Situations :	Persistent & Transient
Load Combination Type :	FZ-MIN
N_{Ed}	$=$ -6884.978 kN
V_{Ed}	$=$ -421.845 kN
M_{Ed}	$=$ 14891.711 kN · m
V_p	$=$ 0.000 kN

- Design strength of concrete

Girder :	$f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c$	$=$	33.333	MPa
Slab :	$f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c$	$=$	23.333	MPa

- Design strength of Reinforcement

Girder :	$f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar}$	$=$	434.783	MPa
Slab :	$f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar}$	$=$	434.783	MPa

- Check whether the section is cracked in bending.

f_b	$=$	10.093	MPa	$>$	f_{ctd}	$=$	1.900	MPa
(The section is cracked.)								
where,	f_b	: Flexural tensile stress			(Tension is taken as positive.)			
	f_{ctd}	$= \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.			

- Design value for the shear resistance $V_{Rd,c}$

$V_{Rd,min}$	$= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p$	$=$	483.826	kN
$V_{Rd,c}$	$= [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p$	$=$	464.711	kN $\geq V_{Rd,min}$
	$=$	483.826	kN $\geq V_{Ed}$	$=$ -421.845 kN \therefore Shear reinforcement is not required
where,	$C_{Rd,c}$	$= 0.18 / \gamma_c$	$=$	0.120
	v_{min}	$= 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)}$	$=$	0.367
	k_1	$= 0.15$		
	k	$= 1 + \sqrt{(200 / d_p)} \leq 2.0$	$=$	1.300
	ρ_1	$= A_{sl} / (b_w \cdot d_p) \leq 0.02$	$=$	0.002
	A_{sl}	$=$	796.200	mm ²
	b_w	$=$	200.0	mm
	d_p	$=$	2225.0	mm
	σ_{cp}	$= N_{Ed} / A_c \leq 0.2 \cdot f_{cd}$	$=$	4.803 MPa
	N_{Ed}	$=$	6884.978	kN (if compression, $N_{Ed} > 0$)
	A_c	$=$	1433365.477	mm ²

2) Check minimum shear reinforcement

(EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w, \min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01540 \geq \rho_{w, \min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w, \min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 308.000 \text{ mm}^2 : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l, \max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 1668.737 \text{ mm}$$

$$s = 100.000 \text{ mm} \leq S_{l, \max} = 1668.737 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 2224.982 \text{ mm} : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1;2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 210.923 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 3317.482 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z \quad \text{OK}$$

where, $M_{Rd} = 20624.953 \text{ kN} : \text{The moment resistance}$

$$z = 1728.191 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1;2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.203 \text{ MPa}$$

where, $\beta = 1.0 : \text{Conservatively adopted to take } \beta = 1.0$

$$V_{Ed} = -421.845 \text{ kN} : \text{Transverse shear force}$$

$$z = 1728.191 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}$$

$$b_i = 1200.000 \text{ mm} : \text{Width of interface}$$

- Design shear resistance at the interface

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.783 \text{ MPa}$$

$$v_{Rd, \lim} = 0.5 v_{fcd} = 8.000 \text{ MPa}$$

$$v_{Edi} = 0.203 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, \lim}] = 1.783 \text{ MPa} \quad \text{OK}$$

where, $f_{ctd} = 1.900 \text{ MPa} : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd})$

$$f_{cd} = 33.333 \text{ MPa} : \text{of concrete between girder and slab}$$

$$c = 0.200 : \text{factors which depend on the roughness of the interface}$$

$$\mu = 0.600 : \text{factors which depend on the roughness of the interface}$$

$$\sigma_n = w_s / A_i = 0.016 \text{ MPa}$$

$$\rho = A_s / A_i = 0.005$$

$$w_s = 19.175 \text{ kN} : \text{Weight of slab}$$

$$A_i = 1200000.000 \text{ mm}^2 : \text{Area of joint (1m for unit length)}$$

$$A_s = 6408.000 \text{ mm}^2 : \text{Area of reinforcement crossing the interface}$$

$$f_{yd} = 434.783 \text{ MPa} : \text{Strength of shear connector}$$

$$\alpha = 90.000^\circ : \text{Angle of reinforcement crossing the interface}$$

($45^\circ \leq \alpha \leq 90^\circ$)

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name : cLCB1
 Design Situations : Persistent & Transient
 Load Combination Type : FZ-MAX

$$\begin{aligned}
N_{Ed} &= -6840.356 \text{ kN} \\
V_{Ed} &= 441.358 \text{ kN} \\
T_{Ed} &= -11.070 \text{ kN} \cdot \text{m} \\
M_{Ed} &= 15467.790 \text{ kN} \cdot \text{m}
\end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 73.1 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 22.0 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.767 \text{ kN} \cdot \text{m} \\
I_{xx_i} &= 1.9053\text{E}+09 \text{ mm}^4 \\
I_{xx} &= 2.1772\text{E}+10 \text{ mm}^4 \\
u &= 2584.0 \text{ mm} \\
A &= 189000 \text{ mm}^2 \\
u_i &= 2281.5 \text{ mm} \\
A_{ki} &= 91689 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 59.003 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
\text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
\alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.149 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
\sigma_{cp} &= N_{Ed} / A_c = 4.971 \text{ MPa} \\
N_{Ed} &= 6840.356 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 1376161 \text{ mm}^2
\end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
t_{ef,i} &= A / u = 87.5 \text{ mm} \\
t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm} \\
A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 36.3 \text{ mm}^2/\text{mm} \\
\text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.403 \text{ kN} \cdot \text{m} \\
I_{xx_i} &= 3.4845\text{E}+09 \text{ mm}^4
\end{aligned}$$

$$\begin{aligned}
I_{xx} &= 2.1772\text{E}+10 \text{ mm}^4 \\
u &= 3200.0 \text{ mm} \\
A &= 280000 \text{ mm}^2 \\
u_i &= 3703.4 \text{ mm} \\
A_{ki} &= 164659 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 264.898 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.149 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 4.971 \text{ MPa}$$

$$N_{Ed} = 6840.356 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 114.7 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 32.5 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 2.291 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 5.6907\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 1796.5 \text{ mm}$$

$$A = 206000 \text{ mm}^2$$

$$u_i = 1431.6 \text{ mm}$$

$$A_{ki} = 116148 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 244.870 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.149 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 4.971 \text{ MPa}$$

$$N_{Ed} = 6840.356 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.146 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 3160.978 \text{ kN}$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : FZ-MIN

