

COMMITTENTE



PROGETTAZIONE:



**DIREZIONE TECNICA  
U.O. INFRASTRUTTURE CENTRO**

**PROGETTO DEFINITIVO**

**RADDOPPIO CESANO – VIGNA DI VALLE**

**CAVALCAFERROVIA per soppressione PL km 29+500**

Struttura scatolare - Relazione di calcolo

SCALA:

-

COMMESSA	LOTTO	FASE	ENTE	TIPO DOC.	OPERA/DISCIPLINA	PROGR.	REV.
NR1J	01	D	29	CL	IV0100	001	A

Rev.	Descrizione	Redatto	Data	Verificato	Data	Approvato	Data	Autorizzato Data
A	EMISSIONE ESECUTIVA	F. Serrau <i>F. Serrau</i>	10-2018	M. Monda G. Passaro	10-2018	T. Paoletti <i>T. Paoletti</i>	10-2018	<i>F. Arduini</i> ITALFERR S.p.A. Direzione Tecnica Infrastrutture Centro Via Roma - Palazzo Arduini 00187 Roma - Italia Tel. +39 06 478392 4001 Fax +39 06 478392 4002

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

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## 1. PREMESSA

Il progetto delle opere d'arte è stato redatto sulla base delle impostazioni ed esigenze espresse dal progetto della viabilità, uniformandosi a quest'ultimo per quanto riguarda ubicazione ed ampiezza del cavalcavia, oltre che, ovviamente, per quanto concerne le rispettive caratteristiche planimetriche ed altimetriche, riservando particolare attenzione ai franchi orizzontali e verticali minimi prescritti dalla normativa vigente.

In particolare la presente relazione riporta il dimensionamento delle strutture scatolari in calcestruzzo armato gettato in opera previste nelle rampe di accesso al cavalcavia stradale IV01 avente una larghezza della carreggiata 9.00 m. Lo scavalco è composto da un cavalcavia a tre campate in c.a.p. a luci uguali pari a 25m e con pile e spalle fondate su fondazioni profonde. Al fine di limitare gli scarichi sui terreni di fondazione dei rilevati stradali in terra si è deciso di ridurre la dimensione e lunghezza dei rilevati stessi sostituendoli con rampe di approccio in struttura scatolare.

La presente relazione ha per oggetto le verifiche secondo il metodo semiprobabilistico agli Stati Limite (S.L.).

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## 2. DOCUMENTAZIONE DI RIFERIMENTO

### 2.1 Normativa di riferimento

Le analisi strutturali e le verifiche di sicurezza sono effettuate in accordo con le prescrizioni di seguito elencate è conformi alle normative vigenti:

- ✓ D.M. 17.01.2018 Aggiornamento delle “Norme tecniche per le costruzioni”;
- ✓ Circolare n. 617 del 2 febbraio 2009 - Istruzioni per l’Applicazione Nuove Norme Tecniche Costruzioni di cui al Decreto Ministeriale 14 gennaio 2008;
- ✓ RFI DTC SI CS MA IFS 001 B del 22-12-17: “Manuale di Progettazione delle Opere Civili” (MdP 2018) Parte II - Sezione 3: Corpo stradale;
- ✓ RFI DTC SI CS MA IFS 001 B del 22-12-17: “Manuale di Progettazione delle Opere Civili” (MdP 2018) Parte II - Sezione 2: Ponti e strutture;
- ✓ UNI EN 206-1:2016. “Calcestruzzo – Parte 1: Specificazione, prestazione, produzione e conformità”;
- ✓ UNI 11104-2016: Calcestruzzo,” Classi di esposizione per calcestruzzo strutturale, in funzione delle condizioni ambientali”;
- ✓ UNI EN 1992-1-1:2015. “Eurocodice 2: Progettazione delle strutture in calcestruzzo – Parte 1-1: Regole generali e regole per gli edifici”;
- ✓ UNI EN 1997-1:2013 – “Eurocodice 7 – Progettazione geotecnica - Parte 1: Regole generali”;
- ✓ UNI EN 1998-5:2005 – “Eurocodice 8 – Progettazione delle strutture per la resistenza sismica - Parte 5: Fondazioni, strutture di contenimento ed aspetti geotecnici”.

### 2.2 Elaborati di progetto di riferimento

IV01 - Relazione tecnica descrittiva	N	R	1	J	0	1	D	2	9	R	O	I	V	0	1	0	0	0	0	1	A
IV01 - Planimetria di progetto	N	R	1	J	0	1	D	2	9	P	8	I	V	0	1	0	0	0	0	1	A
IV01 - Relazione di calcolo impalcato	N	R	1	J	0	1	D	2	9	C	L	I	V	0	1	0	7	0	0	1	A
IV01 - Relazione di calcolo spalla	N	R	1	J	0	1	D	2	9	C	L	I	V	0	1	0	4	0	0	1	A
IV01 - Relazione di calcolo pila	N	R	1	J	0	1	D	2	9	C	L	I	V	0	1	0	5	0	0	1	A

**RELAZIONE DI CALCOLO**

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IV01 - Relazione di calcolo scatolari di appoggio	N	R	1	J	0	1	D	2	9	C	L	I	V	0	1	0	0	0	0	1	A
IV01 - Fasi costruttive - Tav. 1/2	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	0	0	0	3	A
IV01 - Fasi costruttive - Tav. 2/2	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	0	0	0	4	A
IV01 - Scavi, demolizioni - Pianta e sezioni	N	R	1	J	0	1	D	2	9	P	Z	I	V	0	1	0	0	0	0	1	A
IV01 - Pianta fondazioni, pianta impalcato, prospetto e sezione longitudinale	N	R	1	J	0	1	D	2	9	P	Z	I	V	0	1	0	0	0	0	2	A
IV01 - Carpenteria impalcato	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	7	0	0	1	A
IV01 - Carpenteria Spalla A	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	4	0	0	1	A
IV01 - Carpenteria Spalla B	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	4	0	0	2	A
IV01 - Carpenteria Pila P1	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	5	0	0	1	A
IV01 - Carpenteria Pila P2	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	5	0	0	2	A
IV01 - Sezioni tipo struttura scatolare	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	0	0	0	5	A
IV01 - Apparecchi d'appoggio e giunti	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	0	0	0	1	A
IV01 - Particolari costruttivi	N	R	1	J	0	1	D	2	9	B	Z	I	V	0	1	0	0	0	0	2	A
IV01 - Relazione di calcolo opere provvisionali	N	R	1	J	0	1	D	2	9	C	L	I	V	0	1	0	0	0	0	1	A



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### 3. CARATTERISTICHE MATERIALI

#### 3.1 Calcestruzzo

##### 3.1.1 Magrone

Classe di resistenza = C12/15

##### 3.1.2 Calcestruzzo per piedritti e soletta di copertura

$\gamma_c$  = peso specifico = 25.00 kN/m<sup>3</sup>

Classe di resistenza = C32/40

$R_{ck}$  = resistenza cubica = 40.00 N/mm<sup>2</sup>

$f_{ck}$  = resistenza cilindrica caratteristica = 0.83  $R_{ck}$  = 32.0 N/mm<sup>2</sup>

$f_{cm}$  = resistenza cilindrica media =  $f_{ck} + 8$  = 40.0 N/mm<sup>2</sup>

$f_{cd}$  = resistenza di calcolo a compressione =  $\alpha_{cc} * f_{ck} / \gamma_c$  = 0.85 \* 32/1.5 = 18.1 N/mm<sup>2</sup>

$f_{ctm}$  = resistenza a trazione media = 0.30 \*  $f_{ck}^{(2/3)}$  = 3.02 N/mm<sup>2</sup>

$f_{ctm}$  = resistenza a traz. per flessione media = 1.20 \*  $f_{ctm}$  = 3.63 N/mm<sup>2</sup>

$f_{ctk}$  = resistenza a traz. per flessione caratt. = 0.70 \*  $f_{ctm}$  = 2.12 N/mm<sup>2</sup>

$E_{cm}$  = modulo elast. tra 0 e 0.40 $f_{cm}$  = 22000 \*  $(f_{cm}/10)^{0.3}$  = 33345 N/mm<sup>2</sup>

$\sigma_c$  = Resistenza a compressione (Comb. Rara) = 0.6 \*  $f_{ck}$  = 19.2 N/mm<sup>2</sup>

$\sigma_c$  = Resistenza a compressione (Comb. Quasi Permanente) = 0.45 \*  $f_{ck}$  = 14.4 N/mm<sup>2</sup>

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### 3.1.3 Calcestruzzo per soletta di fondazione

$\gamma_c$  = peso specifico = 25.00 kN/m<sup>3</sup>

Classe di resistenza = C25/30

$R_{ck}$  = resistenza cubica = 30.00 N/mm<sup>2</sup>

$f_{ck}$  = resistenza cilindrica caratteristica = 0.83  $R_{ck}$  = 24.9 N/mm<sup>2</sup>

$f_{cm}$  = resistenza cilindrica media =  $f_{ck} + 8$  = 33.0 N/mm<sup>2</sup>

$f_{cd}$  = resistenza di calcolo a compressione =  $\alpha_{cc} * f_{ck} / \gamma_c$  = 0.85 \* 25/15 = 14.16 N/mm<sup>2</sup>

$f_{ctm}$  = resistenza a trazione media = 0.30 \*  $f_{ck}^{(2/3)}$  = 2.56 N/mm<sup>2</sup>

$f_{ctm}$  = resistenza a traz. per flessione media = 1.20 \*  $f_{ctm}$  = 3.08 N/mm<sup>2</sup>

$f_{ctk}$  = resistenza a traz. per flessione caratt. = 0.70 \*  $f_{ctm}$  = 1.79 N/mm<sup>2</sup>

$E_{cm}$  = modulo elast. tra 0 e 0.40 $f_{cm}$  = 22000 \* ( $f_{cm}/10$ )<sup>0.3</sup> = 31475 N/mm<sup>2</sup>

$\sigma_c$  = Resistenza a compressione (Comb. Rara) = 0.6 \*  $f_{ck}$  = 15.0 N/mm<sup>2</sup>

$\sigma_c$  = Resistenza a compressione (Comb. Quasi Permanente) = 0.45 \*  $f_{ck}$  = 11.25 N/mm<sup>2</sup>

## 3.2 Acciaio

### 3.2.1 Acciaio di armatura per c.a.

Tipo = B 450 C

$\gamma_a$  = peso specifico = 78.50 kN/m<sup>3</sup>

$f_{y\ nom}$  = tensione nominale di snervamento = 450 N/mm<sup>2</sup>

$f_{t\ nom}$  = tensione nominale di rottura = 540 N/mm<sup>2</sup>

$f_{yk\ min}$  = minima tensione caratteristica di snervamento = 450 N/mm<sup>2</sup>

$f_{tk\ min}$  = minima tensione caratteristica di rottura = 540 N/mm<sup>2</sup>

$(f_t/f_y)_{k\ min}$  = minimo rapporto tra i valori caratteristici = 1.15

$(f_t/f_y)_{k\ max}$  = massimo rapporto tra i valori caratteristici = 1.35

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$(f_y/f_{y,nom})_k$  = massimo rapporto tra i valori nominali = 1.25

$(A_{gt})_k$  = allungamento caratteristico sotto carico massimo = 7.5 %

$\varphi_{min}$  = minimo diametro consentito delle barre = 6 mm

$\varphi_{max}$  = massimo diametro consentito delle barre = 40 mm

E = modulo di elasticità dell'acciaio = 206000 N/mm<sup>2</sup>

$\alpha_T$  = coefficiente di dilatazione termica = 0.00001 °C<sup>-1</sup>

Qualora la classe di resistenza del calcestruzzo adottata per le analisi di calcolo e per le verifiche dei vari elementi strutturali risulti inferiore a quella indicata nel documento *Tabella Materiali e Note Generali – NR1J01D29EPOC0000001A* è da ritenere valido quanto indicato in quest'ultimo documento cioè in *Tabella Materiali e Note Generali – NR1J01D29EPOC0000001A*.

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#### 4. DESCRIZIONE DELL'OPERA

Il cavalcaferrovia in oggetto risulta sostanzialmente costituito da rampe scatoari in c.a. di approdo all'opera di scavalco della sede ferroviaria costituita da un viadotto a quattro campate a struttura in c.a.p. di luce pari a 1x11.10, 1x10.57 m e 3x20.00 m . Le rampe di approdo all'opera di scavalco sono realizzate mediante strutture scatoari in conglomerato cementizio armato gettato in opera. La semilarghezza netta interna della generica sezione trasversale è 7.60m . L'altezza interna risulta variabile da 6.10 m a 8.40 m. Lo spessore della soletta superiore è 0.80m, dei piedritti è pari a 0.80m e del solettone di fondo è pari a 1.00m. I due marciapiedi laterali ricadono invece su solette a sbalzo dallo scatoare di luce di 1.75m. Il manufatto è suddiviso in conci aventi lunghezza massima di circa 20.0m. Una parete trasversale di chiusura separa il primo concio dal rilevato di approdo alla rampa. Il dimensionamento è stato condotto con riferimento al concio 5 considerando un'altezza di calcolo pari a 10.20 m ed una struttura piana rappresentativa di una striscia trasversale dell'opera avente lunghezza pari a 1.0m.

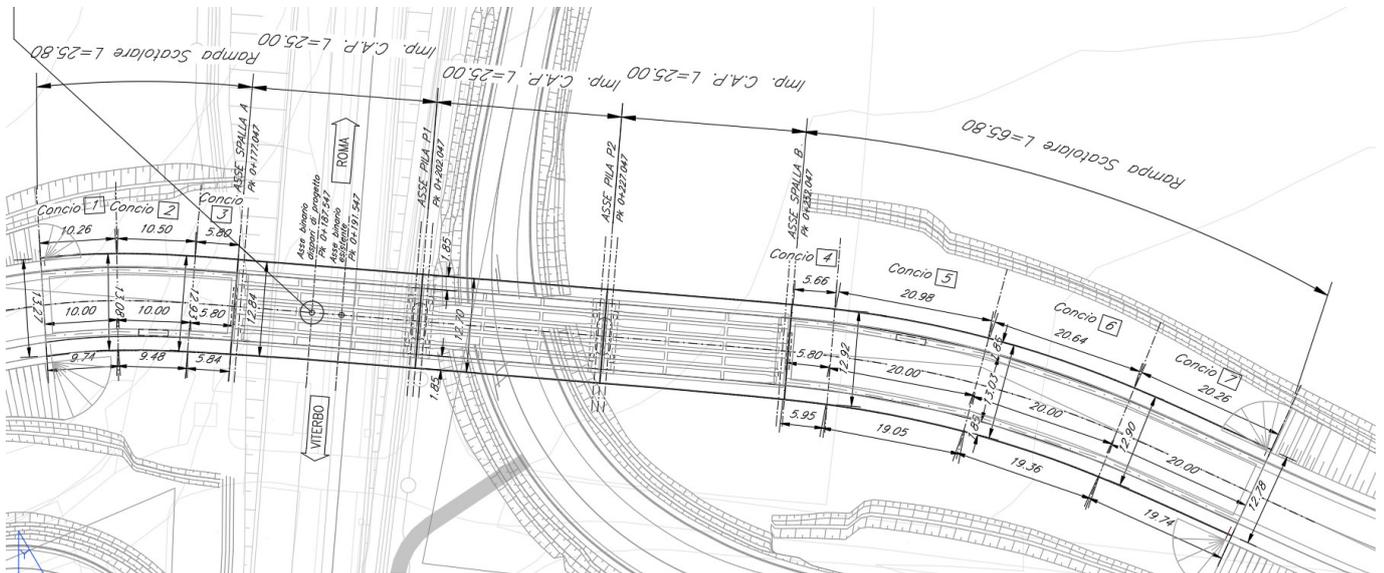


Fig. 1 – Planimetria generale con inquadramento dell'opera

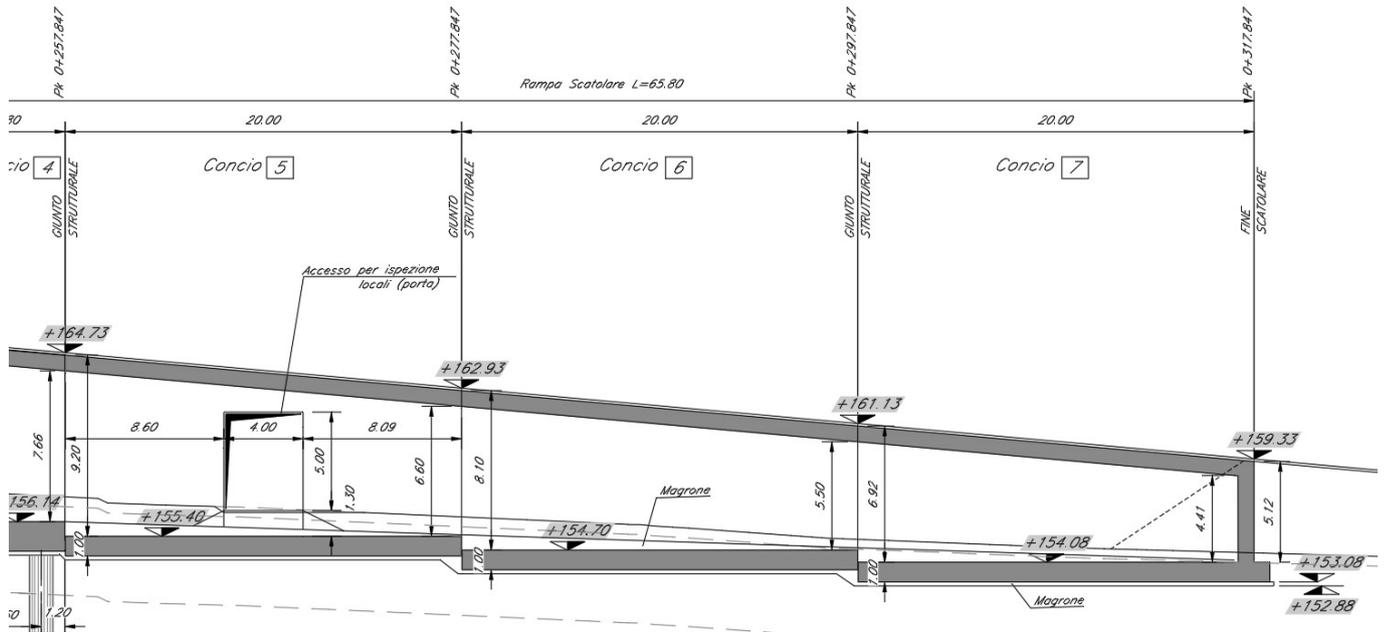


Fig. 2 – Sezione longitudinale – Rampa lato Trieste

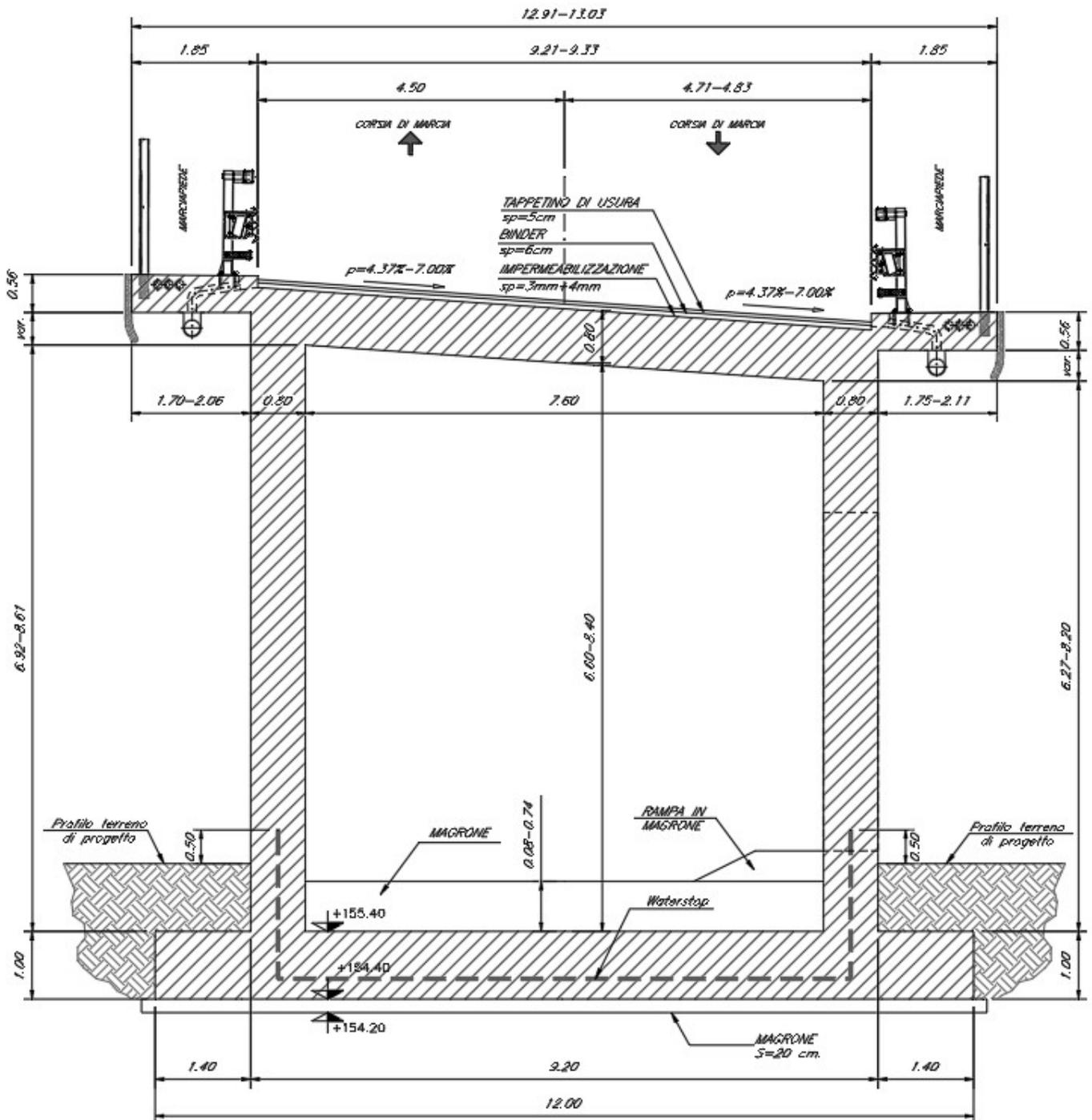


Fig. 3 – Sezione trasversale di calcolo – sezione

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## 5. INQUADRAMENTO GEOTECNICO

Per l'inquadramento geotecnico dell'opera si riporta l'estratto dell'elaborato Relazione Geotecnica [NR1J00D29GEGE0005001A] di cui si riporta un estratto per i dati geotecnici di interesse.