$$N_{Ed} = -6884.978 \text{ kN}$$

$$V_{Ed} = -421.845 \text{ kN}$$

$$T_{Ed} = 14.245 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 14891.711 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 73.1 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 28.3 \text{ mm}^2/\text{mm}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.987 \text{ kN} \cdot \text{m}$

$$I_{xx_i} = 1.9053\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 2584.0 \text{ mm}$$

$$A = 189000 \text{ mm}^2$$

$$u_i = 2281.5 \text{ mm}$$

$$A_{ki} = 91689 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin \theta \cdot \cos \theta = 59.053 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.150 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.003 \text{ MPa}$$

$$N_{Ed} = 6834.978 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 87.5 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 46.7 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.805 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 3.4845\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 3200.0 \text{ mm}$$

$$A = 280000 \text{ mm}^2$$

$$u_i = 3703.4 \text{ mm}$$

$$A_{ki} = 164659 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 265.122 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.150 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.003 \text{ MPa}$$

$$N_{Ed} = 6834.978 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 114.7 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm}$$

$$A_{st}/s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl}/s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 41.8 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 2.949 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 5.6907\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 1796.5 \text{ mm}$$

$$A = 206000 \text{ mm}^2$$

$$u_i = 1431.6 \text{ mm}$$

$$A_{ki} = 116148 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 245.078 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.150 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.003 \text{ MPa}$$

$$N_{Ed} = 6884.978 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

- Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.141 \leq 1.0 \quad \text{OK}$$

$$\text{where, } V_{Rd,max} = 3163.560 \text{ kN}$$

■ Maximum Torsion Moment

1 Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MX-MAX

$$N_{Ed} = -6929.812 \text{ kN}$$

$$V_{Ed} = -55.840 \text{ kN}$$

$$T_{Ed} = 38.928 \text{ kN} \cdot \text{m}$$

$$M_{Ed} = 14272.234 \text{ kN} \cdot \text{m}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 73.1 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 77.2 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 2.698 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.9053\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 2584.0 \text{ mm}$$

$$A = 189000 \text{ mm}^2$$

$$u_i = 2281.5 \text{ mm}$$

$$A_{ki} = 91689 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 59.103 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.151 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 5.036 \text{ MPa} \\ N_{Ed} &= 6929.812 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 1376161 \text{ mm}^2 \end{aligned}$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 87.5 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm} \\ A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 127.6 \text{ mm}^2/\text{mm} \end{aligned}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 4.934 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 3.4845\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 3200.0 \text{ mm}$$

$$A = 280000 \text{ mm}^2$$

$$u_i = 3703.4 \text{ mm}$$

$$A_{ki} = 164659 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 265.347 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\begin{aligned} \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.151 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 5.036 \text{ MPa} \\ N_{Ed} &= 6929.812 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 1376161 \text{ mm}^2 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 114.7 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 109.2 \text{ mm} \\ A_{st}/s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl}/s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 114.2 \text{ mm}^2/\text{mm} \end{aligned}$$

where, $T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 8.058 \text{ kN} \cdot \text{m}$

$$I_{xx,i} = 5.6907\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 2.1772\text{E}+10 \text{ mm}^4$$

$$u = 1796.5 \text{ mm}$$

$$A = 206000 \text{ mm}^2$$

$$u_i = 1431.6 \text{ mm}$$

$$A_{ki} = 116148 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 245.286 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where, $v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.151 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 5.036 \text{ MPa}$$

$$N_{Ed} = 6929.812 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1376161 \text{ mm}^2$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.040 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 3166.155 \text{ kN}$

5.Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB5

Design Situations : Quasi-permanent

Load Combination Type : -

Long/Short Term : Short-term

$$N_{Ed} = -5926.775 \text{ kN}$$

$$M_{Ed} = 539.329 \text{ kN} \cdot \text{m}$$

- Stress check (- : compression + : tension)

$$\sigma = -9.477 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4

Design Situations : Frequent

Load Combination Type : MY-MAX

Long/Short Term : Short-term

$$N_{Ed} = -5926.775 \text{ kN}$$

$$M_{Ed} = 539.329 \text{ kN} \cdot \text{m}$$

- Stress check (-: compression + : tension)

$$\sigma = 0.000 \text{ MPa} \leq \sigma_a = 0.000$$

OK

6. Crack Width

■ Top

1 Calculate crack width, w_k

(EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4

Serviceability Load Type : Frequent

Load Combination Type : MY-MIN

Long/Short Term : Short-term

$N_{Ed} = -5926.775 \text{ kN}$

$M_{Ed} = 539.329 \text{ kN} \cdot \text{m}$

- Stress at top surface

$$\sigma_{top} = -9.464 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
2.19	21.04	2.20	21.04	2.18	21.03	21.04	31.38	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f'_{ci}$

$f'_{ci} = 44.82176 \text{ (MPa)}$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
1.42	1.40	1.41	1.39	1.42	1.40	1.42	24.50	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f'_{ci}$

$f'_{ci} = 35 \text{ (MPa)}$

8. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
12.09	-4.31	12.07	-4.31	12.10	-4.30	12.10	30.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f'_c$

$f'_c = 42.85714 \text{ (MPa)}$

2) Slab

■ Compression

- Load Combination Name : cLCB3
- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
18.47	4.58	18.47	4.58	18.55	16.93	18.55	21.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 30$ (MPa)



intocmit ing. Attila Takacs



Proiectare si Executie "Varianta de Ocolire Timisoara Sud"

Grinda L=40.00m, h=2.00m

Element Number	112
Position Information	I

1.Design Condition

1.1 Design Parameters

- Partial factors for ultimate limit states

(EN 1992-1-1:2004, 2.4.2.4)

Design Situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.500	1.150	1.150
Accidental	1.200	1.000	1.000