Tabelle contenenti la stratigrafia di progetto per l'opera in esame e i relativi parametri geotecnici di calcolo:

Modello geotecnico n° 2, dalla pk 29+050 alla pk 29+900:

Strato	Profondità da (m da p.c.)	Profondità a (m da p.c.)	Descrizione	Peso di volume $\gamma$ [kN/m <sup>3</sup> ]	Tipo di terreno	Angolo di resistenza al taglio $\phi'$ (°)	C' (kPa)	Cu (kPa)	Modulo elastico Eop (MPa)
1	0.00	2.00	Terreno prevalentemente vegetale, depositi vulcanici incoerenti	16.00	-	-	-	-	-
2	2.00	6.50	Depositi vulcanici debolmente addensato con inclusi litici eterogenei	18.00	GG	28.00	10.00	35.00	35.00
3	16.00	30.00	Sabbia fine debolmente limosa da addensata a molto addensata	18.00	GG	30.00	0.00	0.00	25.00
Falda: 8.7 m da b.f									
Categoria di suolo: E									

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## 6. MODELLO DI CALCOLO

La struttura viene schematizzata con un modello di calcolo a telaio chiuso su un letto di molle alla Winkler mediante un'analisi elastico-lineare svolta con il programma di calcolo agli elementi finiti SAP2000 v.20.1 (Computers and Structures®).

Gli elementi frame che schematizzano il telaio piano hanno una sezione rettangolare di larghezza 1.0m ed altezza pari a:

- soletta superiore  $h = 0.80\text{m}$
- piedritti  $h = 0.80\text{ m}$
- soletta di fondazione  $h = 1.00\text{ m}$

Il modulo elastico del materiale assegnato agli elementi asta è assunto:

- Struttura in elevazione  $E = 33346\text{ N/mm}^2$  :  $\text{cls Rck} = 40\text{N/mm}^2$
- Soletta di fondazione  $E = 31476\text{ N/mm}^2$ :  $\text{cls Rck} = 30\text{N/mm}^2$

Nelle successive figure si riporta lo schema di calcolo adottato con la numerazione dei nodi (Figura 4) .

L'opera è stata considerata vincolata alla base mediante dei vincoli fissi a simulare la presenza dei pali di fondazione.



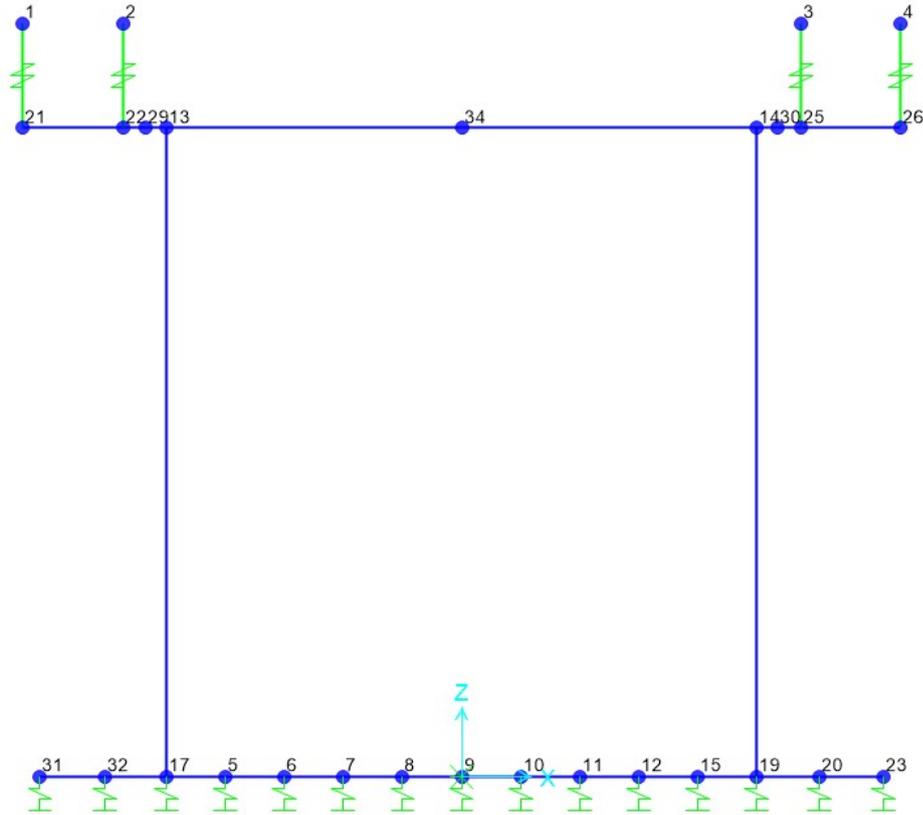


Fig. 4 – Modello di calcolo – numerazione dei nodi

TABLE: Joint Coordinates					
Joint	CoordSys	CoordType	GlobalX	GlobalY	GlobalZ
Text	Text	Text	m	m	m
1	GLOBAL	Cartesian	-6.25	0	10.8
2	GLOBAL	Cartesian	-4.82	0	10.8
3	GLOBAL	Cartesian	4.83	0	10.8
4	GLOBAL	Cartesian	6.25	0	10.8
5	GLOBAL	Cartesian	-3.36	0	0
6	GLOBAL	Cartesian	-2.52	0	0
7	GLOBAL	Cartesian	-1.68	0	0
8	GLOBAL	Cartesian	-0.84	0	0
9	GLOBAL	Cartesian	-1.776E-15	0	0
10	GLOBAL	Cartesian	0.84	0	0
11	GLOBAL	Cartesian	1.68	0	0
12	GLOBAL	Cartesian	2.52	0	0
13	GLOBAL	Cartesian	-4.2	0	9.3
14	GLOBAL	Cartesian	4.2	0	9.3

**RELAZIONE DI CALCOLO**

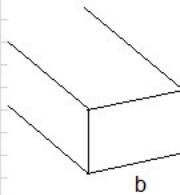
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15	GLOBAL	Cartesian	3.36	0	0
17	GLOBAL	Cartesian	-4.2	0	0
19	GLOBAL	Cartesian	4.2	0	0
20	GLOBAL	Cartesian	5.1	0	0
21	GLOBAL	Cartesian	-6.25	0	9.3
22	GLOBAL	Cartesian	-4.82	0	9.3
23	GLOBAL	Cartesian	6	0	0
25	GLOBAL	Cartesian	4.83	0	9.3
26	GLOBAL	Cartesian	6.25	0	9.3
29	GLOBAL	Cartesian	-4.5	0	9.3
30	GLOBAL	Cartesian	4.5	0	9.3
31	GLOBAL	Cartesian	-6	0	0
32	GLOBAL	Cartesian	-5.08	0	0
34	GLOBAL	Cartesian	-1.776E-15	0	9.3

La fondazione è appoggiata su un insieme di molle elastiche che simulano la risposta del terreno di fondazione sottostante (molle alla Winkler calibrate sul modello geotecnico del terreno).

Per la modellazione del terreno si considera la trave su suolo elastico, modellata con l'utilizzo di molle alla Winkler, aventi la seguente rigidezza (Vesic, 1965):

Per cui risulta:	$K = \frac{0.65E}{1-\nu^2} \cdot \frac{Eb^4}{(EJ)_{fond}}$	
E=	35000 kN/mq	elastico del terreno
$\nu$ =	0.3	coeff. di Poisson
<b>trave di fondazione</b>		
b=	1 m	dimensione trasversale trave
h=	1 m	altezza trave
J=	0.083333 m <sup>4</sup>	inerzia trave
Rck=	30 Mpa	
Ec=	31220186 kN/mq	modulo di elasticità cls
<b>K=</b>	<b>17459 kN/mc</b>	modulo di reazione lineare sulla trave



La soletta inferiore viene divisa in 10 elementi per poter schematizzare, tramite le molle applicate, l'interazione terreno struttura.

Per la rigidezza delle molle, nel caso in esame, si assume un valore del modulo di reazione verticale  $K_s=17459$  kN/m<sup>3</sup>.

Con questo valore si ricavano i valori delle singole molle, ottenendo per le 5 molle centrali un valore di:

$$K_{centrale} = K_s \times 0.84 = 14665.56 \text{ kN/m}$$

$$K_7 = K_8 = K_9 = K_{10} = K_{11} = 14665.56 \text{ kN/m}$$

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I valori delle molle di spigolo si ottengono con la seguente formulazione:

$$K_{31} = K_{23} = K_s \times (0.9/2) = 7856.55 \text{ kN/m}$$

I valori delle molle sotto i piedritto si ottengono con la seguente formulazione:

$$K_{17} = K_{19} = 2 \times K_s \times (0.84/2 + 0.9/2) = 30378.66 \text{ kN/m}$$

Ed infine i valori delle molle nei nodi 5, 6, 12 e 15 come da letteratura si assumono:

$$K_5 = K_6 = K_{12} = K_{15} = K_{32} = K_{20} = 1.5 \times K_{\text{centrale}} = 21998.34 \text{ kN/m}$$

## 7. ANALISI DEI CARICHI

### 7.1 Peso proprio (PP)

Il peso proprio è stato considerato ponendo il peso per unità di volume del calcestruzzo armato pari a  $\gamma = 25.0 \text{ kN/m}^3$ .

### 7.2 Sovraccarico permanente (PERM)

Sul solettone superiore si considera uno spessore medio del massetto delle pendenze  $s_b = 0.15 \text{ m}$  con peso per unità di volume  $\gamma_b = 20.00 \text{ kN/m}^3$ . Nel caso presente, non è presente un ricoprimento

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### 7.2.1 Pravimentazione stradale

Sul solettone superiore si considera uno spessore di pavimentazione stradale pari a 0.12m con peso di unità di volume  $\gamma_{pav\_str} = 20.00\text{kN/m}^3$ .

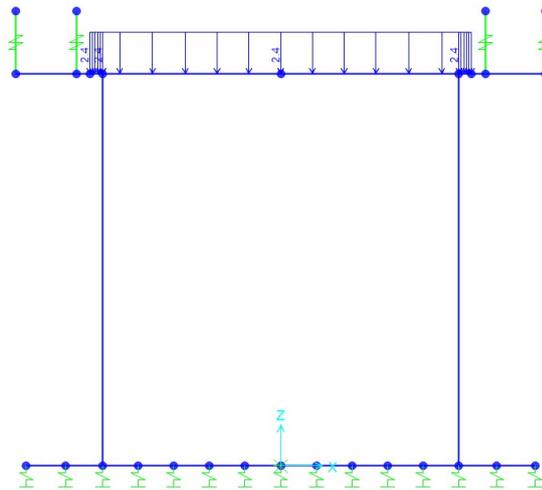


Fig. 5 – G2\_Pavimentazione

### 7.2.2 Peso sicurvia (G2)

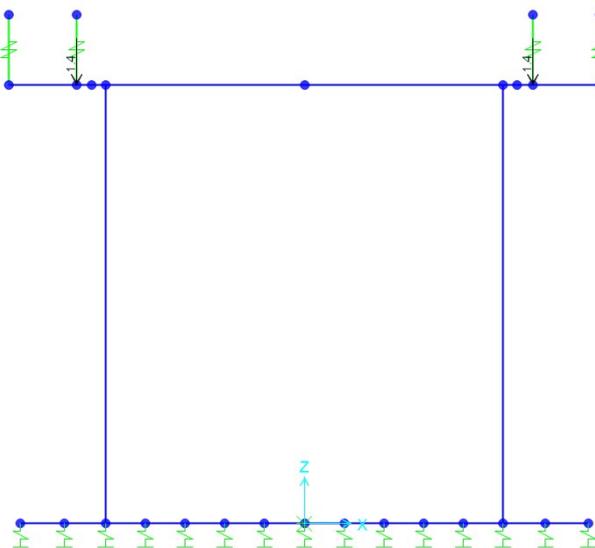


Fig. 6 – G2\_Sicurvia

### 7.2.3 Peso veletta (G2)

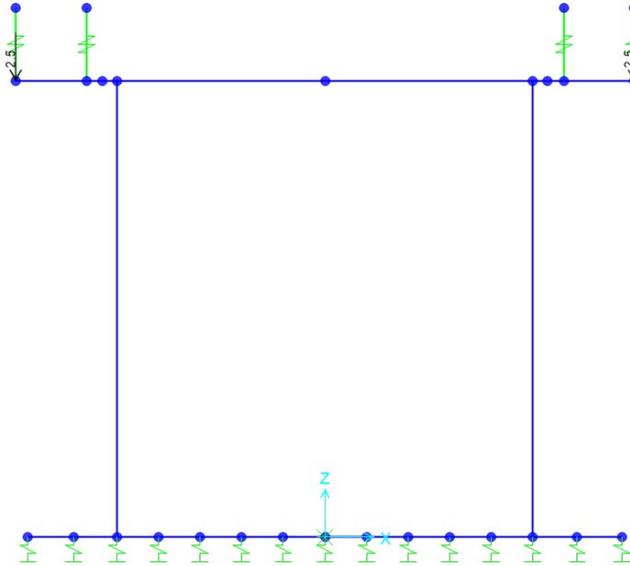


Fig. 7 – G2\_Veletta

### 7.2.4 Peso parapetto (G2)

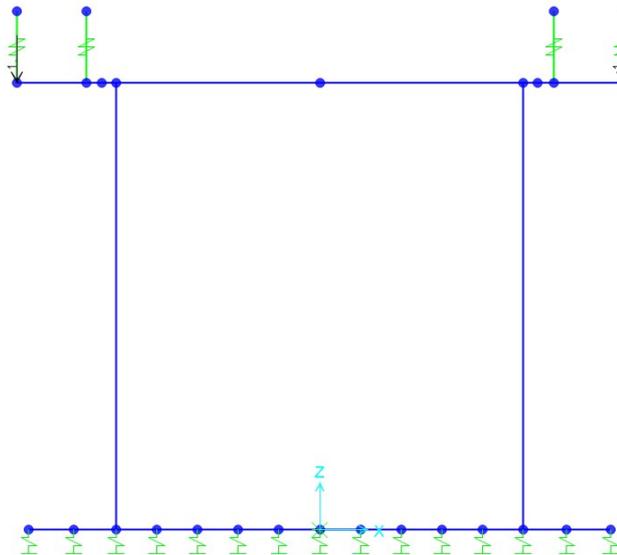


Fig. 8 – G2\_Parapetto

### 7.2.5 Peso impianti (G2)

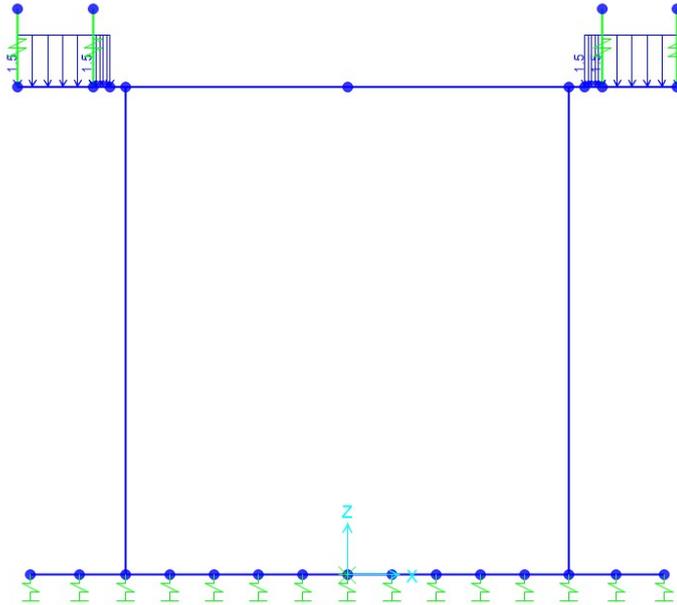


Fig. 9 – G2\_Impinati

### 7.3 Spinte del Terreno e dell'acqua (G3)

Sulle pareti laterali si considera la spinta del terreno valutata in condizioni di spinta in quiete assumendo per il terreno un peso di unità di volume  $\gamma_{\text{ter}} = 18.00 \text{ kN/m}^3$  e un angolo di attrito pari a  $28^\circ$ .

$$K_0 = 1 - \sin\phi = 0.5305$$

$$P = \gamma \cdot h \cdot K_0 = 18 \cdot 2 \cdot 0.5305 = 19.10 \text{ kN/m}$$

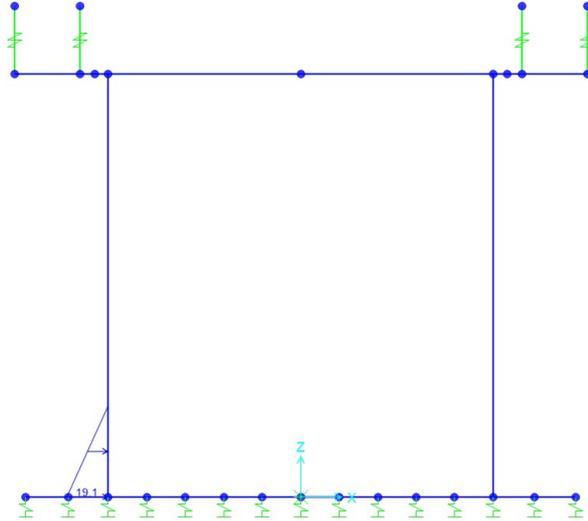


Fig. 10 – G3 Spinta terreno in SX

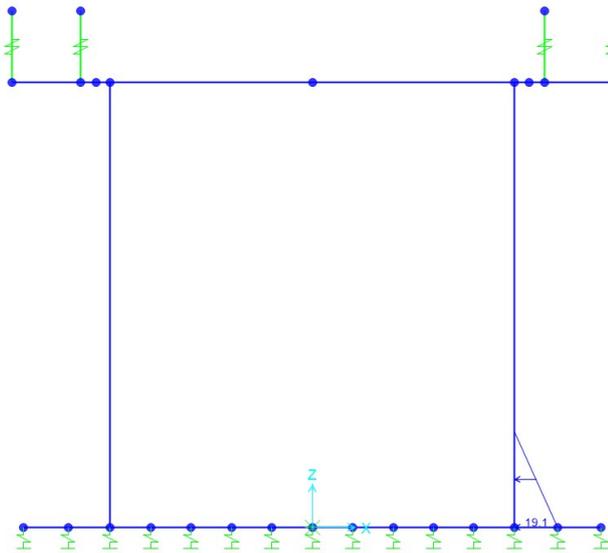


Fig. 11 – G3 Spinta terreno in DX

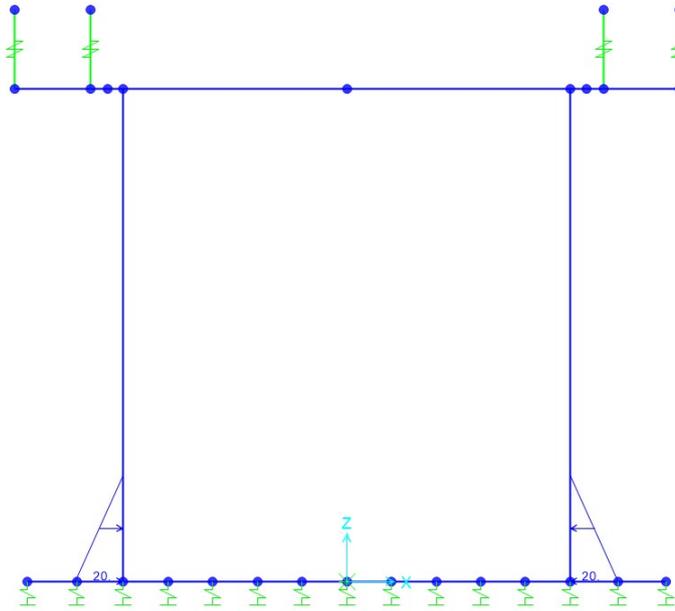


Fig. 12 – G3 Spinta dell'acqua orizzontale

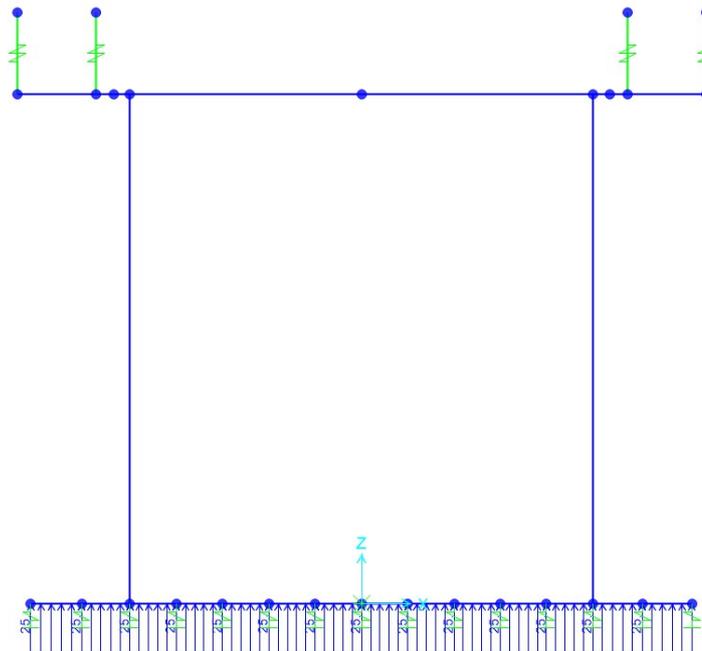


Fig. 13 – G3 Spinta dell'acqua Verticale



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#### 7.4 Azioni variabile da traffico

In conformità alla normativa di riferimento (N.T.C.2018 §5.1.3.3), si prendono in considerazione i carichi mobili per ponti di 1° categoria di seguito riportati:

- prima colonna di carico costituita da due carichi assiali  $Q_{1k} = 300\text{kN}$  e un carico uniformemente distribuito  $q_{1k} = 9\text{kN/m}^2$  su una larghezza convenzionale pari a 3.00m;
- seconda colonna di carico, analoga alla precedente, ma con carichi rispettivamente pari a  $Q_{2k} = 200\text{kN}$  e  $q_{2k} = 2.5\text{kN/m}^2$ ;
- terza colonna di carico, analoga alla precedente, ma con carichi rispettivamente pari a  $Q_{3k} = 100\text{kN}$  e  $q_{3k} = 2.5\text{kN/m}^2$ ;
- quarta colonna di carico e/o area rimanente costituita da un carico uniformemente distribuito pari a  $q_{rk} = 2.5\text{kN/m}^2$ .

I valori dei carichi stradali forniti dalle vigenti NTC2018 sono già comprensivi degli incrementi di natura dinamica. La dimensione delle impronte dei carichi tandem e la loro posizione relativa sono:

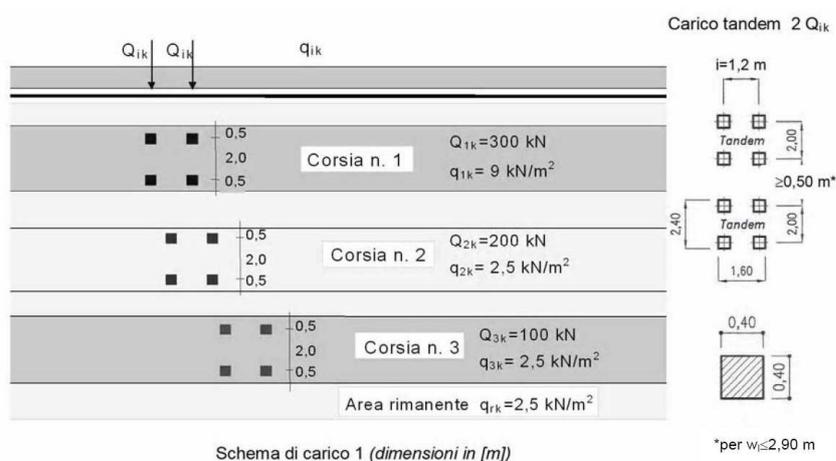


Fig. 14 – NTC18- schema di carico 1

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#### 7.4.1 Definizione delle corsie convenzionali

In relazione alle dimensioni della carreggiata sono state considerate n° 3 “corsie convenzionali” da 3m. I sovraccarichi mobili sono considerati nelle diverse disposizioni longitudinali e trasversali atte a generare le massime sollecitazioni nelle sezioni di verifica.