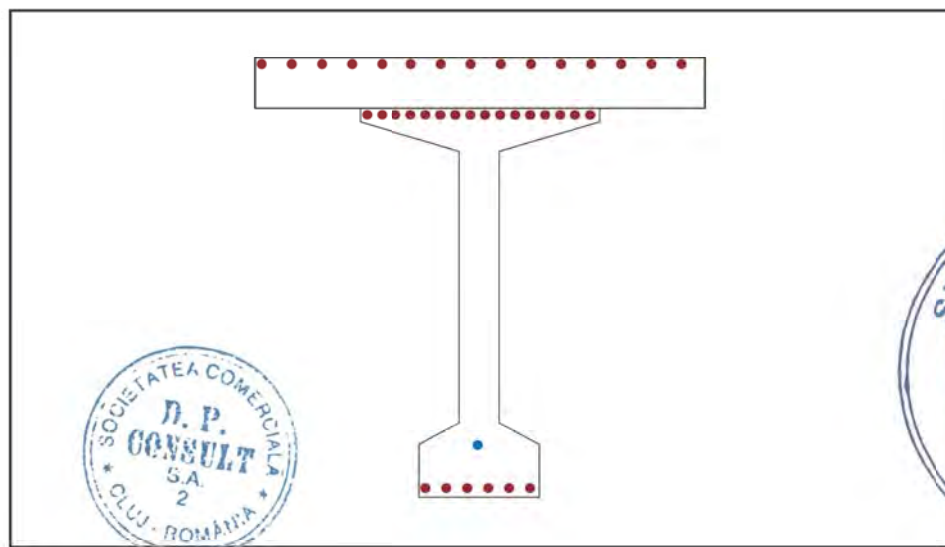
- factor α_{cc} , α_{ct} : Coefficient for long term effects on Compression and Tensile Strength.

α_{cc} = 0.850 (for the Compressive strength)

α_{ct} = 1.000 (for the Tensile strength)

1.2 Sectional Information

Section Information	EqvInt sect(tdn, reinfmnt) (Girder)	EqvInt sect (After Comp) (Girder + Slab)
A (mm ²)	725998.997	1268334.053
I_y (mm ⁴)	387469960817.002	778409105184.964
y_{st} (mm)	-	775.011
y_{sb} (mm)	-	515.011
y_t (mm)	986.449	515.011
y_b (mm)	1013.551	1484.989
Z_{st} (mm ³)	-	295681927.022
Z_{sb} (mm ³)	-	423140274.364
Z_t (mm ³)	42374803.014	423140274.364
Z_b (mm ³)	84749606.028	182711711.044



1.3 Material Information

■ Girder

- Concrete Material Information

(EN 1992-1-1:2004, Table 3.1)

$$f_{ck(g)} = 50.000 \text{ MPa}$$

$$E_{c(g)} = 37277.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 58.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 4.064 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.845 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(g)} = 500.000 \text{ MPa}, \quad E_{s(g)} = 200000.000 \text{ MPa}$$

■ Slab

(EN 1992-1-1:2004, Table 3.1)

- Concrete Material Information

$$f_{ck(s)} = 35.000 \text{ MPa}$$

$$E_{c(s)} = 34077.000 \text{ MPa}$$

$$f_{cm} = f_{ck} + 8 = 43.000 \text{ MPa}$$

$$f_{ctm} = 0.30 \cdot f_{cm}^{(2/3)} = 3.210 \text{ MPa} \quad (f_{ck} \leq C50/60)$$

$$f_{ctk;0.05} = 0.7 \cdot f_{ctm} = 2.247 \text{ MPa}$$

- Reinforcement Material Information

$$f_{yk(s)} = 500.000 \text{ MPa}, \quad E_{s(s)} = 200000.000 \text{ MPa}$$

1.4 Tendon Profile Information

Type	Tendon Name	Location (mm)	Area (mm ²)	Strength (MPa)		E _p (MPa)
				f _{pk}	f _{p0.1k}	
1	Tendon-Span1	2000.4	7489.800	1860.000	1600.000	195000.000

2. Ultimate Moment Resistance

■ Positive Moment

1 Check Moment Resistance, M_{R,d}

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MAX

$$M_{Ed} = 15257.845 \text{ kN} \cdot \text{m}$$

- factor λ, and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

1) Assume neutral axis depth.

2) Calculate the strain of steel and tendon.

3) Calculate the stress of steel and tendon.

- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	1130.0	19360.167	2136.287	524.504	10420.591	1.96403
2	565.0	16724.789	2136.287	524.504	10420.591	1.72324
3	282.5	10085.250	737.609	1923.183	10420.591	0.87679
4	423.8	14280.850	1732.426	524.504	10420.591	1.46305
5	353.1	12367.500	1391.027	524.504	10420.591	1.25705
6	317.8	11345.906	1163.427	524.504	10420.591	1.14292
7	300.2	10715.578	1029.673	1049.009	10420.591	1.02403
8	291.3	10400.414	737.609	1923.183	10420.591	0.90232
9	295.7	10557.996	737.609	1923.183	10420.591	0.91509
10	297.9	10636.787	737.609	1923.183	10420.591	0.92147
11	299.1	10676.183	737.609	1923.183	10420.591	0.92466
12	299.6	10695.880	737.609	1923.183	10420.591	0.92626
13	299.9	10705.729	737.609	1923.183	10420.591	0.92705
14	300.0	10710.654	912.443	1748.348	10420.591	0.95514
15	300.1	10713.116	1029.378	1398.678	10420.591	0.99350
16	300.1	10714.347	1029.508	1223.843	10420.591	1.00854
17	300.1	10713.731	1029.439	1311.261	10420.591	1.00096

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 0.000 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 10713.731 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 1029.439 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 1311.261 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 10420.591 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 240.084 \text{ mm} \\
 x &= 300.105 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 540188.141 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 3103.920 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 3015.900 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

Num	Stress _{Eff} (MPa)	$\Delta \epsilon_{pi}$	$\epsilon_{p(0)i}$	ϵ_{pi}	f_{pi} (MPa)	A_{pi} (mm ²)	F_{pi} (kN)
Tendon-Span1	1071.281	0.01983	0.00549	0.02532	1391.304	7489.800	10420.591
$F_p = \sum (F_{pi}) =$							10420.591

where, StressEff = $\epsilon_{p(0)i} \cdot E_p$

$\Delta \epsilon_{pi}$: concrete strain at tendon depth under fracture state

$\epsilon_{p(0)i}$: tendon strain under the effective Prestressing stress

$$\epsilon_{pi} = \Delta \epsilon_{pi} + \epsilon_{p(0)i}$$

$$f_{pi} = \min(\epsilon_{pi} E_{pi}, f_{pd})$$

$$f_{pd} = f_{p0,1k} / \gamma_s$$

$$F_{pi} = f_{pi} \cdot A_{pi}$$

- Calculate moment resistance, M_{Rd}

$$M_{Rd} = F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi})$$

$$= 20800.356 \text{ kN} \cdot \text{m} \geq M_{Ed} = 15257.845 \text{ kN} \cdot \text{m}$$