#### 7.4.2 Ripartizione dei carichi concentrati Tandem

I carichi concentrati mobili (carico tandem) sono ripartiti sull’impalcato mediante una diffusione a 45° in direzione longitudinale e trasversale a partire dall’estradosso della pavimentazione stradale fino alla linea d’asse della soletta.

Si riporta nella figura seguente lo schema delle impronte di carico e delle zone interessate dalla diffusione alla profondità  $d = 0.12 \text{ m} + 0.80/2 \text{ m} = 0.52 \text{ m}$  dal piano stradale.

Le larghezze di diffusione della coppia di carichi concentrati su due assi in tandem valgono:

- dir. Longitudinale       $L_{\text{long}} = 1.2\text{m} + 2x (0.40\text{m}/2 + 0.12 \text{ m} + 0.80\text{m}/2 ) = 2.64 \text{ m}$
- dir. Trasversale       $L_{\text{trasv}} = 2 \times 0.12 + 0.4 + 0.8 = 1.44\text{m}$

Il carico tandem  $2 Q_{ik}$  determina pertanto un carico uniformemente distribuito sull’asse della soletta di intensità pari a

- $q = 150 \text{ kN} / (1.44 \text{ m} \times 1.44 \text{ m}) = 72.33 \text{ kPa}$       corsia 1
- $q = 100 \text{ kN} / (1.44 \text{ m} \times 1.44 \text{ m}) = 48.23 \text{ kPa}$       corsia 2
- $q = 50 \text{ kN} / (1.44 \text{ m} \times 1.44 \text{ m}) = 24.11 \text{ kPa}$       corsia 3

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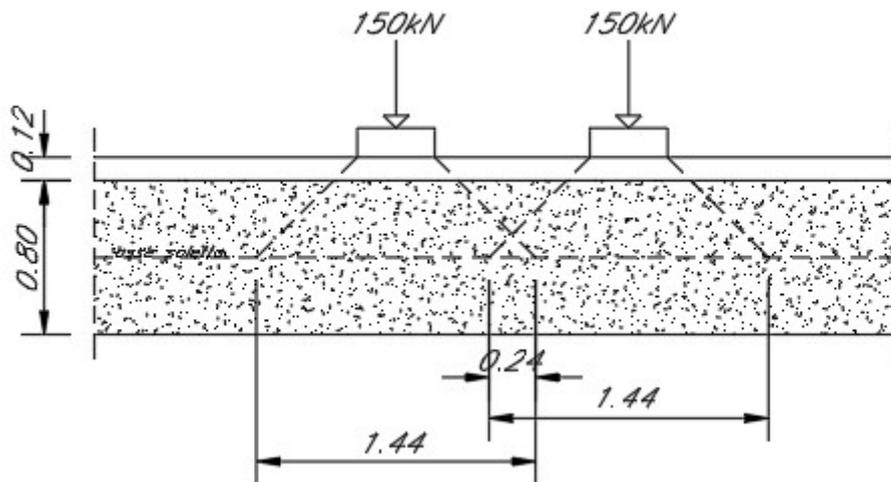


Fig. 15 – Diffusione del carico stradale tandem in direzione longitudinale

Nella striscia in direzione longitudinale di 1 metro il tratto di sovrapposizione risulta pari a:

$$L_{sovr} = 0.24 \text{ cm}$$

Tenendo quindi in conto la sovrapposizione longitudinale delle impronte di carico diffuse, il carico ripartito trasversalmente sull'elemento soletta risulta pari a :  $Q = q \times 0.12 + 2q \times 0.24$

da cui:

$$Q = 72.33 \times 0.76 + 2 \times 72.33 \times 0.24 = 89.68 \text{ KN/m} \quad \text{corsia 1}$$

$$Q = 48.23 \times 0.76 + 2 \times 48.23 \times 0.24 = 59.81 \text{ KN/m} \quad \text{corsia 2}$$

$$Q = 24.11 \times 0.76 + 2 \times 24.11 \times 0.24 = 29.90 \text{ KN/m} \quad \text{corsia 3}$$

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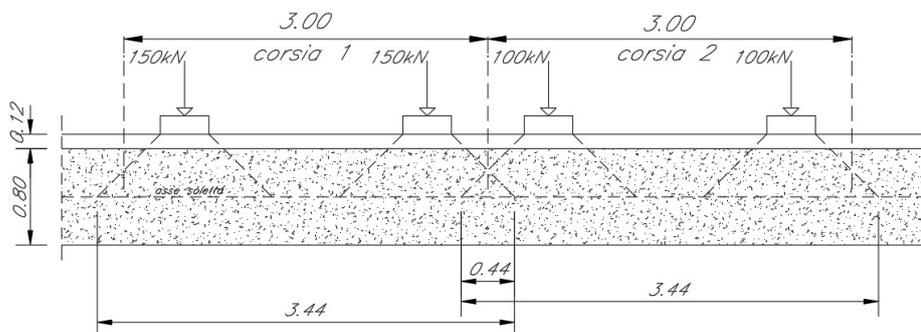


Fig. 16 – Diffusione del carico stradale tandem direzione trasversale

### 7.4.3 Configurazioni di carico

Per massimizzare gli effetti del carico mobile in termini di sollecitazioni flettenti e taglio sugli elementi strutturali, sono state definite due distinte configurazioni di carico.

La prima, massimizza il momento flettente sul traverso superiore della struttura; si ottiene disponendo i carichi tandem di tutte le corsie convenzionali disposte affiancate e centrate rispetto l'asse di simmetria verticale della struttura. Ai carichi tandem si sovrappone il carico distribuito di intensità pari a  $9.00 \text{ kN/m}^2$  sulla corsia 1 e  $2.50 \text{ kN/m}^2$  sulla corsia 2 e 3.

La seconda configurazione massimizza la sollecitazione di taglio sul traverso superiore della struttura; le corsie convenzionali sono disposte affiancate ed allineate con il filo interno del piedritto destro. Ai carichi tandem si sovrappone il carico distribuito di intensità pari a  $9.00 \text{ kN/m}^2$  sulla corsia 1 e pari a  $2.5 \text{ kN/m}^2$  sulla corsia 2.

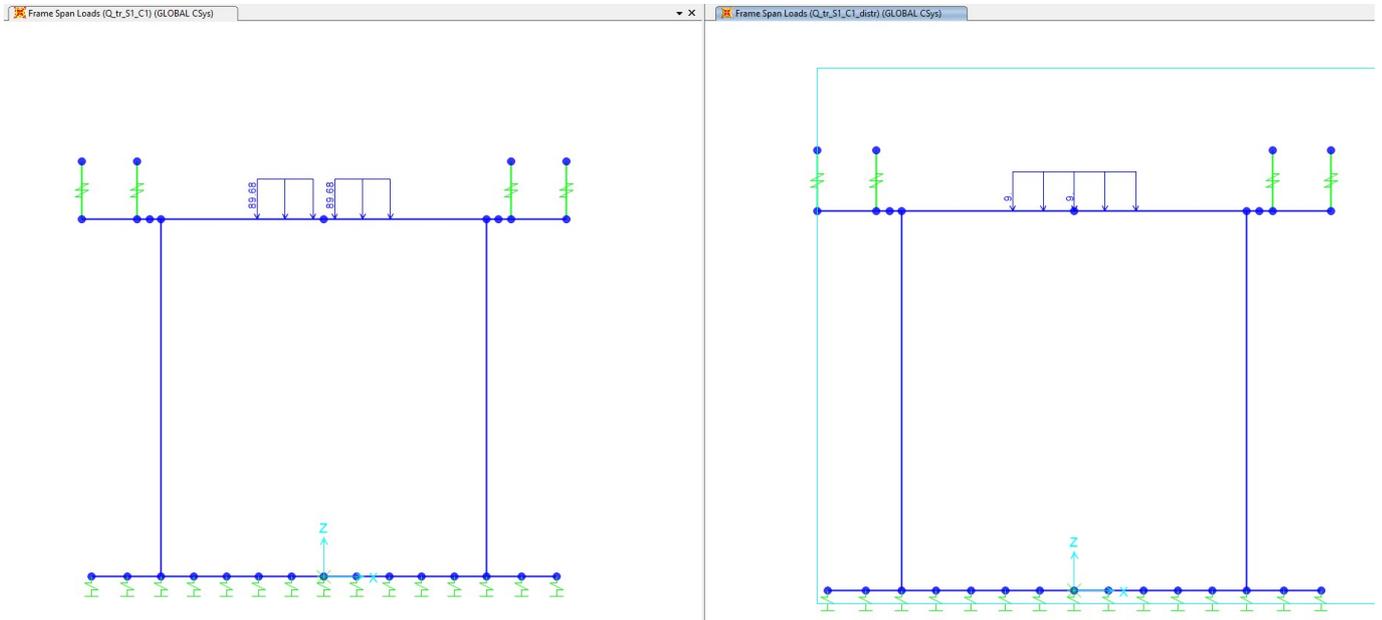


Fig. 17 – Q\_tr\_S1\_C1\_Ruota: Carico tandem e carico distribuito Corsia 1

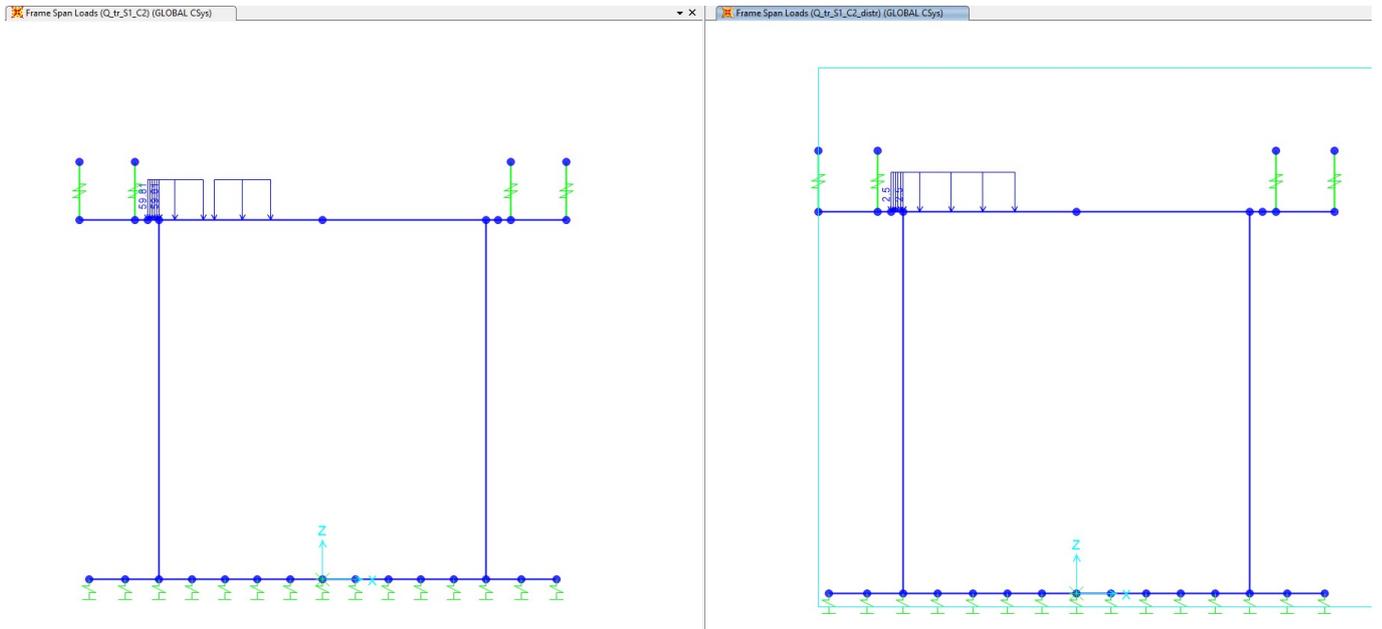


Fig. 18 – Q\_tr\_S1\_C2\_Ruota: Carico tandem e carico distribuito Corsia 2

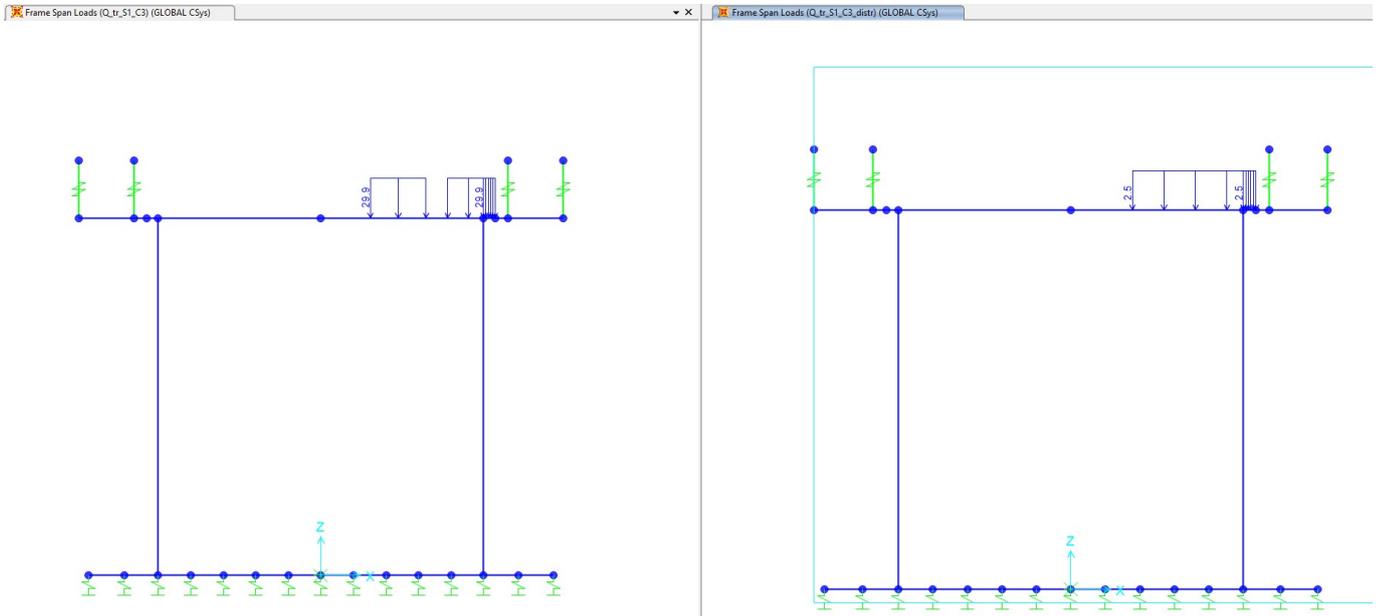


Fig. 19 – Q\_tr\_S1\_C3\_Ruota: Carico tandem e carico distribuito Corsia 3

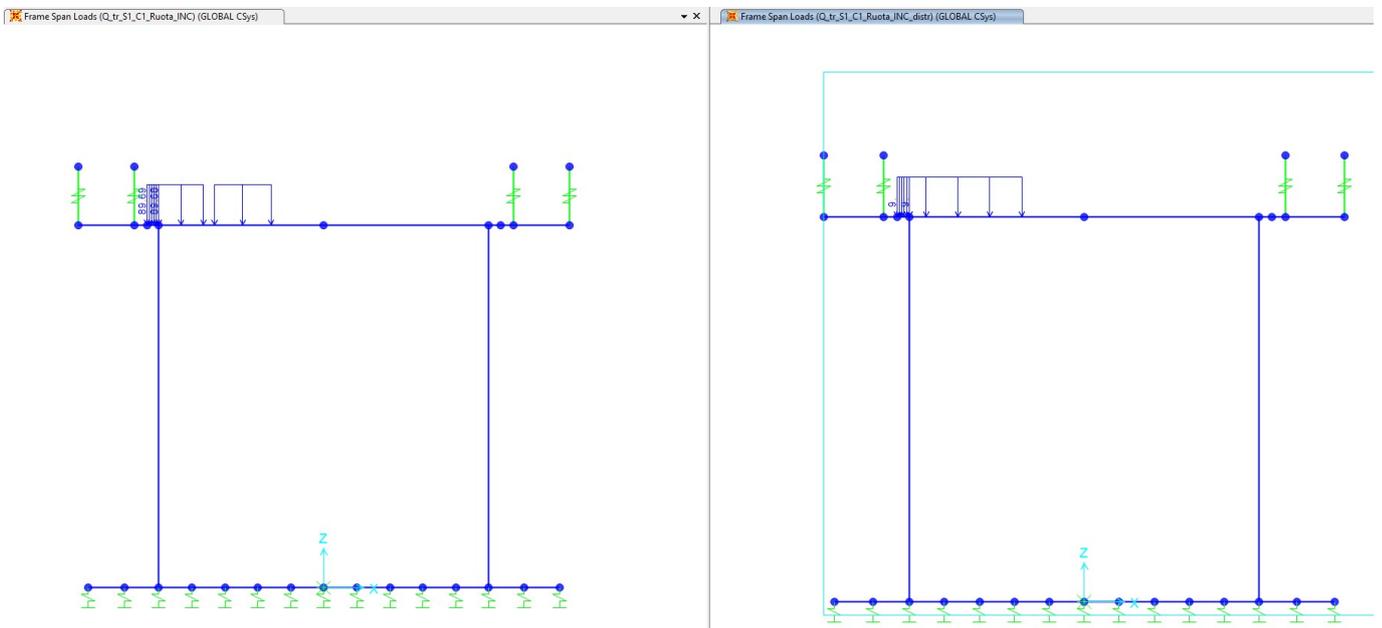


Fig. 20 – Q\_tr\_S1\_C1\_Ruota\_INC: Carico tandem e carico distribuito Corsia 1 all'incastro piedritto

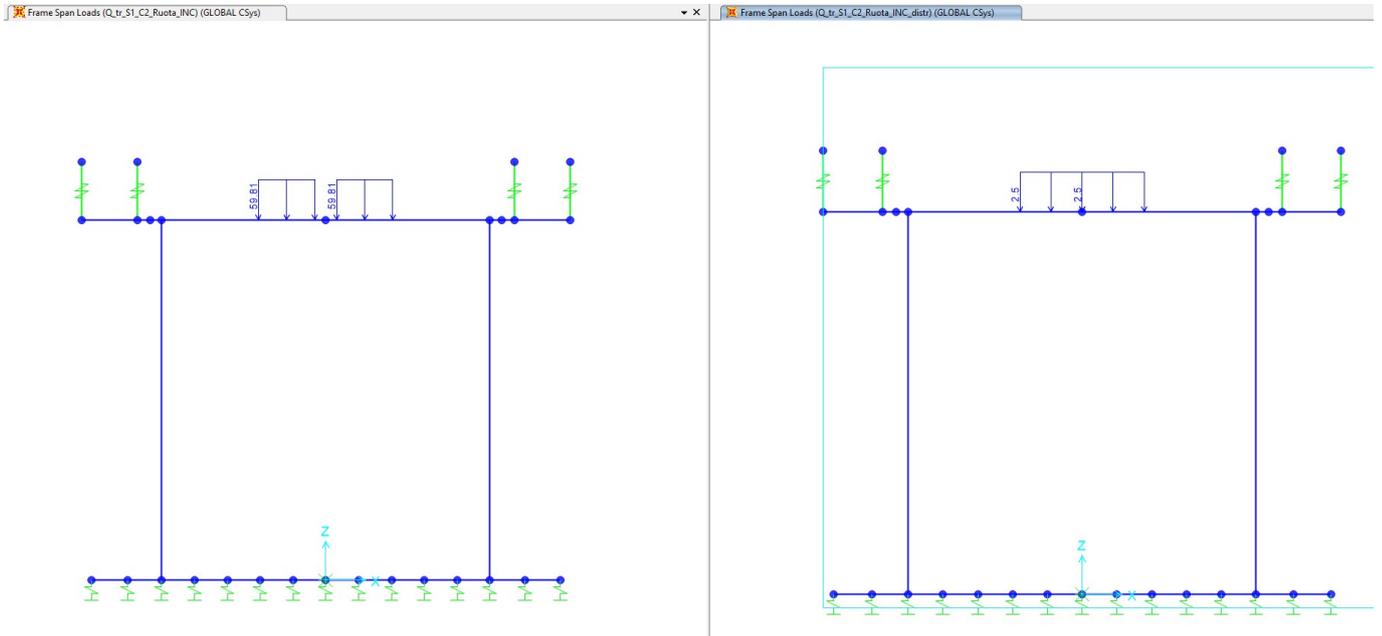


Fig. 21 –  $Q_{tr} S1_C2_{Ruota\_INC}$ : Carico tandem e carico distribuito Corsia 2 all'incastro piedritto

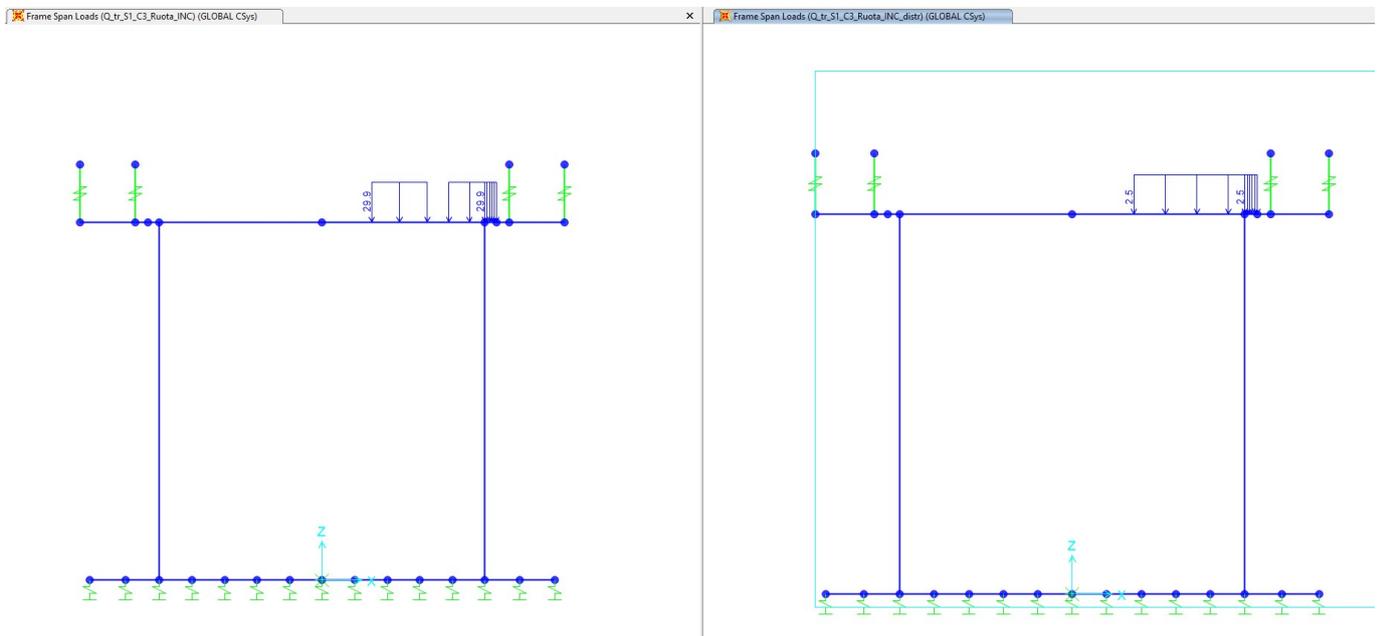


Fig. 22 –  $Q_{tr} S1_C3_{Ruota\_INC}$ : Carico tandem e carico distribuito Corsia 3 all'incastro piedritto

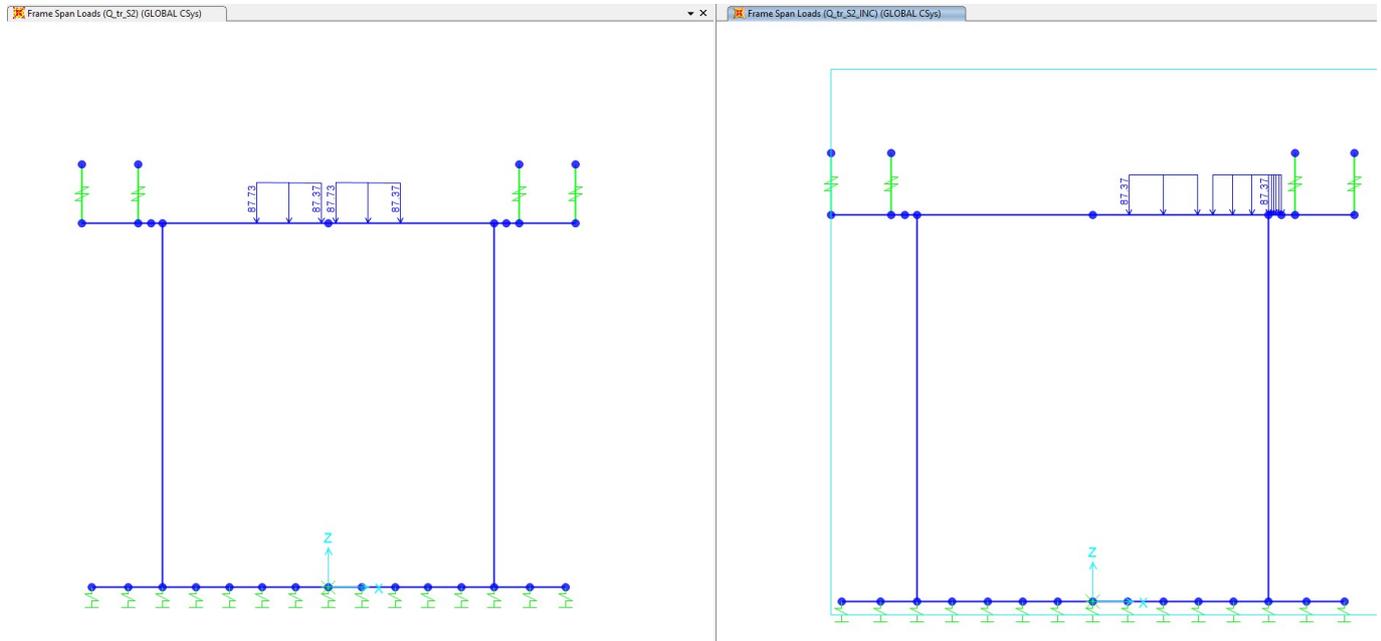


Fig. 23 –  $Q_{tr\_S2\_Ruota/Ruota\_INC}$ : Carico tandem Scheme 2

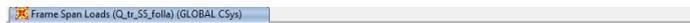


Fig. 24 –  $Q_{tr\_S5\_folla}$



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## 7.5 Azioni centrifuga

Nei ponti con asse curvo di raggio R (in metri) l'azione centrifuga corrispondente ad ogni colonna di carico si valuta convenzionalmente come indicato in Tab. 5.1.III (NTC 2018), essendo  $Q_v = \sum_i 2Q_{ik}$  il carico totale dovuto agli assi tandem dello schema di carico 1 agenti sul ponte.

Il carico concentrato  $q_4$ , applicato a livello della pavimentazione, agisce in direzione normale all'asse del ponte.

According to NTC 2018 , Tab. 5.1.III - Valori caratteristici delle forze centrifughe,

$q_4=0.2 Q_v$ , when  $R<200$  m (applicable to this project) and considering concurrent loading from all lanes.

Assuming that horizontal point loads are dispersed through angle of  $2 \times 45^\circ$ , and considering the worst case of distributing bigger forces over smaller triangular area, we calculate distributed (dispersed over triangular area) force for each lane to be  $Q_{fr,i}/A_{fr,i}$ , refer to Fig.25.

where  $i=1; 2; 3$ ;  $Q_{fr,i} = 0.2 Q_v$ ;  $A_{fr,i} = B_i^2/4$ ;

$Q_{fr,1} = 0.2 Q_{v1} = 120$  KN;  $Q_{fr,2} = 0.2 Q_{v2} = 80$  KN;  $Q_{fr,3} = 0.2 Q_{v4} = 40$  KN

$A_{fr,1} = 3^2/4$ ;  $A_{fr,2} = 9^2/4$ ;  $A_{fr,3} = 15^2/4$

Si applica una forza uniformemente ripartita sulla soletta di copertura pari a

$q_4 = \sum Q_{fr,i}/A_{fr,i} = 9.38$  kN/m' (per 1 m in direzione longitudinale della rampa) e un momento di trasporto in asse soletta di  $m_y = 9.38 * 0.52 = 4.88$  kNm/m'.

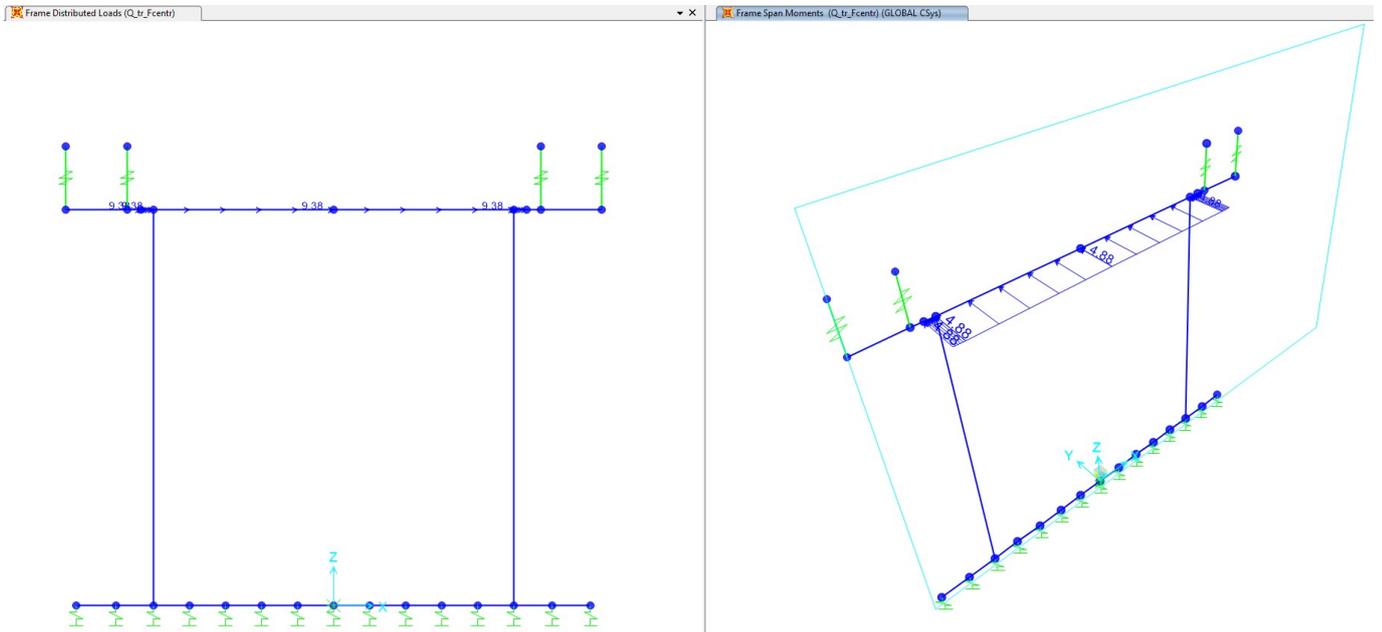
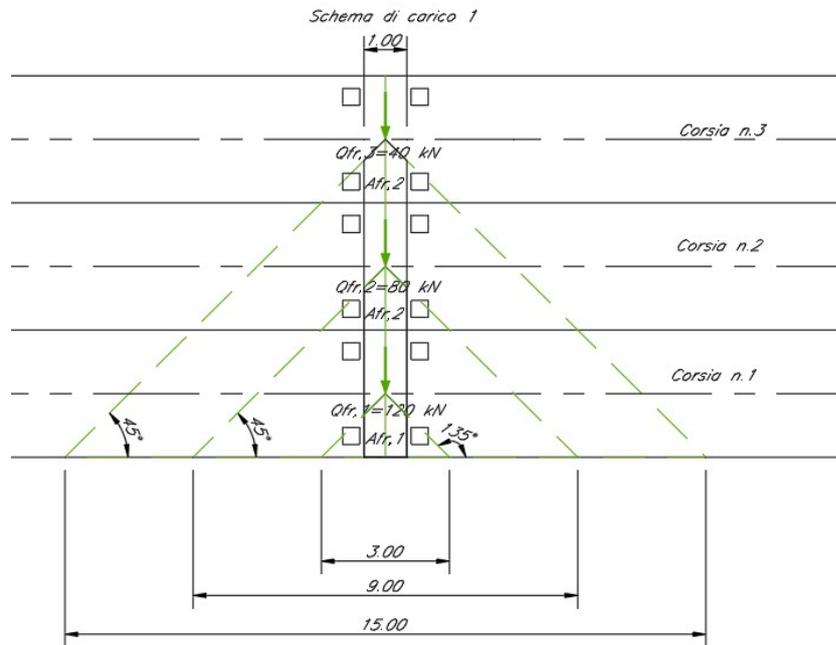


Fig. 25 –  $Q_{tr} F_{centr}$ : Forza e momento distribuiti di centrifuga

## 7.6 Azioni climatiche

### 7.6.1 Azioni termiche uniformi (TU)

Si considera una variazione termica uniforme  $\Delta T = 15.0^{\circ}\text{C}$  sugli elementi della struttura in elevazione, adottando per il coefficiente di dilatazione termica un valore  $\alpha = 10 \times 10^{-6}$ .

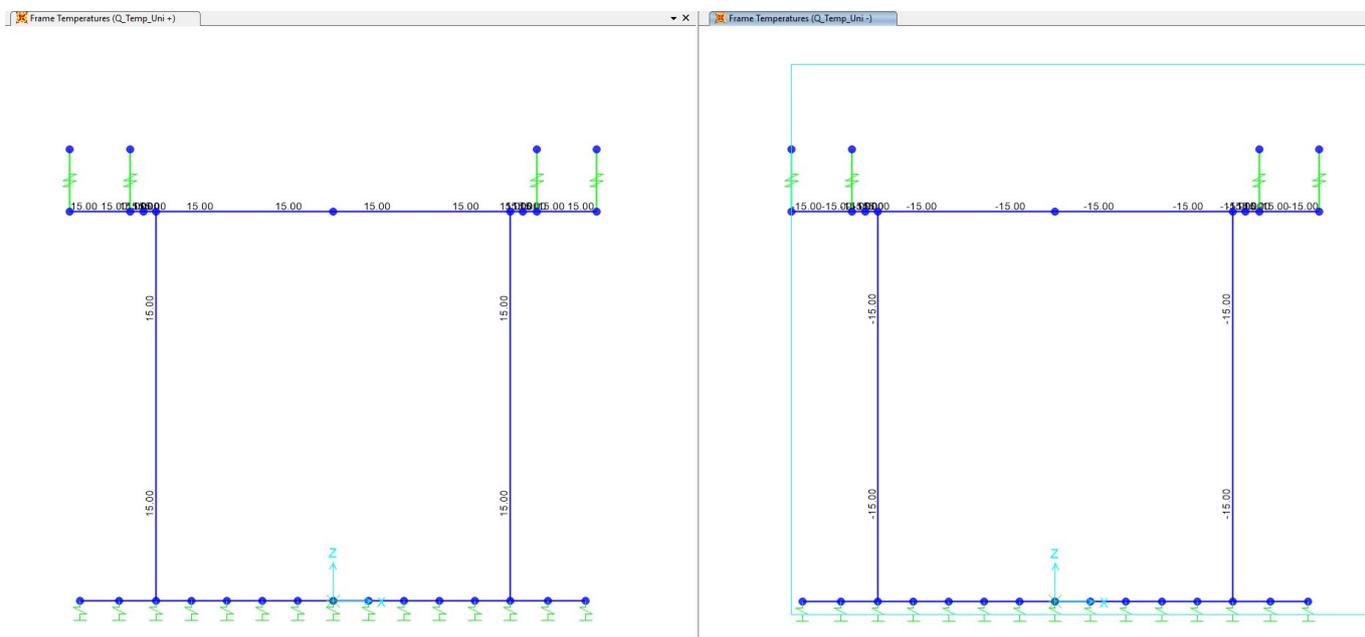


Fig. 26 – Q\_Temp\_Uni+/-: Forza termiche uniformi

### 7.6.2 Azioni termiche differenziali (TF)

Si considera una variazione termica differenziale  $\Delta T = 5.0^{\circ}\text{C}$  su tutti gli elementi della struttura in elevazione, adottando per il coefficiente di dilatazione termica un valore  $\alpha = 10 \times 10^{-6}$ .

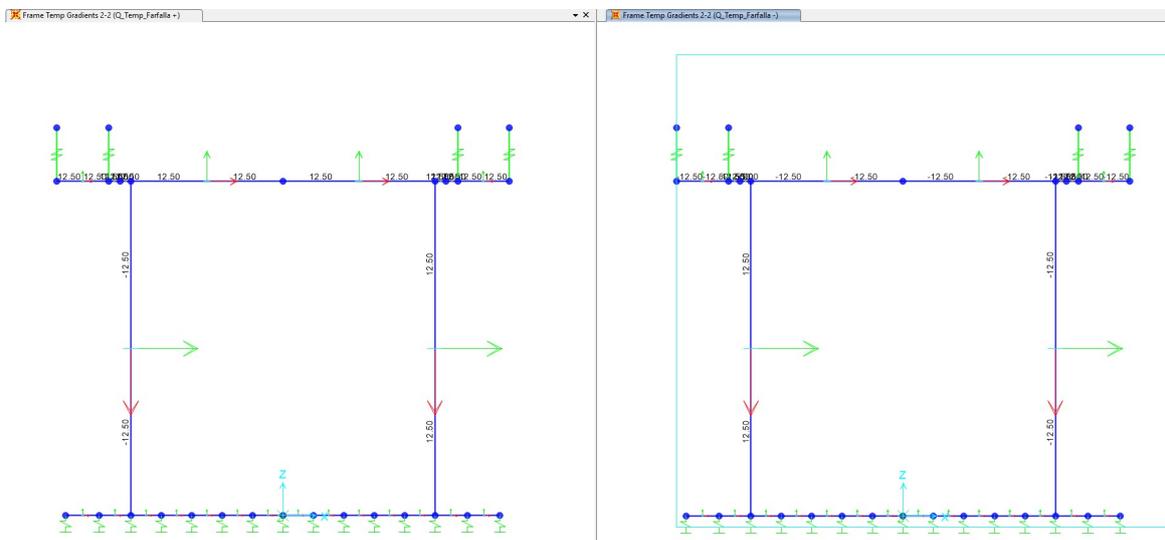


Fig. 27 – Q\_Temp\_Farfalla+/-: Forza termiche differenziali

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

### 7.6.3 Ritiro

Si considera una variazione termica uniforme equivalente  $\Delta T = -9.50^{\circ}\text{C}$  sulla soletta superiore. Il calcolo viene condotto secondo le indicazioni nell'EUROCODICE 2-UNI EN1992-1-1 Novembre 2005 e D.M.11-01-2018.

DATA		
Coefficiente di viscosità		
$\varphi(t, t_0) = \varphi_0 * \beta_c(t, t_0)$	2.4198	creep coefficient
$\varphi_0 = \varphi_{RH} * \beta(f_{cm}) * \beta(t_0)$	2.4589	nominal creep coefficient
$\varphi_{RH}$	1.1559	per $f_{cm} > 35$ Mpa
$\varphi_{RH}$		per $f_{cm} \leq 35$ Mpa
RH	75 [%]	
$\beta(f_{cm})$	2.6563	
$\beta(t_0)$	0.8008	
$h_0$	1800 [mm]	
Ac	9000 [cm <sup>2</sup> ]	
u	100 [cm]	
$\beta_c(t, t_0)$	0.9841	
t	25550 [days]	
$t_0$	2 [days]	
$t - t_0$	25548 [days]	
$\beta_{RH}$	3339.11	FALSE per $f_{cm} > 35$ Mpa
$\beta_{RH}$		per $f_{cm} \leq 35$ Mpa
$\alpha_1$	0.9108	
$\alpha_2$	0.9736	
$\alpha_3$	0.9354	
$f_{cm}$	40.00 [MPa]	
$f_{ck}$	32.00 [MPa]	
DEFORMAZIONE TOTALE DA RITIRO		
$\epsilon_{cs} = \epsilon_{ct} + \epsilon_{ca}$	0.00032491	deformazione totale da ritiro
$\epsilon_{ct}$	0.00026991	deformazione da ritiro per essiccamento
$\epsilon_{ca}$	5.5E-05	deformazione da ritiro autogeno
DEFORMAZIONE DA RITIRO PER ESSICCAMENTO		
$\epsilon_{ct} = k_{RH} * \epsilon_{ct0}$	0.00030218	
$\epsilon_{ct0}$	0.00043168	appendix B for $\epsilon_{ct0}$
$\alpha_{d,1}$	6	CLS class R
$\alpha_{d,2}$	0.11	CLS class R
$\beta_{RH}$	0.8961	
$RH_0$	100 [%]	
$f_{ct0}$	10 [MPa]	
$\beta_{ct}(t, ts)$	0.89320235	
t	25550	
ts	2	
DEFORMAZIONE DA RITIRO AUTOGENO		
$\beta_{ca}(t) =$	1	
t	25550	
$\epsilon_{ca}(\infty) =$	0.000055	

$$\Delta T_{rit} = \epsilon_{cs} / [(1 + \varphi) * \alpha]$$

$$\Delta T_{rit} = -9.5^{\circ}\text{C}$$

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	38 di 155

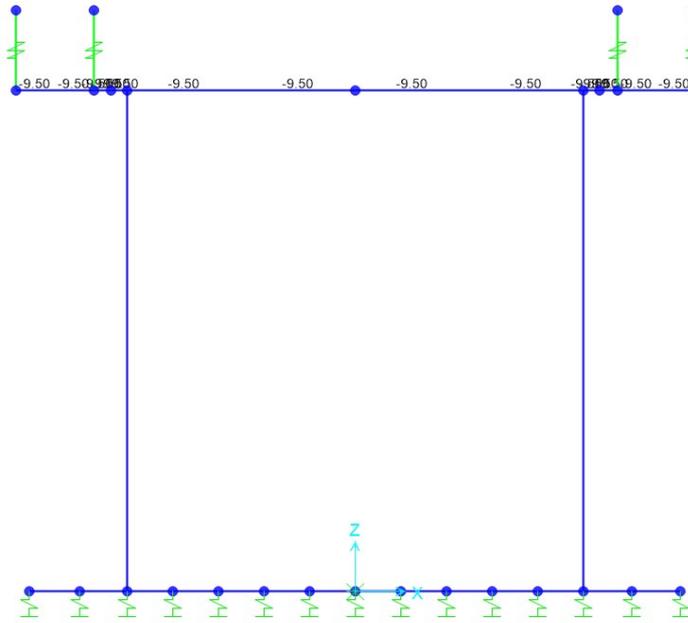


Fig. 28 – Q\_Ritiro

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

## 7.7 Carico del Vento

La pressione del vento è data dall'espressione

$$q_w = q_b * c_e * c_p * c_d$$

dove

$q_b$  è la pressione cinetica di riferimento

$c_e$  è il coefficiente di esposizione

$c_p$  è il coefficiente di forma

$c_d$  è il coefficiente dinamico

La velocità di riferimento  $v_b$  è data dall'espressione:

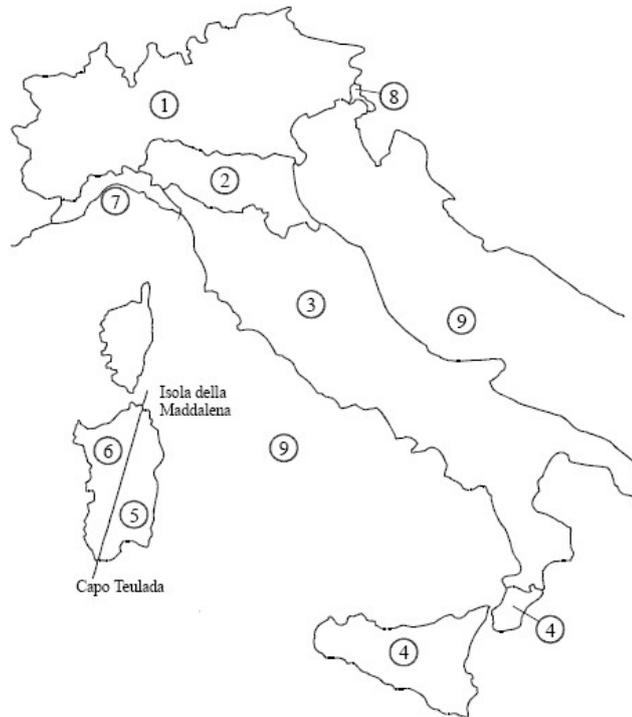
$$v_b = v_{b,0} \quad a_s \leq a_0$$

$$v_b = v_{b,0} + k_a (a_s - a_0) \quad a_0 < a_s \leq 1500m$$

dove  $v_{b,0}$ ,  $a_0$ ,  $k_a$  sono i parametri legati alla zona in cui sorge la costruzione

**Tabella 3.3.I - Valori dei parametri  $v_{b,0}$ ,  $a_0$ ,  $k_a$**

Zona	Descrizione	$v_{b,0}$ [m/s]	$a_0$ [m]	$k_a$ [1/s]
1	Valle d'Aosta, Piemonte, Lombardia, Trentino Alto Adige, Veneto, Friuli Venezia Giulia (con l'eccezione della provincia di Trieste)	25	1000	0,010
2	Emilia Romagna	25	750	0,015
3	Toscana, Marche, Umbria, Lazio, Abruzzo, Molise, Puglia, Campania, Basilicata, Calabria (esclusa la provincia di Reggio Calabria)	27	500	0,020
4	Sicilia e provincia di Reggio Calabria	28	500	0,020
5	Sardegna (zona a oriente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	750	0,015
6	Sardegna (zona a occidente della retta congiungente Capo Teulada con l'Isola di Maddalena)	28	500	0,020
7	Liguria	28	1000	0,015
8	Provincia di Trieste	30	1500	0,010
9	Isole (con l'eccezione di Sicilia e Sardegna) e mare aperto	31	500	0,020



**Figura 3.3.1** – *Mappa delle zone in cui è suddiviso il territorio italiano*

Il coefficiente di esposizione è dato dalla seguente espressione:

$$c_e = k^2 r c_t \ln\left(\frac{z}{z_0}\right) \left(7 + c_t \ln\left(\frac{z}{z_0}\right)\right) \quad \text{per } z > z_{\min}$$

$$c_e(z) = c_e(z_{\min}) \quad \text{per } z < z_{\min}$$

Si assume una superficie dei carichi transitanti sull'opera esposta al vento di altezza pari a 3.0m a partire dal piano stradale.