OK

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

■ Negative Moment

1 Check Moment Resistance, $M_{R,d}$

- Design Load

Load Combination Name : cLCB1

Design Situations : Persistent & Transient

Load Combination Type : MY-MIN

$$M_{Ed} = 0.000 \text{ kN} \cdot \text{m}$$

- factor λ , and factor η

$$\lambda_{(g)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\lambda_{(s)} = 0.800 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(g)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

$$\eta_{(s)} = 1.000 \quad (f_{ck} \leq 50 \text{ MPa})$$

- Design strength of concrete

(EN 1992-1-1:2004, 3.1.6(1))

$$\text{Girder : } f_{cd(g)} = \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 28.333 \text{ MPa}$$

$$\text{Slab : } f_{cd(s)} = \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 19.833 \text{ MPa}$$

- Design strength of Reinforcement

(EN 1992-1-1:2004, 3.2.7)

$$\text{Girder : } f_{yd(g)} = f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

$$\text{Slab : } f_{yd(s)} = f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

- Calculate Neutral Axis

- 1) Assume neutral axis depth.
- 2) Calculate the strain of steel and tendon.
- 3) Calculate the stress of steel and tendon.
- 4) Calculate the axial force in concrete, steel, and tendon.
- 5) Check if the resultant force of cross-section is zero.
- 6) Repeat step 1 through 5 until the resultant force becomes zero.

Num.	Neutral depth (mm)	Compression Force (C) (kN)		Tension Force (T) (kN)		Ratio (C/T)
		Concrete	Reinforcement	Reinforcement	Tendon	
1	1130.0	8806.000	524.504	2136.287	0.000	4.36763
2	565.0	6244.667	524.504	2136.287	0.000	3.16866
3	282.5	3842.000	524.504	2136.287	0.000	2.04397
4	141.3	1921.000	524.504	2136.287	10420.591	0.19475
5	211.9	2881.500	524.504	2136.287	9175.832	0.30109
6	247.2	3361.750	524.504	2136.287	8280.984	0.37306
7	264.8	3601.875	524.504	2136.287	0.000	1.93157
8	256.0	3481.813	524.504	2136.287	8095.843	0.39154
9	260.4	3541.844	524.504	2136.287	0.000	1.90347
10	258.2	3511.828	524.504	2136.287	8051.536	0.39619
11	259.3	3526.836	524.504	2136.287	8029.665	0.39852
12	259.9	3534.340	524.504	2136.287	0.000	1.89995

13	259.6	3530.588	524.504	2136.287	8024.226	0.39910
14	259.7	3532.464	524.504	2136.287	0.000	1.89907
15	259.7	3531.526	524.504	2136.287	0.000	1.89864
16	259.6	3531.057	524.504	2136.287	0.000	1.89842
17	259.6	3530.822	524.504	2136.287	8023.887	0.39914
18	259.6	3530.940	524.504	2136.287	8023.717	0.39916
19	259.6	3530.998	524.504	2136.287	0.000	1.89839
20	259.6	3530.940	524.504	2136.287	8023.717	0.39916

- Calculate $F_{c(g)}$, $F_{c(s)}$, F_s , F_p (EN 1992-1-1:2004, Fig 3.5, Fig 6.1)

$$\begin{aligned}
 F_{c(g)} &= (\eta_{(g)} \cdot f_{cd(g)}) \cdot A_{ca(g)} = 3530.940 \text{ kN} && : \text{Compression Force for Girder} \\
 F_{c(s)} &= (\eta_{(s)} \cdot f_{cd(s)}) \cdot A_{ca(s)} = 0.000 \text{ kN} && : \text{Compression Force for Slab} \\
 F_{s'} &= f_{s'} \cdot A_{s'} = 524.504 \text{ kN} && : \text{Compression reinforcement Force} \\
 F_s &= f_s \cdot A_s = 2136.287 \text{ kN} && : \text{Tension Force in reinforcement} \\
 F_p &= \sum F_{pi} = 8023.717 \text{ kN} && : \text{Tension Force in Tendon} \\
 \text{where, } a &= \lambda \cdot x = 207.704 \text{ mm} \\
 x &= 259.628 \text{ mm} && : \text{neutral axis depth} \\
 A_{ca(g)} &= 124621.399 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for girder part} \\
 A_{ca(s)} &= 0.000 \text{ mm}^2 && : \text{Area of rectangular con'c stress block for slab part} \\
 A_{s'} &= 1206.360 \text{ mm}^2 && : \text{Compression reinforcement areas} \\
 A_s &= 4913.460 \text{ mm}^2 && : \text{Tension reinforcement areas}
 \end{aligned}$$

- Calculate moment resistance, M_{Rd}

$$\begin{aligned}
 M_{Rd} &= F_{c(g)} \cdot a_{c(g)} + F_{c(s)} \cdot a_{c(s)} + F_s \cdot a_s + \sum (F_{pi} \cdot a_{pi}) \\
 &= 4489.341 \text{ kN} \cdot \text{m} \geq M_{Ed} = 0.000 \text{ kN} \cdot \text{m} \quad \text{OK}
 \end{aligned}$$

where,

$a_{c(g)}$, $a_{c(s)}$, a_s , a_p is the distance from the neutral axis to the centroid of the stress block in concrete, RF, TD

3. Shear Resistance

■ Maximum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{cLCB1} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{FZ-MAX} \\
 N_{Ed} &= -7739.765 \text{ kN} \\
 V_{Ed} &= 349.373 \text{ kN} \\
 M_{Ed} &= 13692.934 \text{ kN} \cdot \text{m} \\
 V_p &= 0.000 \text{ kN}
 \end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
 \text{Girder : } f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
 \text{Slab : } f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
 \end{aligned}$$

- Design strength of Reinforcement

$$\begin{aligned}
 \text{Girder : } f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
 \text{Slab : } f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
 \end{aligned}$$

- Check whether the section is cracked in bending.

$$f_b = 4.830 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa}$$

(The section is cracked.)

where, f_b : Flexural tensile stress (Tension is taken as positive.)

$f_{ctd} = \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c$: Design tensile strength of concrete.

- Design value for the shear resistance $V_{Rd,c}$

$$V_{Rd,min} = (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 569.321 \text{ kN}$$

$$V_{Rd,c} = [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 571.708 \text{ kN} \geq V_{Rd,min}$$

$$= 571.708 \text{ kN} \geq V_{Ed} = 349.373 \text{ kN} \quad \therefore \text{Shear reinforcement is not required}$$

where, $C_{Rd,c} = 0.18 / \gamma_c = 0.120$

$$v_{min} = 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.367$$

$$k_1 = 0.15$$

$$k = 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.300$$

$$\rho_1 = A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003$$

$$A_{sl} = 1206.360 \text{ mm}^2$$

$$b_w = 200.0 \text{ mm}$$

$$d_p = 2220.0 \text{ mm}$$

$$\sigma_{cp} = N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 6.102 \text{ MPa}$$

$$N_{Ed} = 7739.765 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1268334.053 \text{ mm}^2$$

2) Check minimum shear reinforcement

(EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\rho_{w,min} = 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113$$

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01540 \geq \rho_{w,min} = 0.00113 \quad \text{OK}$$

where, $\rho_{w,min}$ is the minimum ratio of shear reinforcement

$$A_{sw} = 308.000 \text{ mm}^2 \quad : \text{Area of diagonal reinforcement}$$

- Check Spacing of stirrups

$$S_{l,max} = 0.75 \cdot d \cdot (1 + \cot \alpha) = 1665.016 \text{ mm}$$

$$s = 100.000 \text{ mm} \leq S_{l,max} = 1665.016 \text{ mm} \quad \text{OK}$$

where, s is the spacing of the stirrups

$$d = 2220.022 \text{ mm} \quad : \text{effective depth}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\Delta F_{td} = 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 174.687 \text{ kN}$$

$$(M_{Rd} - M_{Ed}) / z = 4012.608 \text{ kN}$$

$$\Delta F_{td} \leq (M_{Rd} - M_{Ed}) / z \quad \text{OK}$$

where, $M_{Rd} = 20800.356 \text{ kN}$: The moment resistance

$z = 1771.273 \text{ mm}$: Lever arm from the ultimate bending resistance analysis

4) Check shear at the interface between concrete cast at different times

(EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$v_{Edi} = \beta \cdot V_{Ed} / (z \cdot b_i) = 0.164 \text{ MPa}$$

where, $\beta = 1.0$: Conservatively adopted to take $\beta = 1.0$

$$V_{Ed} = 349.373 \text{ kN} \quad : \text{Transverse shear force}$$

$$z = 1771.273 \text{ mm} \quad : \text{Lever arm from the ultimate bending resistance analysis}$$

$$b_i = 1200.000 \text{ mm} \quad : \text{Width of interface}$$

- Design shear resistance at the interface

$$\begin{aligned}
V_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) &= & 1.780 \text{ MPa} \\
V_{Rd,lim} &= 0.5 v f_{cd} &= & 8.000 \text{ MPa} \\
V_{Edi} &= 0.164 \text{ MPa} \leq V_{Rd} = \min[V_{Rdi}, V_{Rd,lim}] &= & 1.780 \text{ MPa} \quad \text{OK}
\end{aligned}$$

where,

$$\begin{aligned}
f_{ctd} &= 1.900 \text{ MPa} && \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
f_{cd} &= 33.333 \text{ MPa} && \text{of concrete between girder and slab} \\
c &= 0.200 && \text{: factors which depend on the roughness of the interface} \\
\mu &= 0.600 && \text{: factors which depend on the roughness of the interface} \\
\sigma_n &= w_s / A_i = 0.012 \text{ MPa} \\
\rho &= A_s / A_i = 0.005 \\
w_s &= 14.625 \text{ kN} && \text{: Weight of slab} \\
A_i &= 1200000.000 \text{ mm}^2 && \text{: Area of joint (1m for unit length)} \\
A_s &= 6408.000 \text{ mm}^2 && \text{: Area of reinforcement crossing the interface} \\
f_{yd} &= 434.783 \text{ MPa} && \text{: Strength of shear connector} \\
\alpha &= 90.000^\circ && \text{: Angle of reinforcement crossing the interface} \\
&&& (45^\circ \leq \alpha \leq 90^\circ)
\end{aligned}$$

■ Minimum Shear Force

1) Check shear resistance of Concrete, $V_{Rd,c}$

- Design Load

$$\begin{aligned}
\text{Load Combination Name :} & \text{ cLCB1} \\
\text{Design Situations :} & \text{ Persistent \& Transient} \\
\text{Load Combination Type :} & \text{ FZ-MIN} \\
N_{Ed} &= -7779.804 \text{ kN} \\
V_{Ed} &= -332.858 \text{ kN} \\
M_{Ed} &= 13176.955 \text{ kN} \cdot \text{m} \\
V_p &= 0.000 \text{ kN}
\end{aligned}$$

- Design strength of concrete

$$\begin{aligned}
\text{Girder :} \quad f_{cd(g)} &= \alpha_{cc} \cdot f_{ck(g)} / \gamma_c = 33.333 \text{ MPa} \\
\text{Slab :} \quad f_{cd(s)} &= \alpha_{cc} \cdot f_{ck(s)} / \gamma_c = 23.333 \text{ MPa}
\end{aligned}$$

- Design strength of Reinforcement

$$\begin{aligned}
\text{Girder :} \quad f_{yd(g)} &= f_{yk(g)} / \gamma_{s_rebar} = 434.783 \text{ MPa} \\
\text{Slab :} \quad f_{yd(s)} &= f_{yk(s)} / \gamma_{s_rebar} = 434.783 \text{ MPa}
\end{aligned}$$

- Check whether the section is cracked in bending.

$$\begin{aligned}
f_b &= 3.689 \text{ MPa} > f_{ctd} = 1.900 \text{ MPa} \\
&&& \text{(The section is cracked.)} \\
\text{where, } f_b &: \text{ Flexural tensile stress} && \text{(Tension is taken as positive.)} \\
f_{ctd} &= \alpha_{ct} \cdot f_{ctk,0.05} / \gamma_c && \text{: Design tensile strength of concrete.}
\end{aligned}$$

- Design value for the shear resistance $V_{Rd,c}$

$$\begin{aligned}
V_{Rd,min} &= (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b_w \cdot d_p = 571.423 \text{ kN} \\
V_{Rd,c} &= [C_{Rd,c} \cdot k \cdot (100 \cdot \rho_1 \cdot f_{ck})^{(1/3)} + k_1 \cdot \sigma_{cp}] \cdot b_w \cdot d_p = 573.811 \text{ kN} \geq V_{Rd,min} \\
&= 573.811 \text{ kN} \geq V_{Ed} = -332.858 \text{ kN} \quad \therefore \text{ Shear reinforcement is not required} \\
\text{where, } C_{Rd,c} &= 0.18 / \gamma_c = 0.120 \\
v_{min} &= 0.035 \cdot k^{(3/2)} \cdot f_{ck}^{(1/2)} = 0.367 \\
k_1 &= 0.15 \\
k &= 1 + \sqrt{(200 / d_p)} \leq 2.0 = 1.300 \\
\rho_1 &= A_{sl} / (b_w \cdot d_p) \leq 0.02 = 0.003
\end{aligned}$$

$$\begin{aligned}
A_{sl} &= 1206.360 \text{ mm}^2 \\
b_w &= 200.0 \text{ mm} \\
d_p &= 2220.0 \text{ mm} \\
\sigma_{ep} &= N_{Ed} / A_c \leq 0.2 \cdot f_{cd} = 6.134 \text{ MPa} \\
N_{Ed} &= 7779.804 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
A_c &= 1268334.053 \text{ mm}^2
\end{aligned}$$