**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	41 di 155

**CALCOLO DELL'AZIONE DEL VENTO**

3) Toscana, Marche, Umbria, Lazio, Abruzzo, Molise, Puglia, Campania, Basilicata, Calabria (esclusa la provincia di Reggio Calabria)

Zona	$v_{b,0}$ [m/s]	$a_0$ [m]	$k_s$ [1/s]
3	27	500	0.02
$a_0$ (altitudine sul livello del mare [m])		155.4	
$T_0$ (Tempo di ritorno)		75	
$v_1 = v_{b,0}$ per $a_0 < a_0$			
$v_2 = v_{b,0} * k_s (a_0 - a_0)$ per $a_0 < a_0 \leq 1500$ m			
$v_0$ ( $T_0 = 50$ [m/s])		27.000	
$q_0$ (Te)		1.02346	
$v_0$ ( $T_0$ ) = $v_2 * q_0$ [m/s]		27.633	



$p$  (pressione del vento [N/m<sup>2</sup>]) =  $q_0 * c_e * c_s$   
 $q_0$  (pressione cinetica di riferimento [N/m<sup>2</sup>])  
 $c_e$  (coefficiente di esposizione)  
 $c_s$  (coefficiente di forma)  
 $c_d$  (coefficiente dinamico)

**Pressione cinetica di riferimento**

$q_0 = 1/2 * \rho * v_0^2$  ( $\rho = 1.25$  kg/m<sup>3</sup>)

$q_0$ [N/m <sup>2</sup> ]	477.25
---------------------------	--------

**Coefficiente di forma**

È il coefficiente di forma (o coefficiente aerodinamico), funzione della tipologia e della geometria della costruzione e del suo orientamento rispetto alla direzione del vento. Il suo valore può essere ricavato dai cataloghi di opportuna documentazione o da prove sperimentali in galleria del vento.

**Coefficiente dinamico**

È il coefficiente dinamico, funzione della tipologia e della geometria della costruzione e del suo orientamento rispetto alla direzione del vento. Il suo valore può essere ricavato dai cataloghi di opportuna documentazione o da prove sperimentali in galleria del vento.

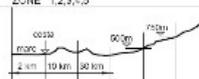
**Coefficiente di esposizione**

Classe di rugosità del terreno

B) Aree urbane (non di classe A), suburbane, industriali e boschive

**Categoria di esposizione**

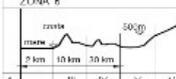
**ZONE 1,2,3,4,5**



A	IV	IV	V	V	V
B	III	III	IV	IV	IV
C	II	II	III	III	III
D	I	I	II	II	II

• Categoria II in zona 1,2,3,4  
 • Categoria III in zona 5  
 • Categoria IV in zona 1

**ZONE 6**



A	III	IV	IV	IV
B	II	III	III	III
C	I	II	II	II
D	I	I	II	II

**ZONE 7,8**



A	IV	IV	IV	IV
B	III	III	III	III
C	II	II	II	II
D	I	I	II	II

**ZONE 9**



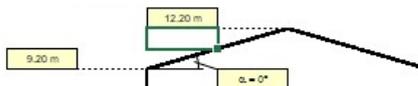
A	IV	IV
B	III	III
C	II	II
D	I	I

$v_{0,z} = k_z * v_0$  ( $k_z$  [1]) per  $z \geq z_{ref}$   
 $k_z = c_d / z_{ref}$  per  $z < z_{ref}$

$z$ [m]	$c_d$
$z \leq 5$	1.768
$z = 9.2$	2.084
$z = 12.2$	2.268

Zona	Classe di rugosità	$a_0$ [m]
3	B	155.4

Cat. Esposiz.	$k_z$	$z_0$ [m]	$z_{ref}$ [m]	$c_s$
III	0.2	0.1	5	1



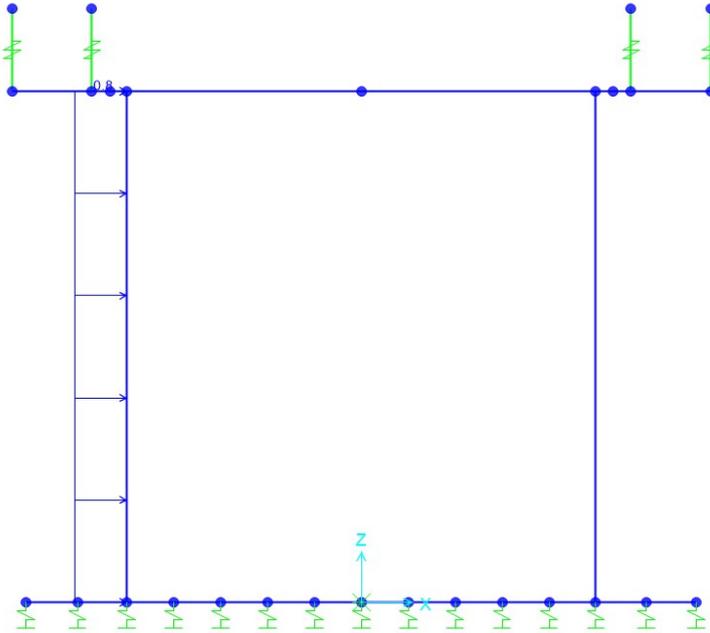


Fig. 29 –  $Q_{\text{Vento Parete sopravento}}$

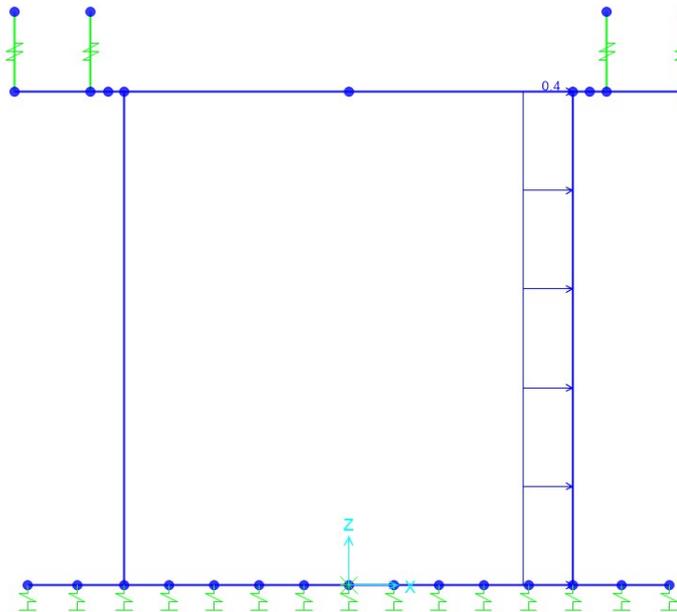


Fig. 30 –  $Q_{\text{Vento Parete sottovento}}$

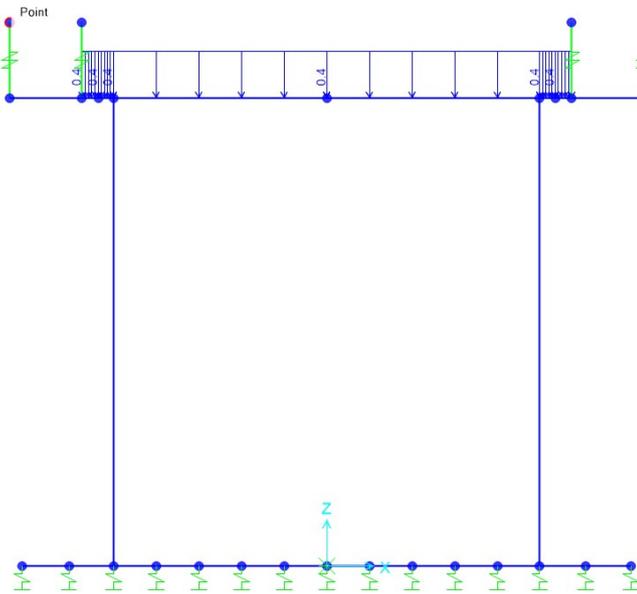


Fig. 31 – Q\_Vento Soletta di copertura

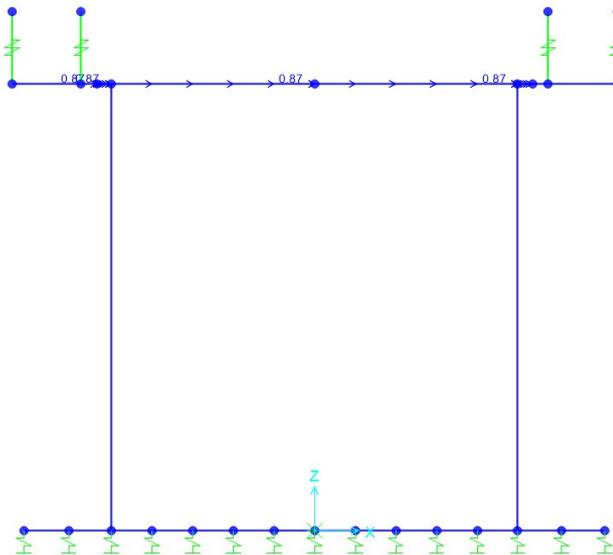


Fig. 32 – Q\_Vento Veicoli

	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

## 7.8 Azioni sismiche

In condizioni sismiche, il rispetto degli stati limite si considera conseguito quando:

- nei confronti degli stati limite di esercizio siano rispettate le verifiche relative allo Stato Limite di Danno;
- nei confronti degli stati limite ultimi siano rispettate le verifiche relative allo Stato Limite di salvaguardia della Vita.

Gli stati limite, sia di esercizio sia ultimi, sono individuati riferendosi alle prestazioni che l'opera a realizzarsi deve assolvere durante un evento sismico; nel caso di specie per la funzione che l'opera deve espletare nella sua vita utile, è significativo calcolare lo Stato Limite di Danno (SLD) per l'esercizio e lo Stato Limite di Salvaguardia della Vita (SLV) per lo stato limite ultimo.

Per la definizione dell'azione sismica si assumono i seguenti parametri di base:

- Categoria di suolo: E
- Categoria topografica: T1
- Vita nominale: VN = 100 anni; (tab 2.4.1);
- Classe d'uso : IV;
- Coeff. d'uso:  $c_u = 2.0$
- Periodo di riferimento per l'azione sismica:  $VR = VN \times c_u = 200$  anni

I parametri che definiscono l'azione sismica, calcolati mediante il documento excel Spettri-NTC.ver.1.0.3.xls fornito dal Consiglio Superiore dei Lavori Pubblici, vengono di seguito riportati:

*Individuazione della pericolosità sismica del sito*

### FASE 1. INDIVIDUAZIONE DELLA PERICOLOSITÀ DEL SITO

Ricerca per coordinate

LONGITUDINE:     LATTITUDINE:

---

Ricerca per comune

REGIONE:     PROVINCIA:     COMUNE:

**Elaborazioni grafiche**

Grafici spettri di risposta

Variabilità dei parametri

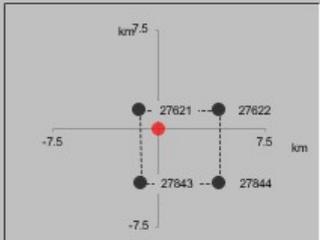
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**Elaborazioni numeriche**

Tabella parametri

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**Nodi del reticolo intorno al sito**



**Reticolo di riferimento**

Controllo sul reticolo

- Sito esterno al reticolo
- Interpolazione su 3 nodi
- Interpolazione corretta

Interpolazione:



La "Ricerca per comune" utilizza le coordinate ISTAT del comune per identificare il sito. Si sottolinea che all'interno del territorio comunale le azioni sismiche possono essere significativamente diverse da quelle così individuate e si consiglia, quindi, la "Ricerca per coordinate".

*Scelta della strategia di progettazione*

### FASE 2. SCELTA DELLA STRATEGIA DI PROGETTAZIONE

Vita nominale della costruzione (in anni) -  $V_N$   info

Coefficiente d'uso della costruzione -  $c_U$   info

---

**Valori di progetto**

Periodo di riferimento per la costruzione (in anni) -  $V_R$   info

Periodi di ritorno per la definizione dell'azione sismica (in anni) -  $T_R$  info

Stati limite di esercizio - SLE	SLO - $P_{VR} = 81\%$	<input type="text" value="120"/>
	SLD - $P_{VR} = 63\%$	<input type="text" value="201"/>
Stati limite ultimi - SLU	SLV - $P_{VR} = 10\%$	<input type="text" value="1898"/>
	SLC - $P_{VR} = 5\%$	<input type="text" value="2475"/>

**Elaborazioni**

Grafici parametri azione

Grafici spettri di risposta

Tabella parametri azione

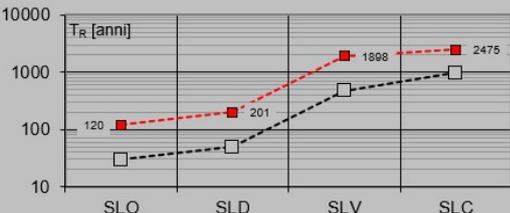
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**LEGENDA GRAFICO**

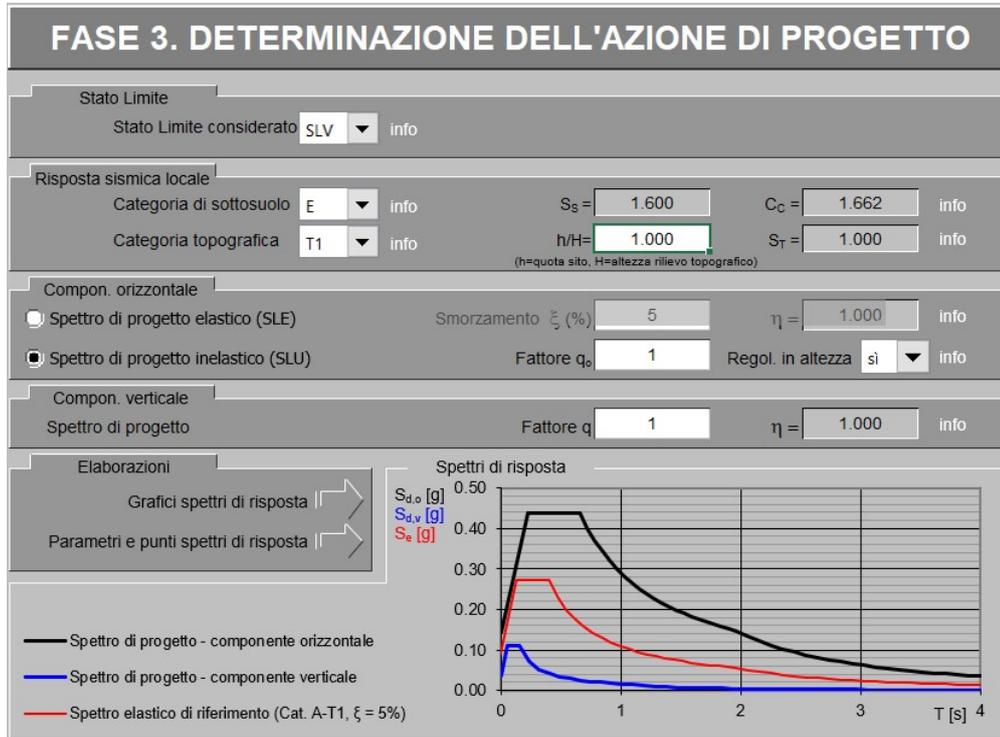
---□--- Strategia per costruzioni ordinarie

---■--- Strategia scelta

**Strategia di progettazione**



*Determinazione dell'azione di progetto SLV*



**Parametri e punti dello spettro di risposta orizzontale per lo stato SLV**

**Parametri indipendenti**

STATO LIMITE	SLV
$a_g$	0.090 g
$F_0$	3.034
$T_C$	0.398 s
$S_a$	1.600
$C_C$	1.662
$S_T$	1.000
$q$	1.000

**Parametri dipendenti**

$S$	1.600
$\eta$	1.000
$T_B$	0.221 s
$T_C$	0.662 s
$T_D$	1.960 s

**Espressioni dei parametri dipendenti**

$$S = S_a \cdot S_T \quad (\text{NTC-08 Eq. 3.2.5})$$

$$\eta = \sqrt{10 / (5 + \xi)} \geq 0,55; \quad \eta = 1/q \quad (\text{NTC-08 Eq. 3.2.6; §. 3.2.3.5})$$

$$T_B = T_C / 3 \quad (\text{NTC-07 Eq. 3.2.8})$$

$$T_C = C_C \cdot T_C^* \quad (\text{NTC-07 Eq. 3.2.7})$$

$$T_D = 4,0 \cdot a_g / g + 1,6 \quad (\text{NTC-07 Eq. 3.2.9})$$

**Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.4)**

$$0 \leq T < T_B \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left[ \frac{T}{T_B} + \frac{1}{\eta \cdot F_0} \left( 1 - \frac{T}{T_B} \right) \right]$$

$$T_B \leq T < T_C \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0$$

$$T_C \leq T < T_D \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left( \frac{T_C}{T} \right)$$

$$T_D \leq T \quad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left( \frac{T_C \cdot T_D}{T^2} \right)$$

Lo spettro di progetto  $S_d(T)$  per le verifiche agli Stati Limite Ultimi è ottenuto dalle espressioni dello spettro elastico  $S_e(T)$  sostituendo  $\eta$  con  $1/q$ , dove  $q$  è il fattore di struttura. (NTC-08 § 3.2.3.5)

**Punti dello spettro di risposta**

	T [s]	Se [g]
	0.000	0.144
$T_B$ ←	0.221	0.437
$T_C$ ←	0.662	0.437
	0.724	0.400
	0.785	0.369
	0.847	0.342
	0.909	0.318
	0.971	0.298
	1.033	0.280
	1.095	0.264
	1.157	0.250
	1.218	0.238
	1.280	0.226
	1.342	0.216
	1.404	0.206
	1.466	0.198
	1.528	0.190
	1.589	0.182
	1.651	0.175
	1.713	0.169
	1.775	0.163
	1.837	0.158
	1.899	0.152
$T_D$ ←	1.960	0.148
	2.058	0.134
	2.155	0.122
	2.252	0.112
	2.349	0.103
	2.446	0.095
	2.543	0.088
	2.640	0.081
	2.737	0.076
	2.835	0.071
	2.932	0.066
	3.029	0.062
	3.126	0.058
	3.223	0.055
	3.320	0.051
	3.417	0.049
	3.514	0.046
	3.612	0.044
	3.709	0.041
	3.806	0.039
	3.903	0.037
	4.000	0.035

**Parametri e punti dello spettro di risposta verticale per lo stato linSLV**

Parametri indipendenti

STATO LIMITE	SLV
$a_{ov}$	0.037 g
$S_s$	1.000
$S_T$	1.000
$q$	1.000
$T_B$	0.050 s
$T_C$	0.150 s
$T_D$	1.000 s

Parametri dipendenti

$F_v$	1.230
$S$	1.000
$\eta$	1.000

Espressioni dei parametri dipendenti

$$S = S_s \cdot S_T \quad (\text{NTC-08 Eq. 3.2.5})$$

$$\eta = 1/q \quad (\text{NTC-08 §. 3.2.3.5})$$

$$F_v = 1,35 \cdot F_o \cdot \left(\frac{a_v}{g}\right)^{0,5} \quad (\text{NTC-08 Eq. 3.2.11})$$

Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.10)

$$0 \leq T < T_B \quad \left| \quad S_c(T) = a_v \cdot S \cdot \eta \cdot F_v \cdot \left[ \frac{T}{T_B} + \frac{1}{\eta \cdot F_v} \left( 1 - \frac{T}{T_B} \right) \right] \right.$$

$$T_B \leq T < T_C \quad \left| \quad S_c(T) = a_v \cdot S \cdot \eta \cdot F_v \right.$$

$$T_C \leq T < T_D \quad \left| \quad S_c(T) = a_v \cdot S \cdot \eta \cdot F_v \cdot \left( \frac{T_C}{T} \right) \right.$$

$$T_D \leq T \quad \left| \quad S_c(T) = a_v \cdot S \cdot \eta \cdot F_v \cdot \left( \frac{T_C \cdot T_D}{T^2} \right) \right.$$

Punti dello spettro di risposta

	T [s]	Se [g]
	0.000	0.037
$T_B$ ←	0.050	0.111
$T_C$ ←	0.150	0.111
	0.235	0.071
	0.320	0.052
	0.405	0.041
	0.490	0.034
	0.575	0.029
	0.660	0.025
	0.745	0.022
	0.830	0.020
	0.915	0.018
$T_D$ ←	1.000	0.017
	1.094	0.014
	1.188	0.012
	1.281	0.010
	1.375	0.009
	1.469	0.008
	1.563	0.007
	1.656	0.006
	1.750	0.005
	1.844	0.005
	1.938	0.004
	2.031	0.004
	2.125	0.004
	2.219	0.003
	2.313	0.003
	2.406	0.003
	2.500	0.003
	2.594	0.002
	2.688	0.002
	2.781	0.002
	2.875	0.002
	2.969	0.002
	3.063	0.002
	3.156	0.002
	3.250	0.002
	3.344	0.001
	3.438	0.001
	3.531	0.001
	3.625	0.001
	3.719	0.001
	3.813	0.001
	3.906	0.001
	4.000	0.001



Determinazione dell'azione di progetto SLD

**Parametri e punti dello spettro di risposta orizzontale per lo stato SLD**

Parametri indipendenti

STATO LIMITE	SLD
$a_g$	0.056 g
$F_o$	2.783
$T_c$	0.312 s
$S_s$	1.600
$C_c$	1.832
$S_T$	1.000
$q$	1.000

Parametri dipendenti

$S$	1.600
$\eta$	1.000
$T_B$	0.191 s
$T_C$	0.572 s
$T_D$	1.824 s

Espressioni dei parametri dipendenti

$$S = S_s \cdot S_T \quad (\text{NTC-08 Eq. 3.2.5})$$

$$\eta = \sqrt{10/(5 + \xi)} \geq 0,55; \quad \eta = 1/q \quad (\text{NTC-08 Eq. 3.2.6; §. 3.2.3.5})$$

$$T_B = T_c / 3 \quad (\text{NTC-07 Eq. 3.2.8})$$

$$T_C = C_c \cdot T_c^* \quad (\text{NTC-07 Eq. 3.2.7})$$

$$T_D = 4,0 \cdot a_g / g + 1,6 \quad (\text{NTC-07 Eq. 3.2.9})$$

Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.4)

$$0 \leq T < T_B \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left[ \frac{T}{T_B} + \frac{1}{\eta \cdot F_o} \left( 1 - \frac{T}{T_B} \right) \right]$$

$$T_B \leq T < T_C \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_o$$

$$T_C \leq T < T_D \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left( \frac{T_C}{T} \right)$$

$$T_D \leq T \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_o \cdot \left( \frac{T_C \cdot T_D}{T^2} \right)$$

Lo spettro di progetto  $S_d(T)$  per le verifiche agli Stati Limite Ultimi è ottenuto dalle espressioni dello spettro elastico  $S_e(T)$  sostituendo  $\eta$  con  $1/q$ , dove  $q$  è il fattore di struttura. (NTC-08 § 3.2.3.5)

Punti dello spettro di risposta

	T [s]	Se [g]
	0.000	0.089
$T_B$ ←	0.191	0.249
$T_C$ ←	0.572	0.249
	0.631	0.225
	0.691	0.208
	0.751	0.190
	0.810	0.176
	0.870	0.164
	0.929	0.153
	0.989	0.144
	1.049	0.136
	1.108	0.128
	1.168	0.122
	1.227	0.116
	1.287	0.111
	1.347	0.106
	1.408	0.101
	1.468	0.097
	1.528	0.093
	1.585	0.090
	1.645	0.087
	1.704	0.084
	1.764	0.081
$T_D$ ←	1.824	0.078
	1.927	0.070
	2.031	0.063
	2.135	0.057
	2.238	0.052
	2.342	0.047
	2.445	0.043
	2.549	0.040
	2.653	0.037
	2.758	0.034
	2.860	0.032
	2.964	0.030
	3.067	0.028
	3.171	0.028
	3.275	0.024
	3.378	0.023
	3.482	0.021
	3.585	0.020
	3.689	0.019
	3.793	0.018
	3.896	0.017
	4.000	0.016

**Parametri e punti dello spettro di risposta verticale per lo stato lirSLD**

Parametri indipendenti

STATO LIMITE	SLD
$a_{ov}$	0.018 g
$S_a$	1.000
$S_T$	1.000
q	1.000
$T_B$	0.050 s
$T_C$	0.150 s
$T_D$	1.000 s

Parametri dipendenti

$F_v$	0.888
S	1.000
$\eta$	1.000

Espressioni dei parametri dipendenti

$$S = S_a \cdot S_T \quad (\text{NTC-08 Eq. 3.2.5})$$

$$\eta = 1/q \quad (\text{NTC-08 §. 3.2.3.5})$$

$$F_v = 1,35 \cdot F_o \cdot \left(\frac{a_x}{g}\right)^{0,5} \quad (\text{NTC-08 Eq. 3.2.11})$$

Espressioni dello spettro di risposta (NTC-08 Eq. 3.2.10)

$$0 \leq T < T_B \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left[ \frac{T}{T_B} + \frac{1}{\eta \cdot F_o} \left( 1 - \frac{T}{T_B} \right) \right]$$

$$T_B \leq T < T_C \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_v$$

$$T_C \leq T < T_D \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left( \frac{T_C}{T} \right)$$

$$T_D \leq T \quad S_c(T) = a_g \cdot S \cdot \eta \cdot F_v \cdot \left( \frac{T_C \cdot T_D}{T} \right)$$

Punti dello spettro di risposta

	T [s]	Se [g]
	0.000	0.018
$T_B$ ←	0.050	0.050
$T_C$ ←	0.150	0.050
	0.235	0.032
	0.320	0.023
	0.405	0.018
	0.490	0.015
	0.575	0.013
	0.660	0.011
	0.745	0.010
	0.830	0.009
	0.915	0.008
$T_D$ ←	1.000	0.007
	1.094	0.006
	1.188	0.005
	1.281	0.005
	1.375	0.004
	1.469	0.003
	1.563	0.003
	1.656	0.003
	1.750	0.002
	1.844	0.002
	1.938	0.002
	2.031	0.002
	2.125	0.002
	2.219	0.002
	2.313	0.001
	2.406	0.001
	2.500	0.001
	2.594	0.001
	2.688	0.001
	2.781	0.001
	2.875	0.001
	2.969	0.001
	3.063	0.001
	3.156	0.001
	3.250	0.001
	3.344	0.001
	3.438	0.001
	3.531	0.001
	3.625	0.001
	3.719	0.001
	3.813	0.001
	3.906	0.000
	4.000	0.000

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

Per il calcolo in condizioni sismiche si utilizza il metodo dell'analisi pseudostatica in cui l'azione sismica è rappresentata da una forza statica equivalente pari al prodotto delle forze di gravità per un opportuno coefficiente sismico  $k$ . Le forze sismiche sono pertanto:

– Forza sismica orizzontale  $F_h = k_h W$

– Forza sismica verticale  $F_v = k_v W$

I valori dei coefficienti sismici orizzontali  $k_h$  e verticale  $k_v$  sono posti pari all'accelerazione massima degli spettri di progetto relativi allo stato limite considerato (SLV, SLD).

Stato limite	$k_h$	$k_v$
SLD	0.249	0.050
SLV	0.437	0.111
sld / slv	0.570	0.451

Gli effetti dell'azione sismica sono valutati tenendo conto delle masse associate ai seguenti carichi gravitazionali:

$$G_1 + G_2 + \psi_{2j} Q_{kj}$$

Nel caso specifico per i carichi dovuti al transito dei veicoli stradali si assume  $\psi_{2j} = 0$ .

I carichi gravitazionali coinvolti dall'azione sismica sono:

**Soletta di copertura** 296 kN

**Piedritti** 0.80m x 25kN/mc = 20.00 kN/m

### 7.8.1 Forze d'inerzia orizzontali (Sisma H)

#### *Soletta di copertura*

SLD  $f_h = k_h W = 0.249 \times 296 \text{ kN} = 73.71 \text{ kN}$

SLV  $f_h = k_h W = 0.437 \times 296 \text{ kN} = 129.35 \text{ kN}$

#### *Piedritti*

SLD  $f_h = k_h W = 0.249 \times 20.00 \text{ kN/m} = 4.98 \text{ kN/m}^2$

SLV  $f_h = k_h W = 0.437 \times 20.00 \text{ kN/m} = 8.74 \text{ kN/m}^2$

Le forze d'inerzia orizzontali relative allo SLV sono applicate come un carico  $f_h$  uniformemente distribuito lungo lo sviluppo dell'elemento agente da sinistra verso destra.

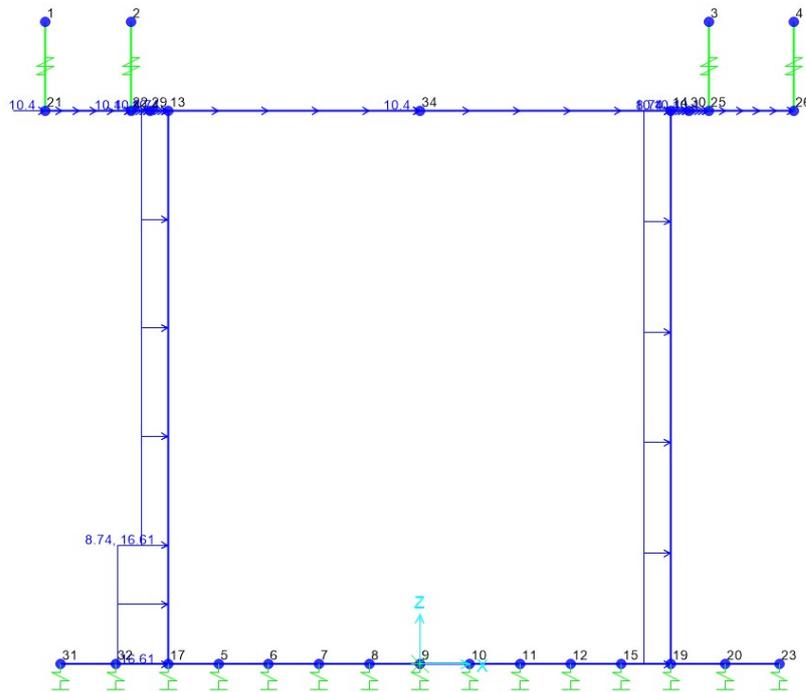


Fig. 33 – Sisma SLV – Forze inerziali e incremento si spinta del terro alla Wood

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### 7.8.2 Forze d'inerzia verticali (Sisma V)

#### Soletta di copertura

$$\text{SLD} \quad f_v = k_v W = 0.050 \times 23.68 \text{ kN/m} = 1.18 \text{ kN/m}$$

$$\text{SLV} \quad f_v = k_v W = 0.111 \times 23.68 \text{ kN/m} = 2.62 \text{ kN/m}$$

#### Piedritti

$$\text{SLD} \quad f_v = k_v W = 0.050 \times 20.00 \text{ kN/m} = 1.00 \text{ kN/m}^2$$

$$\text{SLV} \quad f_v = k_v W = 0.111 \times 20.00 \text{ kN/m} = 2.22 \text{ kN/m}^2$$

Le forze d'inerzia verticali relative allo SLV sono applicate come un carico  $f_v$  uniformemente distribuito lungo lo sviluppo dell'elemento agente verso l'alto o verso il basso, in modo da produrre gli effetti più sfavorevoli.

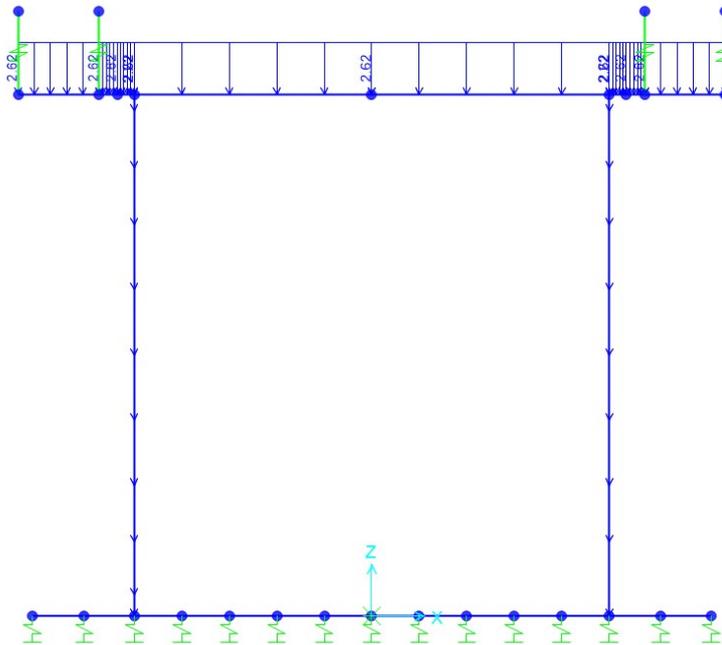


Fig. 34 – Sisma SLV\_Z – Forze inerziali verticali

### 7.9 Veicoli in svio

I sicurvia e gli elementi strutturali a cui sono collegati devono essere dimensionati per una forza orizzontale trasversale non inferiore a 100 kN. Si ipotizza questa forza ripartita su 3 montanti del sicurvia posti ad una distanza di 1.5m e con una ripartizione 25 – 50 – 25. Quindi si applica una forza di 50 kN.

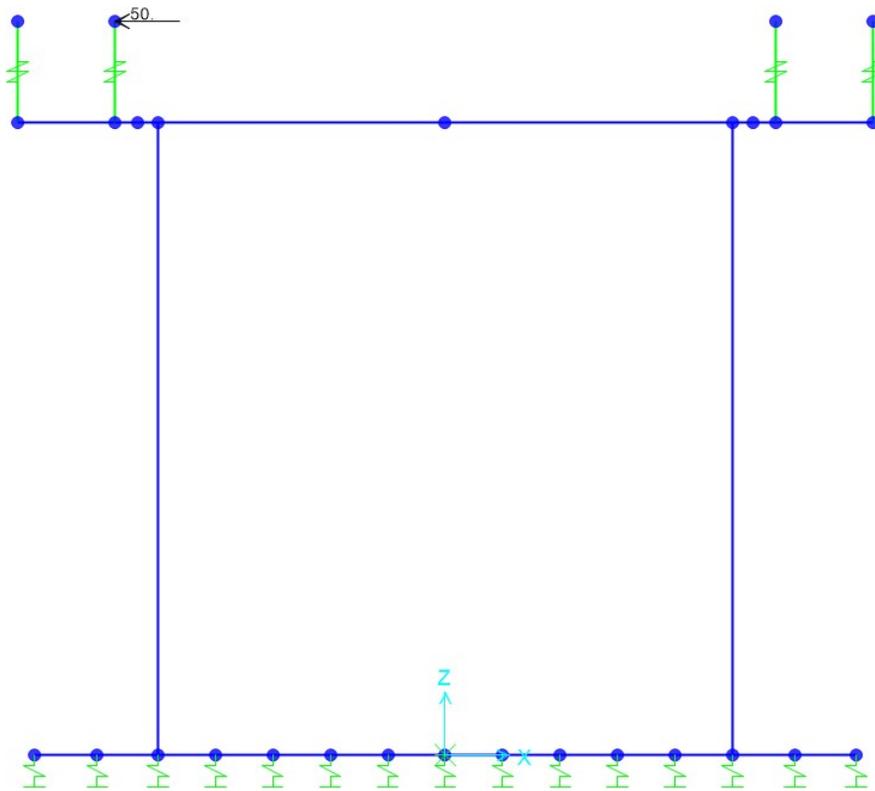


Fig. 35 – Q\_Svio

**La pressione aerodinamica del treno non è stata considerata nel calcolo degli stessi perché non è dimensionante.**

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

## 8. COMBINAZIONI DI CARICO

Le combinazioni di carico, considerate ai fini delle verifiche, sono stabilite in modo da garantire la sicurezza in conformità a quanto prescritto ai capp. 2 e 5 del DM 17/01/2018.

Gli stati limite ultimi analizzati si riferiscono al raggiungimento della resistenza degli elementi strutturali che compongono l'opera ed allo sviluppo di meccanismi di collasso determinati dalla mobilitazione della resistenza del terreno.

Le verifiche agli stati limite ultimi devono essere eseguiti in riferimento ai seguenti stati limite:

- SLU di tipo geotecnico (GEO)
  - collasso per carico limite dell'insieme fondazione-terreno;
- SLU di tipo strutturale (STR)
  - raggiungimento della resistenza negli elementi strutturali.

Le verifiche della fondazione possono essere condotte secondo l'approccio progettuale "Approccio 1", utilizzando i coefficienti parziali riportati nelle Tabelle 6.2.I e 5.1.V delle NTC per i parametri geotecnici e le azioni.

- combinazione 1  $\rightarrow (A1+M1+R1)$   $\rightarrow$  generalmente dimensionante per STR
- combinazione 2  $\rightarrow (A2+M2+R2)$   $\rightarrow$  generalmente dimensionante per GEO (carico limite)

Ai fini delle verifiche degli stati limite ultimi si definiscono le seguenti combinazioni:

$$\text{STR)} \Rightarrow \gamma_{G1} \cdot G1 + \gamma_{G2} \cdot G2 + \gamma_{Q1} \cdot Q_{k1} + \sum_i \psi_{0i} \cdot Q_{ki} \Rightarrow (\Phi_d = \Phi_k)$$

$$\text{GEO)} \Rightarrow \gamma_{G1} \cdot G1 + \gamma_{G2} \cdot G2 + \gamma_{Q1} \cdot Q_{k1} + \sum_i \psi_{0i} \cdot Q_{ki} \Rightarrow (\text{spinte } \Phi_d = \tan^{-1}(\tan \Phi_k / \gamma_{\phi}))$$

Ai fini delle verifiche degli stati limite di esercizio si definiscono le seguenti combinazioni:

$$\text{Rara)} \Rightarrow G1 + G2 + Q_{k1} + \sum_i \psi_{0i} \cdot Q_{ki} \quad \text{controllo tensioni cls / acc}$$

$$\text{Frequente)} \Rightarrow G1 + G2 + \psi_{11} \cdot Q_{k1} + \sum_i \psi_{2i} \cdot Q_{ki} \quad \text{controllo apertura fessure}$$

$$\text{Quasi permanente)} \Rightarrow G1 + G2 + \psi_{21} \cdot Q_{k1} + \sum_i \psi_{2i} \cdot Q_{ki} \quad \text{controllo tensioni cls}$$

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Per la condizione sismica, le combinazioni per gli stati limite da prendere in considerazione sono le seguenti:

$$\text{STR}) \quad \Rightarrow \quad E+G1+G2+\sum_i \psi_{2i} \cdot Q_{ki} \quad \Rightarrow (\Phi_d = \Phi_k)$$

$$\text{GEO}) \quad \Rightarrow \quad E+G1+G2+\sum_i \psi_{2i} \cdot Q_{ki} \quad \Rightarrow (\text{spinte } \Phi_d = \tan^{-1}(\tan \Phi_k / \square \Phi))$$

Gli effetti dell'azione sismica saranno valutati tenendo conto delle masse associate ai seguenti carichi gravitazionali:

$$G1+G2+\sum_i \psi_{2i} \cdot Q_{ki}$$

I valori del coefficiente  $\psi_{2i}$  sono quelli riportati nella tabella 2.5.I della norma; la stessa propone nel caso di ponti, di assumere per i carichi dovuti al transito dei mezzi  $\psi_{2i} = 0.2$  solo quando rilevanti.

Nel caso in esame si è posto  $\Psi_2 = 0$ .

Le condizioni elementari di carico, riportate nella tabella sottostante, sono state combinate in modo da determinare gli effetti più gravosi per la struttura.

Dead	Peso proprio
G2_Impianti	Peso impianti
G2_Parapetto	Peso parapetto
G2_Pavimentazione	Peso pavimentazione
G2_Sicurvia	Peso sicurvia
G2_Veletta	Peso veletta
G2_Sovraccarico pendenze	Peso sovraccarico pendenze
G3_Spinta dell'acqua orizzontale	Spinta dell'acqua orizzontale
G3_Spinta dell'acqua verticale	Spinta dell'acqua verticale
G3_Spinta terreno in DX	Spinte del terreno k0
G3_Spinta terreno in SX	Spinte del terreno k0
G3_Spinta terreno verticale	Spinta del terreno verticale
Q_Ritiro	Ritiro del calcestruzzo
Q_Svio	Svio dei veicolo
Q_Temp_Farfalla +	Variazione termica a farfalla +
Q_Temp_Farfalla -	Variazione termica a farfalla -
Q_Temp_Uni +	Variazione termica uniforme +
Q_Temp_Uni -	Variazione termica uniforme -



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Q_tr_frenatura	Q Frenatura
Q_tr_Fcentr	Q centrifuga
Q_tr_S1_C1_distr	Carico distribuito scheme 1 corsia 1
Q_tr_S1_C1_Ruota_INC_distr	Carico distribuito scheme 1 corsia 1 Incastro
Q_tr_S1_C1	Carico tandem scheme 1 corsia 1
Q_tr_S1_C1_Ruota_INC	Carico tandem scheme 1 corsia 1 Incastro
Q_tr_S1_C2_distr	Carico distribuito scheme 1 corsia 2
Q_tr_S1_C2_Ruota_INC_distr	Carico distribuito scheme 1 corsia 2 Incastro
Q_tr_S1_C2	Carico tandem scheme 1 corsia 2
Q_tr_S1_C2_Ruota_INC	Carico tandem scheme 1 corsia 2 Incastro
Q_tr_S1_C3_distr	Carico distribuito scheme 1 corsia 3
Q_tr_S1_C3_Ruota_INC_distr	Carico distribuito scheme 1 corsia 3 Incastro
Q_tr_S1_C3	Carico tandem scheme 1 corsia 3
Q_tr_S1_C3_Ruota_INC	Carico tandem scheme 1 corsia 3 Incastro
Q_tr_S2	Carico tandem scheme 2
Q_tr_S2_INC	Carico tandem scheme 2 Incastro
Q_tr_S5_folla	Schema di carico 5 - Folla
Q_Vento_Parete_Sopravento	Vento parete sopravento
Q_Vento_Parete_Sottovento	Vento parete sottovento
Q_Vento_Soletta	Vento soletta
Q_Vento_Veicoli	Vento veicoli

*Tabella 1 - Condizioni elementari di carico definite nel modello di calcolo*

Si riportano in par. 10.14 i coefficienti parziali utilizzati nelle combinazioni agli SLU, agli SLE ed agli SLV e sisma, relativamente scelti in accordo con le tabelle 5.1.IV, 5.1.V, 5.1.VI delle NTC2018 e MdP RFI 2016.

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

## 9. RISULTATI DELLE ANALISI

Si riportano di seguito una sintesi dei risultati delle analisi espressi in forma tabellare delle sollecitazioni lungo gli elementi.