2) Check minimum shear reinforcement (EN 1992-1-1:2004, 9.2.2(5))

- Check ratio of shear reinforcement

$$\begin{aligned}
\rho_{w, \min} &= 0.08 \cdot \sqrt{f_{ck}} / f_{yk} = 0.00113 \\
\rho_w &= A_{sw} / (s \cdot b_w \cdot \sin \alpha) = 0.01540 \geq \rho_{w, \min} = 0.00113 \quad \text{OK} \\
\text{where, } \rho_{w, \min} &\text{ is the minimum ratio of shear reinforcement} \\
A_{sw} &= 308.000 \text{ mm}^2 : \text{Area of diagonal reinforcement}
\end{aligned}$$

- Check Spacing of stirrups

$$\begin{aligned}
S_{l, \max} &= 0.75 \cdot d \cdot (1 + \cot \alpha) = 1665.016 \text{ mm} \\
s &= 100.000 \text{ mm} \leq S_{l, \max} = 1665.016 \text{ mm} \quad \text{OK} \\
\text{where, } s &\text{ is the spacing of the stirrups} \\
d &= 2220.022 \text{ mm} : \text{effective depth}
\end{aligned}$$

3) Check additional tension force in the longitudinal reinforcement due to shear force (EN 1992-1-1:2004, 6.2.3.(7))

$$\begin{aligned}
\Delta F_{td} &= 0.5 \cdot V_{Ed} \cdot (\cot \theta - \cot \alpha) = 166.429 \text{ kN} \\
(M_{Rd} - M_{Ed}) / z &= 4303.911 \text{ kN} \\
\Delta F_{td} &\leq (M_{Rd} - M_{Ed}) / z \quad \text{OK} \\
\text{where, } M_{Rd} &= 20800.356 \text{ kN} : \text{The moment resistance} \\
z &= 1771.273 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis}
\end{aligned}$$

4) Check shear at the interface between concrete cast at different times (EN 1992-1-1:2004, 6.2.5)

- Shear stress in the interface

$$\begin{aligned}
v_{Edi} &= \beta \cdot V_{Ed} / (z \cdot b_i) = 0.157 \text{ MPa} \\
\text{where, } \beta &= 1.0 : \text{Conservatively adopted to take } \beta = 1.0 \\
V_{Ed} &= -332.858 \text{ kN} : \text{Transverse shear force} \\
z &= 1771.273 \text{ mm} : \text{Lever arm from the ultimate bending resistance analysis} \\
b_i &= 1200.000 \text{ mm} : \text{Width of interface}
\end{aligned}$$

- Design shear resistance at the interface

$$\begin{aligned}
v_{Rdi} &= c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) = 1.780 \text{ MPa} \\
v_{Rd, \lim} &= 0.5 v f_{cd} = 8.000 \text{ MPa} \\
v_{Edi} &= 0.157 \text{ MPa} \leq v_{Rd} = \min[v_{Rdi}, v_{Rd, \lim}] = 1.780 \text{ MPa} \quad \text{OK} \\
\text{where, } f_{ctd} &= 1.900 \text{ MPa} : \text{Minimum design tensile strength}(f_{ctd}) \text{ and design strength}(f_{cd}) \\
f_{cd} &= 33.333 \text{ MPa} : \text{of concrete between girder and slab} \\
c &= 0.200 : \text{factors which depend on the roughness of the interface} \\
\mu &= 0.600 : \text{factors which depend on the roughness of the interface} \\
\sigma_n &= w_s / A_i = 0.012 \text{ MPa} \\
\rho &= A_s / A_i = 0.005 \\
w_s &= 14.625 \text{ kN} : \text{Weight of slab} \\
A_i &= 1200000.000 \text{ mm}^2 : \text{Area of joint (1m for unit length)} \\
A_s &= 6408.000 \text{ mm}^2 : \text{Area of reinforcement crossing the interface} \\
f_{yd} &= 434.783 \text{ MPa} : \text{Strength of shear connector} \\
\alpha &= 90.000^\circ : \text{Angle of reinforcement crossing the interface}
\end{aligned}$$

$$(45^\circ \leq \alpha \leq 90^\circ)$$

4. Torsional Resistance

■ Maximum Shear Force

1 Design Load

Load Combination Name : cLCB1
 Design Situations : Persistent & Transient
 Load Combination Type : FZ-MAX
 $N_{Ed} = -7739.765 \text{ kN}$
 $V_{Ed} = 349.373 \text{ kN}$
 $T_{Ed} = -8.825 \text{ kN} \cdot \text{m}$
 $M_{Ed} = 13692.934 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 73.1 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot \theta) = 0.0 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot \theta / (2 \cdot A_{ki} \cdot f_{yd}) = 20.6 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 0.720 \text{ kN} \cdot \text{m} \\ I_{xx_i} &= 1.9053\text{E}+09 \text{ mm}^4 \\ I_{xx} &= 1.9898\text{E}+10 \text{ mm}^4 \\ u &= 2584.0 \text{ mm} \\ A &= 189000 \text{ mm}^2 \\ u_i &= 2281.5 \text{ mm} \\ A_{ki} &= 91689 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin \theta \cdot \cos \theta = 61.201 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.192 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.398 \text{ MPa}$$

$$N_{Ed} = 7739.765 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u &= & 87.5 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} &= & 106.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) &= & 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) &= & 34.0 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} &= & 1.316 \text{ kN} \cdot \text{m} \\
 I_{xx,i} &= 3.4845\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 1.9898\text{E}+10 \text{ mm}^4 \\
 u &= 3200.0 \text{ mm} \\
 A &= 280000 \text{ mm}^2 \\
 u_i &= 3703.4 \text{ mm} \\
 A_{ki} &= 164659 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
 T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta &= & 274.767 \text{ kN} \cdot \text{m} \geq T_{Ed,i} && \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) &= & 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} &= & 1.192 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c &= & 6.398 \text{ MPa} \\
 N_{Ed} &= 7739.765 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 1209784 \text{ mm}^2
 \end{aligned}$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u &= & 114.7 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} &= & 106.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) &= & 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) &= & 30.5 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} &= & 2.149 \text{ kN} \cdot \text{m} \\
 I_{xx,i} &= 5.6907\text{E}+09 \text{ mm}^4 \\
 I_{xx} &= 1.9898\text{E}+10 \text{ mm}^4 \\
 u &= 1796.5 \text{ mm} \\
 A &= 206000 \text{ mm}^2 \\
 u_i &= 1431.6 \text{ mm} \\
 A_{ki} &= 116148 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 253.993 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.192 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.398 \text{ MPa}$$