La convenzione adottata per i segni delle sollecitazioni prevede che

$N < 0$                       compressione

$M > 0$                       fibre tese sul lato interno allo scatolare

Le unità di misura adottate sono

Momenti                      kNm

Forze                          kN

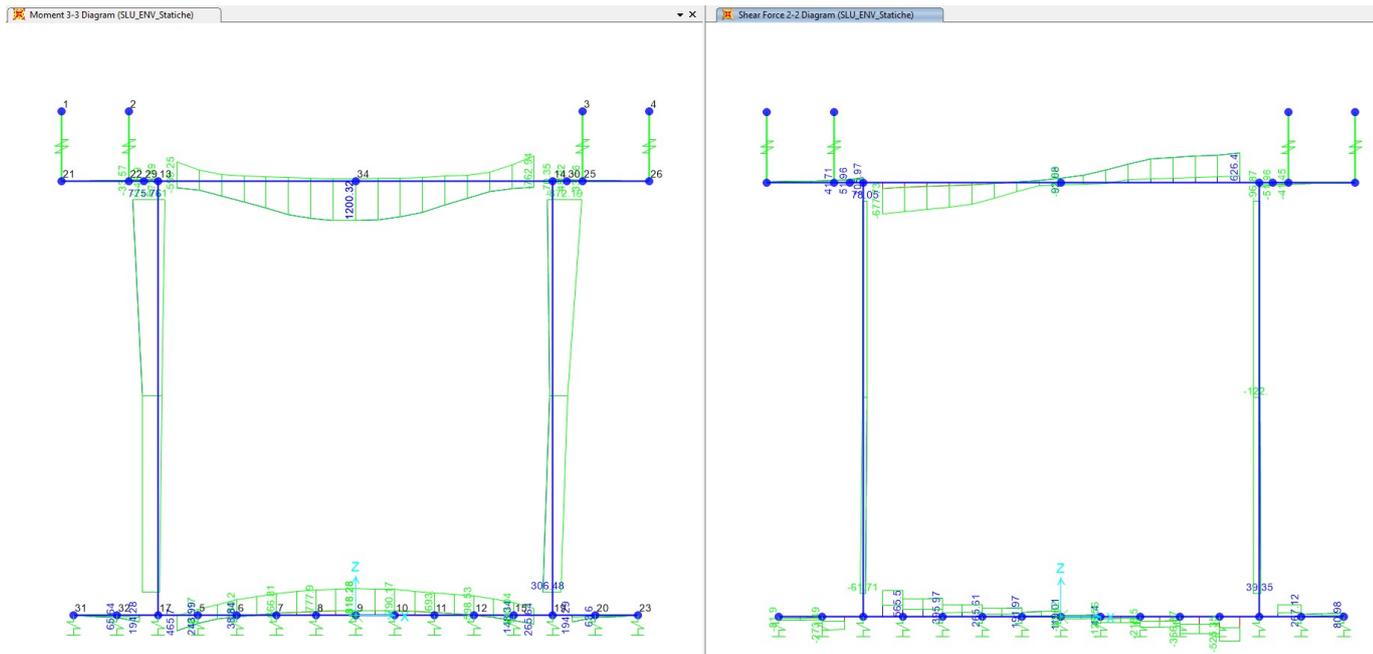


Fig. 36 – Involuppo SLU\_Static – Momenti Flettenti/ Sforzo di taglio

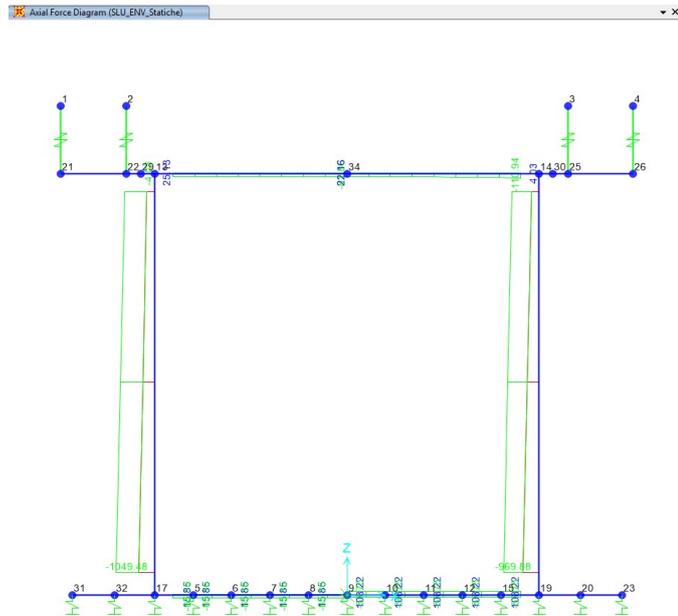


Fig. 37 – Involuppo SLU\_Static – Sforzo Normale

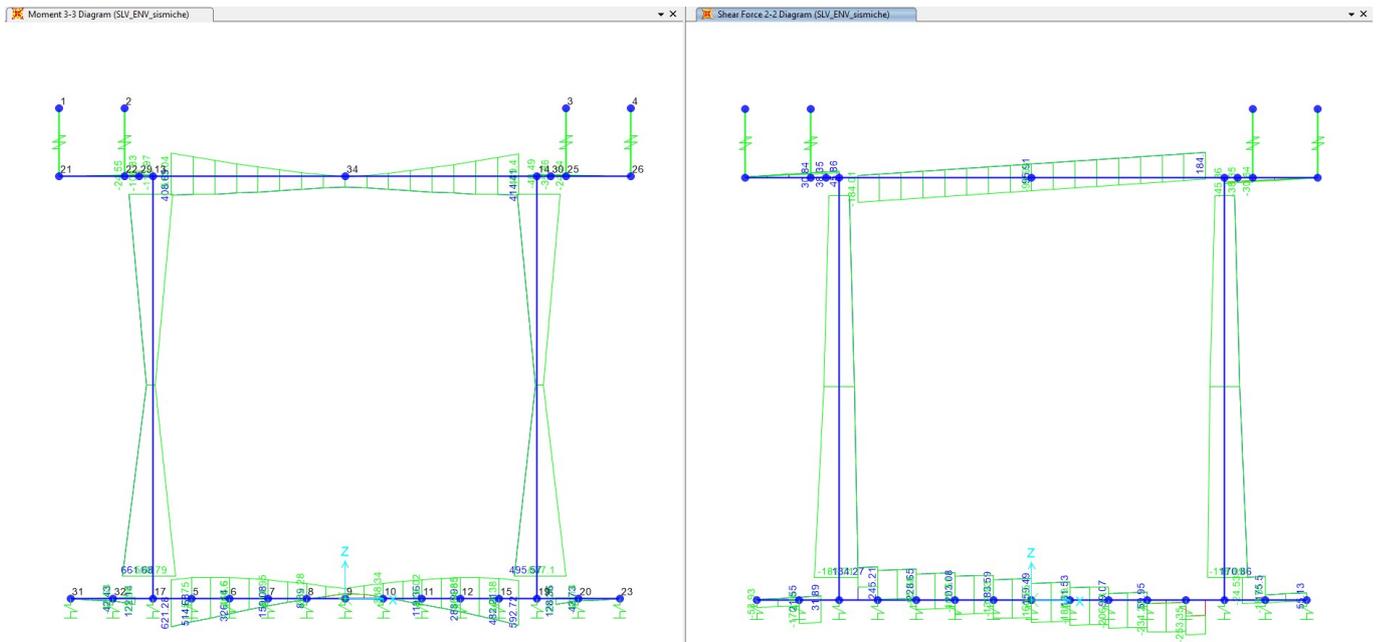


Fig. 38 – Involuppo SLV\_Sismic – Momenti Flettenti/ Sforzo di taglio

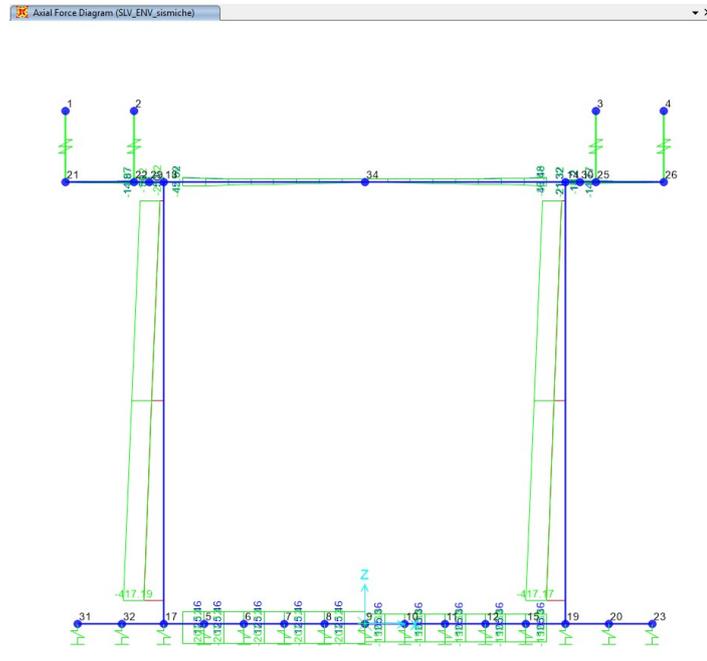


Fig. 39 – Involuppo SLV\_Sismic – Sforzo Normale

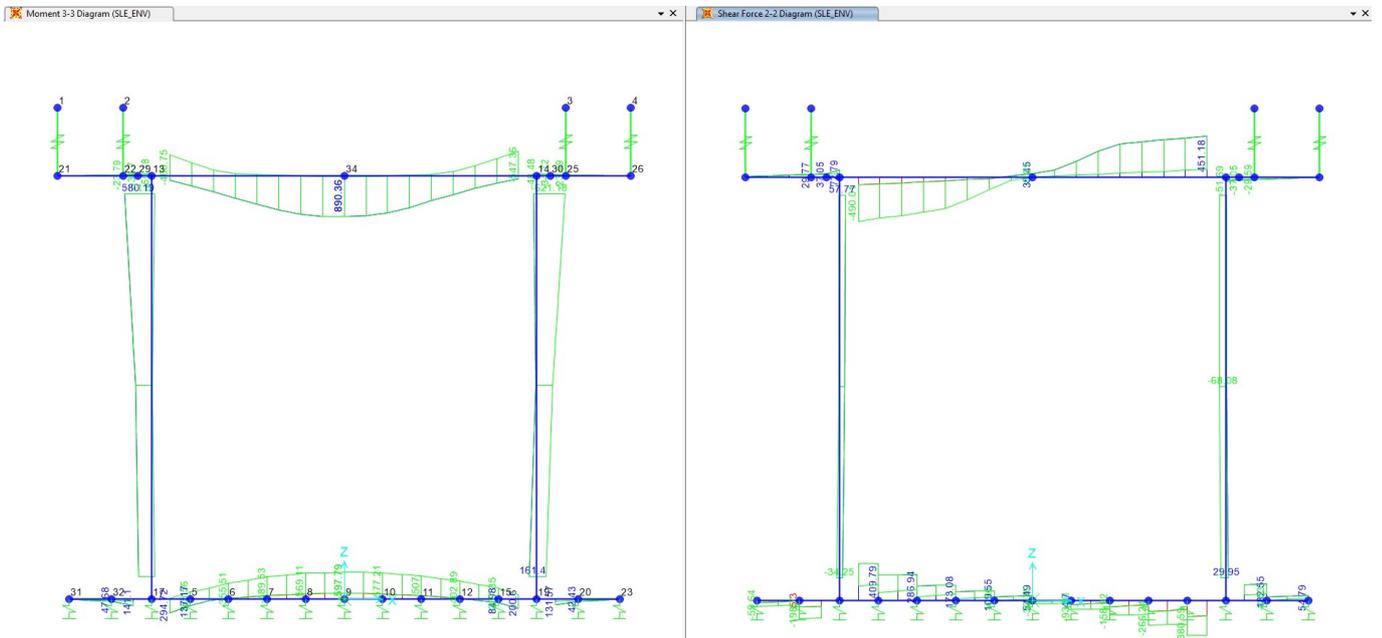


Fig. 40 – Involuppo SLE – Momento Flettente/ Sforzo di Taglio

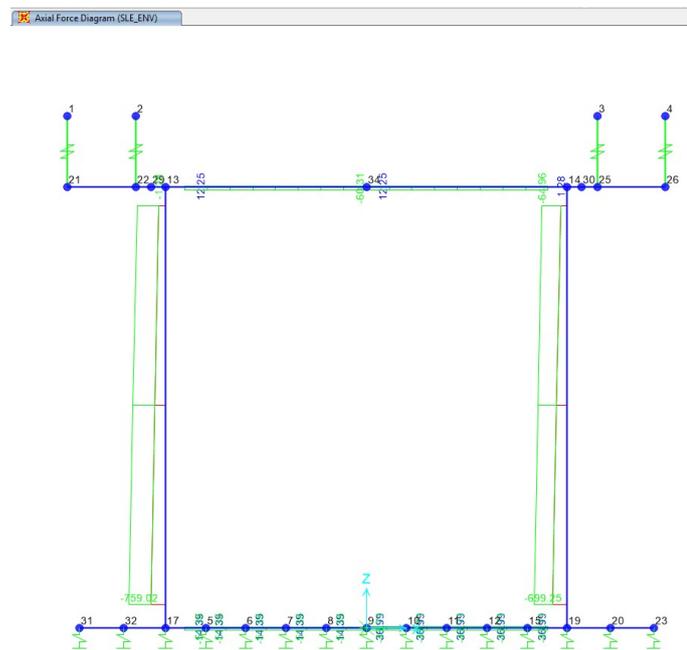


Fig. 41 – Involuppo SLE – Sforzo Normale

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## 9.1 Sollecitazioni soletta superiore – Incastro (S1)

### 9.1.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	3.8	SLU_GR_2B_S1_5	-110.935	554.75	-762.943
Nmax	37	0.4	SLU_GR_1_S2_29	25.131	-126.186	213.0641
Mmin	38	3.8	SLU_GR_2B_S1_5	-110.935	554.75	-762.943
Mmax	37	0.4	SLU_GR_1_S2_29	25.131	-126.186	213.0641
Vmin	37	0.4	SLU_GR_1_s1_1	-64.871	-677.732	-314.705
Vmax	38	3.8	SLU_GR_1_s1_5	-84.834	626.395	-655.28

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	37	0.4	SLV_5	-42.063	-184.011	-513.037
Nmax	38	3.8	SLV_6	46.476	-13.817	414.4148
Mmin	37	0.4	SLV_5	-42.063	-184.011	-513.037
Mmax	38	3.8	SLV_6	46.476	-13.817	414.4148
Vmin	37	0.4	SLV_5	-42.063	-184.011	-513.037
Vmax	38	3.8	SLV_1	-33.521	183.996	-320.238

### 9.1.2 SLE Rara/ Frequente/Quasi Permanenti

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	3.8	SLERR_GR_1_S1_5	-62.291	451.183	-487.752
Nmax	38	3.8	SLERR_GR_1_S1_38	4.299	86.596	-41.4961
Mmin	38	3.8	SLERR_GR_1_S1_25	-52.837	349.113	-488.68
Mmax	37	0.4	SLERR_GR_1_S1_29	1.72	-289.157	100.6915
Vmin	37	0.4	SLERR_GR_1_S1_5	-58.324	-490.01	-446.324
Vmax	38	3.8	SLERR_GR_1_S1_5	-62.291	451.183	-487.752

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	3.8	SLEFR_GR_2B_S1_25	-64.962	366.261	-547.365
Nmax	38	3.8	SLEFR_GR_2B_S1_35	1.683	85.09	-51.7593
Mmin	38	3.8	SLEFR_GR_2B_S1_25	-64.962	366.261	-547.365
Mmax	37	0.4	SLEFR_GR_2B_S1_29	1.215	-291.56	90.633
Vmin	37	0.4	SLEFR_GR_2A_S1_1	-32.013	-387.806	-138.34

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Vmax	38	3.8	SLEFR_GR_2B_S1_1	-48.951	368.663	-236.592
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SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	37	0.4	SLEQP_GR_1_S1_10	-3.013	-85.062	-147.199
Nmax	37	0.4	SLEQP_GR_1_S1_4	12.252	-85.062	69.2515
Mmin	38	3.8	SLEQP_GR_1_S1_10	-3.013	85.133	-147.471
Mmax	37	0.4	SLEQP_GR_1_S1_9	12.252	-85.062	69.2515
Vmin	37	0.4	SLEQP_GR_1_S1_1	6.633	-85.063	47.4693
Vmax	38	3.8	SLEQP_GR_1_S1_10	-3.013	85.133	-147.471

## 9.2 Sollecitazioni soletta superiore – Mezzeria (S2)

### 9.2.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	0	SLU_GR_1_s1_5	-81.858	23.115	888.4121
Nmax	38	0	SLU_GR_1_S2_24	22.155	5.453	442.4569
Mmin	38	0	SLU_GR_1_S2_25	-14.481	5.455	-77.0299
Mmax	38	0	SLU_GR_1_s1_4	-59.755	23.115	1200.323
Vmin	38	0	SLU_GR_1_S2_9	0.128	-61.89	497.4424
Vmax	38	0	SLU_GR_2B_S1_3	-46.862	92.084	772.7127

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	0	SLV_5	-2.543	-95.927	18.8459
Nmax	38	0	SVIO_1	34.918	-33.846	194.8826
Mmin	38	0	SLV_5	-2.543	-95.927	18.8459
Mmax	38	0	SLV_4	12.507	-28.785	243.5082
Vmin	38	0	SLV_5	-2.543	-95.927	18.8459
Vmax	38	0	SLV_2	1.825	95.913	31.0795

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### 9.2.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	0	SLERR_GR_1_S1_5	-60.308	16.722	630.6175
Nmax	38	0	SLERR_GR_1_S1_38	4.299	1.499	125.8849
Mmin	38	0	SLERR_GR_1_S1_17	0.782	-0.012	111.5318
Mmax	38	0	SLERR_GR_1_S1_4	-41.99	16.722	890.3609
Vmin	38	0	SLERR_GR_1_S1_17	0.782	-0.012	111.5318
Vmax	38	0	SLERR_GR_1_S1_7	-25.635	35.446	323.2058

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	0	SLEFR_GR_2B_S1_25	-50.043	30.594	361.9004
Nmax	38	0	SLEFR_GR_2B_S1_35	1.683	-0.00769	109.8968
Mmin	38	0	SLEFR_GR_2B_S1_35	1.683	-0.00769	109.8968
Mmax	38	0	SLEFR_GR_2A_S1_24	-19.505	11.042	794.8199
Vmin	38	0	SLEFR_GR_2B_S1_35	1.683	-0.00769	109.8968
Vmax	38	0	SLEFR_GR_2B_S1_1	-32.71	32.996	681.8001

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	38	0	SLEQP_GR_1_S1_10	-3.013	0.036	14.3505
Nmax	38	0	SLEQP_GR_1_S1_4	12.252	0.035	230.8033
Mmin	38	0	SLEQP_GR_1_S1_10	-3.013	0.036	14.3505
Mmax	38	0	SLEQP_GR_1_S1_4	12.252	0.035	230.8033
Vmin	38	0	SLEQP_GR_1_S1_4	12.252	0.035	230.8033
Vmax	38	0	SLEQP_GR_1_S1_2	2.606	0.036	36.1318



 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

### 9.3 Sollecitazioni Piedritto –Incastro (S3)

#### 9.3.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	0.4	SLU_GR_1_s1_1	-822.751	64.034	500.8249
Nmax	43	0.4	SLU_GR_1_S2_27	-212.32	-0.119	252.8137
Mmin	39	0.4	SLU_GR_2B_S1_5	-661.939	-120.491	-872.189
Mmax	43	0.4	SLU_GR_1_s1_5	-822.749	78.047	775.761
Vmin	39	0.4	SLU_GR_2B_S1_5	-661.939	-120.491	-872.189
Vmax	43	0.4	SLU_GR_1_s1_5	-822.749	78.047	775.761

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	0.4	SLV_5	-243.644	71.039	516.1586
Nmax	39	0.4	SLV_6	-41.432	75.452	430.0632
Mmin	39	0.4	SLV_2	-233.272	-66.671	-495.037
Mmax	43	0.4	SLV_5	-243.644	71.039	516.1586
Vmin	43	0.4	SLV_1	-51.806	-74.495	-425.487
Vmax	39	0.4	SLV_6	-41.432	75.452	430.0632

#### 9.3.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	0.4	SLERR_GR_1_S1_3	-591.068	48.608	453.9789
Nmax	39	0.4	SLERR_GR_1_S1_17	-138.626	0.782	-49.2693
Mmin	39	0.4	SLERR_GR_1_S1_5	-531.304	-62.753	-603.085
Mmax	43	0.4	SLERR_GR_1_S1_5	-591.067	57.767	580.1856
Vmin	39	0.4	SLERR_GR_1_S1_5	-531.304	-62.753	-603.085
Vmax	43	0.4	SLERR_GR_1_S1_5	-591.067	57.767	580.1856

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	0.4	SLEFR_GR_2A_S1_1	-482.729	31.456	237.5186
Nmax	39	0.4	SLEFR_GR_2B_S1_35	-142.531	1.683	-45.276

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Mmin	39	0.4	SLEFR_GR_2B_S1_25	-443.381	-67.742	-621.184
Mmax	43	0.4	SLEFR_GR_2A_S1_25	-471.63	49.189	549.1547
Vmin	39	0.4	SLEFR_GR_2B_S1_25	-443.381	-67.742	-621.184
Vmax	43	0.4	SLEFR_GR_2A_S1_25	-471.63	49.189	549.1547

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	39	0.4	SLEQP_GR_1_S1_10	-142.574	-3.013	-139.126
Nmax	43	0.4	SLEQP_GR_1_S1_2	-142.502	-2.606	119.3059
Mmin	39	0.4	SLEQP_GR_1_S1_5	-142.574	-3.013	-139.126
Mmax	43	0.4	SLEQP_GR_1_S1_10	-142.503	3.013	138.8406
Vmin	43	0.4	SLEQP_GR_1_S1_4	-142.503	-12.252	-71.5039
Vmax	39	0.4	SLEQP_GR_1_S1_9	-142.573	12.252	71.2227

## 9.4 Sollecitazioni Piedritto –Spiccato (S4)

### 9.4.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	8.8	SLU_GR_1_s1_1	-1049.48	28.295	3.1831
Nmax	43	8.8	SLU_GR_1_S2_22	-439.053	-38.556	307.3398
Mmin	39	8.8	SLU_GR_1_S2_22	-449.964	16.203	-246.41
Mmax	43	8.8	SLU_GR_2B_S1_2	-813.7	-43.687	466.1161
Vmin	39	8.8	SLU_GR_2B_S1_5	-888.672	-114.813	148.2874
Vmax	43	8.8	SLU_GR_1_s1_5	-1049.48	40.62	161.2583

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	8.8	SLV_5	-417.189	134.266	-386.773
Nmax	39	8.8	SLV_6	-203.788	170.862	-523.081
Mmin	39	8.8	SLV_5	-225.335	161.363	-637.105
Mmax	43	8.8	SLV_2	-203.803	-177.536	661.6788
Vmin	43	8.8	SLV_1	-225.351	-181.71	528.4717
Vmax	39	8.8	SLV_6	-203.788	170.862	-523.081

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#### 9.4.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	8.8	SLERR_GR_1_S1_1	-759.019	20.166	-8.863
Nmax	39	8.8	SLERR_GR_1_S1_17	-306.577	0.782	-55.8388
Mmin	39	8.8	SLERR_GR_1_S1_27	-464.496	-8.569	-182.898
Mmax	43	8.8	SLERR_GR_1_S1_27	-566.671	-6.763	232.828
Vmin	39	8.8	SLERR_GR_1_S1_5	-699.254	-47.073	-76.3426
Vmax	43	8.8	SLERR_GR_1_S1_5	-759.018	31.741	122.8758

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	8.8	SLEFR_GR_2A_S1_1	-650.679	5.43	1.223
Nmax	39	8.8	SLEFR_GR_2B_S1_35	-310.481	12.933	-65.0345
Mmin	39	8.8	SLEFR_GR_2B_S1_27	-465.374	-4.777	-197.052
Mmax	43	8.8	SLEFR_GR_2B_S1_22	-620.029	-2.263	283.1352
Vmin	39	8.8	SLEFR_GR_2B_S1_25	-611.331	-50.718	-58.1734
Vmax	43	8.8	SLEFR_GR_2A_S1_25	-639.58	25.851	152.6126

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	39	8.8	SLEQP_GR_1_S1_2	-310.525	20.302	-150.332
Nmax	43	8.8	SLEQP_GR_1_S1_2	-310.453	-24.6	152.1926
Mmin	39	8.8	SLEQP_GR_1_S1_7	-310.525	20.302	-150.332
Mmax	43	8.8	SLEQP_GR_1_S1_2	-310.453	-24.6	152.1926
Vmin	43	8.8	SLEQP_GR_1_S1_4	-310.454	-34.246	42.4103
Vmax	39	8.8	SLEQP_GR_1_S1_9	-310.524	29.948	-40.5428

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## 9.5 Sollecitazioni Piedritto –Mezzeria (S5)

### 9.5.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	4.6	SLU_GR_1_s1_4	-936.117	52.919	244.0841
Nmax	43	4.6	SLU_GR_1_S2_22	-325.687	-5.84	261.6151
Mmin	39	4.6	SLU_GR_1_s1_2	-856.516	-78.825	-454.735
Mmax	43	4.6	SLU_GR_2B_S1_22	-739.923	25.135	464.1955
Vmin	39	4.6	SLU_GR_2B_S1_5	-775.305	-122.003	-362.951
Vmax	43	4.6	SLU_GR_1_s1_5	-936.116	75.023	454.3145