$$N_{Ed} = 7739.765 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

- Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} = 0.110 \leq 1.0 \quad \text{OK}$$

where, $V_{Rd,max} = 3352.861 \text{ kN}$

■ Minimum Shear Force

1 Design Load

Load Combination Name : cLCB1
 Design Situations : Persistent & Transient
 Load Combination Type : FZ-MIN
 $N_{Ed} = -7779.804 \text{ kN}$
 $V_{Ed} = -332.858 \text{ kN}$
 $T_{Ed} = 12.583 \text{ kN} \cdot \text{m}$
 $M_{Ed} = 13176.955 \text{ kN} \cdot \text{m}$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 73.1 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 29.4 \text{ mm}^2/\text{mm}$$

where,

$$T_{Ed,i} = T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.026 \text{ kN} \cdot \text{m}$$

$$I_{xx_i} = 1.9053\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.9898\text{E}+10 \text{ mm}^4$$

$$u = 2584.0 \text{ mm}$$

$$A = 189000 \text{ mm}^2$$

$$u_i = 2281.5 \text{ mm}$$

$$A_{ki} = 91689 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 61.252 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.193 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.431 \text{ MPa}$$

$$N_{Ed} = 7779.804 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 87.5 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 48.5 \text{ mm}^2/\text{mm}$$

where,

$$T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 1.876 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 3.4845\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.9898\text{E}+10 \text{ mm}^4$$

$$u = 3200.0 \text{ mm}$$

$$A = 280000 \text{ mm}^2$$

$$u_i = 3703.4 \text{ mm}$$

$$A_{ki} = 164659 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 274.995 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

where,

$$v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.193 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.431 \text{ MPa}$$

$$N_{Ed} = 7779.804 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 114.7 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 43.4 \text{ mm}^2/\text{mm}$$

where,

$$T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 3.064 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 5.6907\text{E}+09 \text{ mm}^4$$

$$I_{xx} = 1.9898\text{E}+10 \text{ mm}^4$$

$$\begin{aligned}
 u &= 1796.5 \text{ mm} \\
 A &= 206000 \text{ mm}^2 \\
 u_i &= 1431.6 \text{ mm} \\
 A_{ki} &= 116148 \text{ mm}^2 \\
 t_{cover,i} &= 40.0 \text{ mm} \\
 t_{sect,lim} &= 200.0 \text{ mm}
 \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned}
 T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 254.205 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\
 \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\
 \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.193 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\
 \sigma_{cp} &= N_{Ed} / A_c = 6.431 \text{ MPa} \\
 N_{Ed} &= 7779.804 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\
 A_c &= 1209784 \text{ mm}^2
 \end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$\begin{aligned}
 T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} &= 0.108 \leq 1.0 \quad \text{OK} \\
 \text{where, } V_{Rd,max} &= 3355.545 \text{ kN}
 \end{aligned}$$

■ Maximum Torsion Moment

1 Design Load

$$\begin{aligned}
 \text{Load Combination Name : } & \text{cLCB1} \\
 \text{Design Situations : } & \text{Persistent \& Transient} \\
 \text{Load Combination Type : } & \text{MX-MAX} \\
 N_{Ed} &= -7875.981 \text{ kN} \\
 V_{Ed} &= -199.307 \text{ kN} \\
 T_{Ed} &= 23.256 \text{ kN} \cdot \text{m} \\
 M_{Ed} &= 12676.962 \text{ kN} \cdot \text{m}
 \end{aligned}$$

- Design strength of concrete

$$f_{cd} = \alpha_{cc} \cdot f_{ck} / \gamma_c = 33.333 \text{ MPa}$$

- Design strength of Reinforcement

$$f_{yd} = f_{yk} / \gamma_{s_rebar} = 434.783 \text{ MPa}$$

2 Check Torsional Resistance

■ Top Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned}
 t_{ef,i} &= A / u = 73.1 \text{ mm} \\
 t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\
 A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm} \\
 A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 54.3 \text{ mm}^2/\text{mm} \\
 \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx_i} / I_{xx} = 1.896 \text{ kN} \cdot \text{m} \\
 I_{xx_i} &= 1.9053\text{E}+09 \text{ mm}^4
 \end{aligned}$$

$$\begin{aligned}
I_{xx} &= 1.9898E+10 \text{ mm}^4 \\
u &= 2584.0 \text{ mm} \\
A &= 189000 \text{ mm}^2 \\
u_i &= 2281.5 \text{ mm} \\
A_{ki} &= 91689 \text{ mm}^2 \\
t_{cover,i} &= 40.0 \text{ mm} \\
t_{sect,lim} &= 200.0 \text{ mm}
\end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 61.374 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.195 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.510 \text{ MPa}$$

$$N_{Ed} = 7875.981 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

■ Web

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$t_{ef,i} = A / u = 87.5 \text{ mm}$$

$$t_i = \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm}$$

$$A_{st} / s_t = T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.0 \text{ mm}^2/\text{mm}$$

$$A_{sl} / s_l = T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 89.7 \text{ mm}^2/\text{mm}$$

$$\text{where, } T_{Ed,i} = T_{Ed} \cdot I_{xx,i} / I_{xx} = 3.468 \text{ kN} \cdot \text{m}$$

$$I_{xx,i} = 3.4845E+09 \text{ mm}^4$$

$$I_{xx} = 1.9898E+10 \text{ mm}^4$$

$$u = 3200.0 \text{ mm}$$

$$A = 280000 \text{ mm}^2$$

$$u_i = 3703.4 \text{ mm}$$

$$A_{ki} = 164659 \text{ mm}^2$$

$$t_{cover,i} = 40.0 \text{ mm}$$

$$t_{sect,lim} = 200.0 \text{ mm}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$T_{Rd,maxi} = 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{ki} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 275.545 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK}$$

$$\text{where, } v = 0.6 \cdot (1 - f_{ck} / 250) = 0.480$$

$$\alpha_{cw} = 1 + \sigma_{cp} / f_{cd} = 1.195 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd})$$

$$\sigma_{cp} = N_{Ed} / A_c = 6.510 \text{ MPa}$$

$$N_{Ed} = 7875.981 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0)$$

$$A_c = 1209784 \text{ mm}^2$$

■ Bottom Flange

- Design Parameters

(EN 1992-1-1:2004+A1:2014 cl. 6.3.2 (1))

$$\begin{aligned} t_{ef,i} &= A / u = 114.7 \text{ mm} \\ t_i &= \max[t_{ef,i}, 2t_{cover,i}] < t_{sect,lim} = 106.8 \text{ mm} \\ A_{st} / s_t &= T_{Ed,i} / (2 \cdot A_{ki} \cdot f_{yd} \cdot \cot\theta) = 0.1 \text{ mm}^2/\text{mm} \\ A_{sl} / s_l &= T_{Ed,i} \cdot \cot\theta / (2 \cdot A_{ki} \cdot f_{yd}) = 80.3 \text{ mm}^2/\text{mm} \\ \text{where, } T_{Ed,i} &= T_{Ed} \cdot I_{xx,i} / I_{xx} = 5.663 \text{ kN} \cdot \text{m} \\ I_{xx,i} &= 5.6907\text{E}+09 \text{ mm}^4 \\ I_{xx} &= 1.9898\text{E}+10 \text{ mm}^4 \\ u &= 1796.5 \text{ mm} \\ A &= 206000 \text{ mm}^2 \\ u_i &= 1431.6 \text{ mm} \\ A_{ki} &= 116148 \text{ mm}^2 \\ t_{cover,i} &= 40.0 \text{ mm} \\ t_{sect,lim} &= 200.0 \text{ mm} \end{aligned}$$

$t_{ef,i}$ is the effective wall thickness

u is the outer circumference of the cross-section

A is the total area of the cross-section within the outer circumference, including inner hollow areas

u_i is the outer circumference of the cross-section

A_{ki} is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

- Check the crushing limit for combined shear and torsion

$$\begin{aligned} T_{Rd,maxi} &= 2 \cdot v \cdot \alpha_{cw} \cdot f_{cd} \cdot A_{k,i} \cdot t_i \cdot \sin\theta \cdot \cos\theta = 254.713 \text{ kN} \cdot \text{m} \geq T_{Ed,i} \quad \text{OK} \\ \text{where, } v &= 0.6 \cdot (1 - f_{ck} / 250) = 0.480 \\ \alpha_{cw} &= 1 + \sigma_{cp} / f_{cd} = 1.195 \quad (\text{for } 0 < \sigma_{cp} \leq 0.25 \cdot f_{cd}) \\ \sigma_{cp} &= N_{Ed} / A_c = 6.510 \text{ MPa} \\ N_{Ed} &= 7875.981 \text{ kN} \quad (\text{if compression, } N_{Ed} > 0) \\ A_c &= 1209784 \text{ mm}^2 \end{aligned}$$

■ Calculate maximum torsion resistance, $T_{Rd,max}$ of total section

$$\begin{aligned} T_{Ed} / \sum T_{Rd,max} + V_{Ed} / V_{Rd,max} &= 0.075 \leq 1.0 \quad \text{OK} \\ \text{where, } V_{Rd,max} &= 3361.992 \text{ kN} \end{aligned}$$

5. Decompression check

■ Top

(EN 1992-1-1:2004, 7.3.1)

- Design Load

$$\begin{aligned} \text{Load Combination Name : } & \text{cLCB5} \\ \text{Design Situations : } & \text{Quasi-permanent} \\ \text{Load Combination Type : } & - \\ \text{Long/Short Term : } & \text{Short-term} \\ N_{Ed} &= -8002.759 \text{ kN} \\ M_{Ed} &= 1234.449 \text{ kN} \cdot \text{m} \end{aligned}$$

- Stress check (- : compression + : tension)

$$\sigma = -8.042 \text{ MPa} \leq \sigma_a = 0.000 \quad \text{OK}$$

■ Bottom

(EN 1992-1-1:2004, 7.3.1)

- Design Load

Load Combination Name : cLCB4
 Design Situations : Frequent
 Load Combination Type : MY-MAX
 Long/Short Term : Short-term
 $N_{Ed} = -7880.743 \text{ kN}$
 $M_{Ed} = 4010.047 \text{ kN} \cdot \text{m}$

- Stress check (- : compression + : tension)

$$\sigma = -4.910 \text{ MPa} \leq \sigma_a = 0.000$$

OK

6. Crack Width

■ Top

1 Calculate crack width, w_k

(EN 1992-1-1:2004, 7.3.4)

- Design Load

Load Combination Name : cLCB4
 Serviceability Load Type : Frequent
 Load Combination Type : MY-MIN
 Long/Short Term : Short-term
 $N_{Ed} = -8003.947 \text{ kN}$
 $M_{Ed} = 1213.008 \text{ kN} \cdot \text{m}$

- Stress at top surface

$$\sigma_{top} = -8.028 \text{ MPa} \quad (\text{For compressive stresses, the crack check is skipped.})$$

7. Stress check at transfer and during construction

1) Girder

■ Compression

- Critical Stage Name : Stage1

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
3.97	20.54	3.98	20.54	3.96	20.53	20.54	31.38	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f'_{ci}$$

$$f'_{ci} = 44.82176 \text{ (MPa)}$$

2) Slab

■ Compression

- Critical Stage Name : Stage4

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
1.36	1.83	1.36	1.82	1.36	1.83	1.83	24.50	OK

For post-tensioned elements $k_6 = 0.70$

$$ALW = \text{Limiting stress} = k_6 \cdot f'_{ci}$$

$$f'_{ci} = 35 \text{ (MPa)}$$

8. Stress check for service load combination

1) Girder

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
11.06	0.67	11.15	0.72	10.97	0.63	11.15	30.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 42.85714$ (MPa)



2) Slab

■ Compression

- Load Combination Name : cLCB3

- Load Combination Type : MY-MAX

FT (MPa)	FB (MPa)	FTL (MPa)	FBL (MPa)	FTR (MPa)	FBR (MPa)	FMAX (MPa)	ALW (MPa)	OK/NG
18.68	5.38	18.67	5.38	18.37	17.25	18.68	21.00	OK

For post-tensioned elements $k_6 = 0.70$

ALW = Limiting stress = $k_6 \cdot f_c$

$f_c = 30$ (MPa)

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