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	4.6	SLV_5	-330.417	107.747	140.708
Nmax	39	4.6	SLV_6	-122.61	112.16	36.0765
Mmin	39	4.6	SLV_2	-314.45	-103.379	-137.934
Mmax	43	4.6	SLV_5	-330.417	107.747	140.708
Vmin	43	4.6	SLV_1	-138.578	-111.203	-35.5199
Vmax	39	4.6	SLV_6	-122.61	112.16	36.0765

### 9.5.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	4.6	SLERR_GR_1_S1_3	-675.043	46.592	254.0599
Nmax	39	4.6	SLERR_GR_1_S1_17	-222.601	0.782	-52.554
Mmin	39	4.6	SLERR_GR_1_S1_22	-506.927	-43.069	-348.815
Mmax	43	4.6	SLERR_GR_1_S1_22	-549.921	35.059	351.7657
Vmin	39	4.6	SLERR_GR_1_S1_5	-615.279	-63.761	-337.407
Vmax	43	4.6	SLERR_GR_1_S1_5	-675.042	55.751	341.7987

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SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	43	4.6	SLEFR_GR_2A_S1_1	-566.704	29.44	109.6388
Nmax	39	4.6	SLEFR_GR_2B_S1_35	-226.506	1.683	-52.3428
Mmin	39	4.6	SLEFR_GR_2A_S1_22	-507.805	-39.949	-348.454
Mmax	43	4.6	SLEFR_GR_2B_S1_22	-536.054	20.403	356.4181
Vmin	39	4.6	SLEFR_GR_2B_S1_25	-527.356	-68.078	-335.961
Vmax	43	4.6	SLEFR_GR_2A_S1_25	-555.605	48.517	343.974

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	39	4.6	SLEQP_GR_1_S1_7	-226.549	2.606	-130.539
Nmax	43	4.6	SLEQP_GR_1_S1_2	-226.478	-2.606	130.2508
Mmin	39	4.6	SLEQP_GR_1_S1_7	-226.549	2.606	-130.539
Mmax	43	4.6	SLEQP_GR_1_S1_2	-226.478	-2.606	130.2508
Vmin	43	4.6	SLEQP_GR_1_S1_4	-226.478	-12.252	-20.0452
Vmax	39	4.6	SLEQP_GR_1_S1_9	-226.549	12.252	19.764

## 9.6 Sollecitazioni soletta inferiore –incastro (S6)

### 9.6.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	2	0.4	SLU_GR_1_S2_29	-85.16	245.681	92.7611
Nmax	16	0.44	SLU_GR_2B_S1_5	108.224	-415.018	-84.5859
Mmin	16	0.44	SLU_GR_2B_S1_1	68.432	-403.578	-211.159
Mmax	2	0.4	SLU_GR_2B_S1_2	-67.141	500.921	465.7012
Vmin	16	0.44	SLU_GR_1_s1_2	38.046	-519.418	164.7427
Vmax	2	0.4	SLU_GR_1_s1_2	10.641	559.564	250.3648

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	2	0.4	SLV_1	-207.121	240.818	505.645
Nmax	2	0.4	SLV_5	125.465	117.809	-314.074
Mmin	2	0.4	SLV_6	115.966	97.598	-413.835
Mmax	2	0.4	SLV_2	-202.947	240.061	621.2757
Vmin	16	0.44	SLV_5	-182.839	-248.951	592.7188
Vmax	2	0.4	SLV_1	-207.121	240.818	505.645

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
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### 9.6.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	2	0.4	SLERR_GR_1_S1_29	-45.649	294.994	84.5492
Nmax	16	0.44	SLERR_GR_1_S1_5	33.429	-373.238	98.2648
Mmin	16	0.44	SLERR_GR_1_S1_21	4.682	-303.235	-54.6972
Mmax	2	0.4	SLERR_GR_1_S1_27	-24.109	310.996	261.7145
Vmin	16	0.44	SLERR_GR_1_S1_2	26.686	-376.197	131.002
Vmax	2	0.4	SLERR_GR_1_S1_2	7.652	405.391	190.6702

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	2	0.4	SLEFR_GR_2B_S1_29	-41.925	295.11	74.4446
Nmax	16	0.44	SLEFR_GR_2B_S1_25	36.994	-316.34	83.2611
Mmin	16	0.44	SLEFR_GR_2B_S1_21	17.702	-299.355	-104.664
Mmax	2	0.4	SLEFR_GR_2B_S1_22	-19.449	355.864	294.717
Vmin	16	0.44	SLEFR_GR_2A_S1_22	8.865	-328.594	200.6017
Vmax	2	0.4	SLEFR_GR_2B_S1_22	-19.449	355.864	294.717

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	2	0.4	SLEQP_GR_1_S1_9	-51.352	171.943	73.896
Nmax	16	0.44	SLEQP_GR_1_S1_10	-28.447	-177.404	137.806
Mmin	16	0.44	SLEQP_GR_1_S1_1	-38.093	-168.912	43.8437
Mmax	2	0.4	SLEQP_GR_1_S1_7	-41.706	180.438	167.9396
Vmin	16	0.44	SLEQP_GR_1_S1_2	-34.066	-179.87	165.087
Vmax	2	0.4	SLEQP_GR_1_S1_7	-41.706	180.438	167.9396

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## 9.7 Sollecitazioni soletta inferiore –Mezzeria (S7)

### 9.7.1 SLU

Statiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	6	0.84	SLU_GR_1_S2_29	-85.16	32.861	-326.454
Nmax	7	0	SLU_GR_2B_S1_5	108.224	34.267	-582.169
Mmin	6	0.84	SLU_GR_1_s1_1	4.841	53.741	-818.283
Mmax	7	0	SLU_GR_1_s1_1	32.246	-39.138	-818.283
Vmin	7	0	SLU_GR_1_s1_2	38.046	-41.618	-680.73
Vmax	6	0.84	SLU_GR_2B_S1_2	-67.141	119.007	-544.338

Sismiche	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	6	0.84	SLV_1	-207.121	158.686	-231.377
Nmax	6	0	SLV_5	125.465	-130.6	-270.104
Mmin	7	0.84	SLV_1	101.183	131.528	-338.335
Mmax	6	0	SLV_2	-202.947	151.092	8.392
Vmin	7	0	SLV_5	-182.839	-160.252	-163.925
Vmax	6	0.84	SLV_2	-202.947	159.485	-122.05

### 9.7.2 SLE Rara/ Frequente/Quasi Permanente

SLE_RR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	6	0.84	SLERR_GR_1_S1_29	-45.649	29.893	-396.606
Nmax	7	0	SLERR_GR_1_S1_5	33.429	-30.299	-508.957
Mmin	6	0.84	SLERR_GR_1_S1_1	2.819	38.341	-597.792
Mmax	7	0	SLERR_GR_1_S1_1	21.853	-28.698	-597.792
Vmin	7	0	SLERR_GR_1_S1_2	26.686	-30.764	-483.164
Vmax	6	0.84	SLERR_GR_1_S1_2	7.652	40.453	-483.164

SLE_FR	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	6	0.84	SLEFR_GR_2B_S1_29	-41.925	26.477	-398.251
Nmax	7	0	SLEFR_GR_2B_S1_25	36.994	-9.234	-388.85

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Mmin	6	0.84	SLEFR_GR_2A_S1_21	-10.628	29.87	-536.978
Mmax	7	0	SLEFR_GR_2A_S1_21	0.81	-26.906	-536.978
Vmin	7	0	SLEFR_GR_2A_S1_22	8.865	-30.35	-345.932
Vmax	6	0.84	SLEFR_GR_2B_S1_1	-30.087	54.487	-498.322

SLE_QP	Frame	Station	OutputCase	P	V2	M3
	Text	m	Text	kN	kN	kN*m
Nmin	6	0.84	SLEQP_GR_1_S1_9	-51.352	18.01	-207.243
Nmax	7	0	SLEQP_GR_1_S1_10	-28.447	-18.165	-154.707
Mmin	6	0.84	SLEQP_GR_1_S1_9	-51.352	18.01	-207.243
Mmax	7	0.84	SLEQP_GR_1_S1_10	-28.447	-9.771	-142.974
Vmin	7	0	SLEQP_GR_1_S1_10	-28.447	-18.165	-154.707
Vmax	6	0.84	SLEQP_GR_1_S1_9	-51.352	18.01	-207.243

## 10. VERIFICHE

### 10.1 Criteri di verifica

Per la verifica strutturale dell'opera in esame si adotta il metodo agli Stati limite. Negli Stati Limite Ultimi (SLU-SLV) si eseguono le verifiche di resistenza a pressoflessione e taglio mentre negli Stati Limite di Esercizio (SLE-SLD) si esegue il controllo delle tensioni nei materiali e dell'apertura delle fessure negli elementi in c.a..

### 10.2 Stati limite ultimi

Per le verifiche agli stati limite ultimi si adottano i valori dei coefficienti parziali nella tabella 5.1.V del DM 17.01.2018 relativa alle azioni sui ponti stradali.

La verifica di resistenza a pressoflessione delle sezioni viene eseguita con il metodo dello stato limite assumendo le ipotesi di conservazione piana delle sezioni, calcestruzzo non reagente a trazione e di perfetta aderenza tra acciaio e cls.

La verifica sugli elementi viene condotta calcolando il momento resistente massimo della sezione in presenza o meno di sforzo assiale di compressione. Il calcolo si basa sull'assunzione dei diagrammi di calcolo a tensione-deformazione del calcestruzzo e dell'acciaio previsti dalla normativa.



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Con riferimento alla sezione presso inflessa, sotto rappresentata assieme ai diagrammi di deformazione e di sforzo così come dedotti dalle ipotesi e dai modelli  $\sigma - \varepsilon$  di definiti ai paragrafi 4.1.2.1.2.2 e 4.1.2.1.2.3 del D.M.14/01/08, la verifica di resistenza (SLU) si esegue controllando che:

$$M_{Rd} = M_{Rd}(N_{Ed}) \geq M_{Ed}$$

dove:

- $N_{Ed}$  è il valore di calcolo della componente assiale (sforzo normale) dell'azione;
- $M_{Rd}$  è il valore di calcolo del momento resistente corrispondente a  $N_{Ed}$ ;
- $M_{Ed}$  è il valore di calcolo della componente flettente dell'azione.

Per la verifica nello stato limite ultimo a taglio si valuta inizialmente la resistenza  $V_{Rd1}$  dell'elemento privo di armatura trasversale. Qualora risulti inferiore alla sollecitazione di taglio  $V_{Ed}$  agente nella sezione, si predispone un'armatura trasversale e si valuta il taglio resistente  $V_{Rd}$  dell'elemento ricorrendo alla schematizzazione a traliccio ideale.

Per gli elementi senza armature trasversali resistenti a taglio la resistenza si valuta con l'espressione 4.1.14 delle DM14.01.08 di seguito riportata

$$V_{Rd} = \left\{ 0,18 \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} / \gamma_c + 0,15 \cdot \sigma_{cp} \right\} \cdot b_w \cdot d \geq (v_{\min} + 0,15 \cdot \sigma_{cp}) \cdot b_w \cdot d$$

in cui

$$k = 1 + (200/d)^{0.5} \leq 2$$

$$v_{\min} = 0,035 k^{3/2} f_{ck}^{0.5}$$

$d$  altezza utile della sezione

$\rho_l = A_{sl} / (b_w d)$  rapporto geometrico di armatura longitudinale ( $\leq 0.02$ )

$\sigma_{cp} = N_{Ed} / A_c$  tensione media di compressione nella sezione ( $\leq 0.2 f_{cd}$ ) assunta pari a 0

$b_w$  larghezza sezione

Per gli elementi dotati di specifica armatura a taglio la verifica di resistenza diventa

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$$V_{Rd} \geq V_{Ed}$$

La resistenza a taglio della trave è pari a

$$V_{Rd} = \min (V_{Rsd}; V_{Rcd})$$

in cui con riferimento all'armatura trasversale, la resistenza di calcolo a "taglio trazione" è pari a

$$V_{Rsd} = 0.9 \cdot d \cdot \frac{A_{sw}}{s} \cdot f_{yd} \cdot (ctg \alpha + ctg \theta) \cdot \sin \alpha$$

E con riferimento al calcestruzzo d'anima, la resistenza di calcolo a "taglio compressione" si calcola con

$$V_{Rcd} = 0.9 \cdot d \cdot b_w \cdot \alpha_c \cdot f'_{cd} \cdot \frac{(ctg \alpha + ctg \theta)}{(1 + ctg^2 \theta)}$$

dove  $S_d$ ,  $b_w$  e  $\sigma_{cp}$  hanno il significato già visto e inoltre si è posto:

$A_{sw}$  area dell'armatura trasversale;

$s$  interasse tra due armature trasversali consecutive;

$\alpha$  angolo di inclinazione dell'armatura trasversale rispetto all'asse della trave;

$f'_{cd}$  resistenza a compressione ridotta del calcestruzzo d'anima ( $f'_{cd} = 0,5 f_{cd}$ );

$\alpha_c$  coefficiente maggiorativo assunto pari a 1

L'inclinazione  $\theta$  dei puntoni di calcestruzzo rispetto all'asse della trave deve rispettare i limiti seguenti:  $1 \leq ctg \theta \leq 2,5$ . Nel caso in esame si è assunto  $\theta = 45^\circ$ .

### 10.3 Stati limite di esercizio

Per le verifiche agli stati limite di esercizio si adottano i valori dei coefficienti parziali di sicurezza definiti nella tabella 5.1.VI del DM 17.01.2018 relativa alle azioni sui ponti stradali. Per le azioni da traffico considerate (schema 1 con carichi tandem), si adotta il coefficiente  $\gamma_0 = 0.75$ .

I valori limite delle tensioni adottati nelle verifiche sono:

- combinazione di carico rara
  - tensione di trazione nell'acciaio 0.8  $f_{yk}$
  - tensione di compressione nel cls 0.6  $f_{ck}$

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- combinazione di carico quasi permanente:
  - tensione di compressione nel cls 0.45 f<sub>ck</sub>

I valori limite di apertura delle fessure sono quelli previsti dalle specifica di progettazione RFI per i cavalcavia stradali di cui l'opera costituisce naturale prolungamento.

Per le verifiche a fessurazione:

- condizioni XC4 e armatura ordinaria
- comb. freq. con  $w_2=0.3$  mm
- comb. q. perm. con  $w_1=0.2$  mm

Si riportano di seguito le verifiche negli stati limite analizzati relative alle sezioni significative maggiormente sollecitate individuate nella Fig. 42.

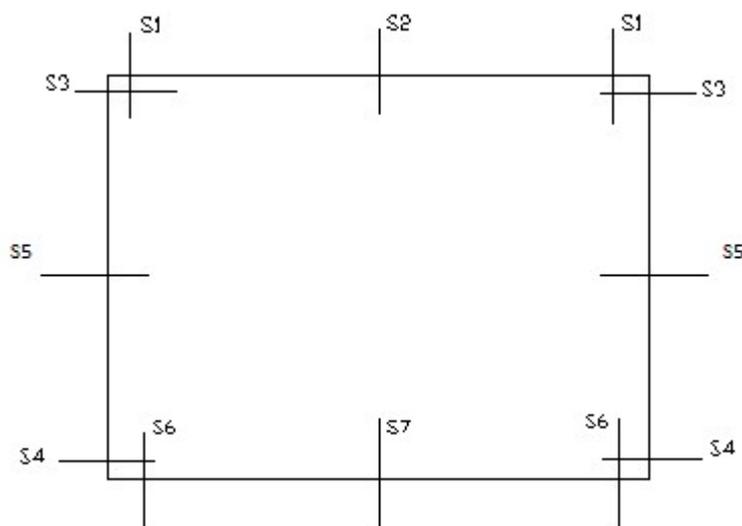


Fig. 42 –Sezioni di verifica

Per facilitare la lettura delle tabelle di sintesi delle verifiche di resistenza si riporta una legenda con i simboli e le convenzioni adottate.

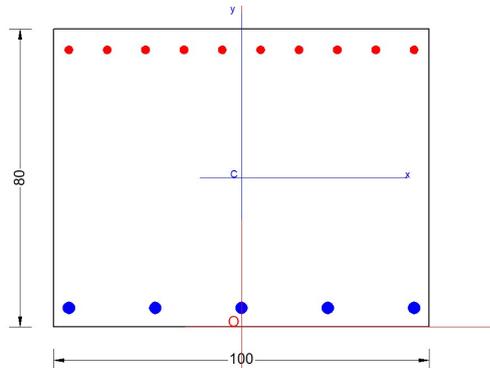
b, h: larghezza ed altezza della sezione trasversale dell'elemento esaminato

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$A_a$ :	area armatura generico strato	
$c$	copriferro nominale	
$\phi_{st}$	diametro staffe	
$c'$	copriferro totale	
$c''$	copriferro di calcolo in asse barra	
$M_{Rd}$ :	momento flettente resistente ultimo della sezione	
$M_{Ed}$ :	momento flettente di calcolo	
$\sigma_{cls} +$ :	tensione massima di compressione nel cls	[ >0 compressione]
$\sigma_{acc} -$ :	tensione massima di trazione nelle barre di acciaio	[ >0 trazione]
$w$ :	apertura fessure [ - ] : sezione non fessurata	
$R$	capacità resistente	
$E$	valore di progetto dell'effetto delle azioni	
$R / E$	coefficiente di sicurezza della verifica in esame	
$E / R$	tasso di sfruttamento del materiale	

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#### 10.4 S1 Soletta superiore – appoggio



Carpenteria		
b	h	Acls
mm	mm	m <sup>2</sup>
1000	800	0.80

Copriferro	
lato	c [mm]
esterno	40
interno	40

Materiali			
<i>Barre acciaio</i>		<i>Calcestruzzo</i>	
Tipo	B450C	Tipo	C32/40
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	32.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	40.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rar	
rara		$0.6 f_{ck}$	19.2 N/mm <sup>2</sup>
$0.8f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		$0.45 f_{ck}$	14.4 N/mm <sup>2</sup>

Armature									
Lato superiore					Lato inferiore				
n	$\phi$	Aa	c'	Aa/Acls	n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%	-	mm	mm <sup>2</sup>	mm	%
10	22	3800	51		5	30	3530	55	
-	-	-	-						
Aa,tot =		3800	0.48		Aa,tot =		3530	0.45	

#### GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S1.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\RampalRC SEC Checks\S1\Soletta superiore S1.secEC)

Section description:  
 Reference code: EN 1992-1-1 [EC2] Italian Annex  
 Exposure Class: XC4 - Carbonation (cyclic wet and dry)  
 Structural member of belonging: Beam (or slab) without stirrup [§6.2.1(4) EC2]  
 Section type: Predefined section  
 Cross-section shape: Rectangular  
 Stress path: Constant axial force to achieve bending ULS  
 Reference of assigned forces: Principal axes x,y of inertia

#### MATERIALS DATA

CONCRETE - Class: C32/40  
 Design compressive strength  $f_{cd}$ : 18.1 MPa  
 Shear reduced compressive strength  $v1*f_{cd}$ : 9.1 MPa (6.9)EC2

**RELAZIONE DI CALCOLO**

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Coeff. Crd,c:	0.1200	§ 6.2.2(1) EC2
Coeff. K1:	0.1500	§ 6.2.2(1) EC2
Coeff. v min:	0.0115	Mpa (§ 6.2.2(1) EC2
Strain at max strength ec2:	0.0020	
Ultimate strain ecu:	0.0035	
Compression diagram stress-strain:	Parabola-Rectangle	
Mean elastic modulus Ecm:	33345.8	MPa
Mean tensile strength fctm:	3.0	MPa
Es/Ec in SLS combination:	15.00	
Stress limit Characteristic SLS:	19.2	MPa
Stress limit Frequent SLS:	19.2	MPa
wk lim =limit crack width Frequent SLS:	0.300	mm
Coeff. k3:	3.4000	§ 7.3.4(3) EC2
Coeff. k4:	0.4250	§ 7.3.4(3) EC2
Stress limit Quasi-Permanent SLS:	14.4	MPa
wk lim =limit crack-width Quasi-Permanent SLS:	0.200	mm

**STEEL -**

Longitudinal Bars:	B450C	
Characteristic yield stress fyk:	450.0	MPa
Tensile strength ftk:	540.0	MPa
Design yield stress fyd:	391.3	MPa
Design strength ftd:	391.3	MPa
Design ultimate strain esu:	0.068	
Mean elastic modulus Es:	200000.0	MPa
Stirrups:	B500A	
Characteristic yield stress fyk:	500.0	MPa
Tensile strength ftk:	540.00	MPa
Design yield stress fyd:	434.8	MPa
Design strength ftd:	434.8	MPa
Mean elastic modulus Es:	200000.0	MPa
SLS Charact. - Ss Limit:	360.00	MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	80.0	cm
Bottom bars:	5L130 + 5L10	(35.3 cm?)
Top bars:	10L22 + 5L10	(38.0 cm?)
Low Cover (from centroid of bar):	5.5	cm
Top Cover (from centroid of bar):	5.1	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
VY	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.	N	Mx	Vy	MT
1	110.90	-762.90	554.80	0.00
2	-25.10	213.10	-126.20	0.00
3	110.90	-762.90	554.80	0.00
4	-25.10	213.10	-126.20	0.00
5	64.90	-314.70	-677.70	0.00
6	84.80	-655.30	626.40	0.00
7	42.10	-513.00	-184.00	0.00

**RELAZIONE DI CALCOLO**

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8	-46.50	414.40	-13.80	0.00
9	42.10	-513.00	-184.00	0.00
10	-46.50	414.40	-13.80	0.00
11	42.10	-513.00	-184.00	0.00
12	33.50	-320.20	184.00	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	62.30	-487.80
2	-4.30	-41.50
3	52.80	-488.70
4	-1.70	100.70
5	58.30	-446.30
6	62.30	-487.80

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	65.00	-547.40
2	-1.70	-51.80
3	65.00	-547.40
4	-1.20	90.60
5	32.00	-138.30
6	49.00	-236.60

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	3.00	-147.20
2	-12.30	69.30
3	3.00	-147.50
4	-12.30	69.30
5	-6.60	47.50
6	3.00	-147.50

**CHECKS RESULTS**

Min edge cover of longitudinal bars: 2.5 cm  
Min edge distance between longitudinal bars: 4.0 cm

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

### ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	110.90	-762.90	110.74	-1097.03	1.438	7.1	-1051.01	---	---	---
2	OK	-25.10	213.10	-25.31	973.89	4.564	73.8	937.75	---	---	---
3	OK	110.90	-762.90	110.74	-1097.03	1.438	7.1	-1051.01	---	---	---
4	OK	-25.10	213.10	-25.31	973.89	4.564	73.8	937.75	---	---	---
5	OK	64.90	-314.70	65.06	-1081.32	3.435	7.0	-1037.01	---	---	---
6	OK	84.80	-655.30	84.70	-1088.08	1.660	7.0	-1043.11	---	---	---
7	OK	42.10	-513.00	42.01	-1073.39	2.092	6.9	-1029.96	---	---	---
8	OK	-46.50	414.40	-46.52	966.48	2.330	73.9	930.93	---	---	---
9	OK	42.10	-513.00	42.01	-1073.39	2.092	6.9	-1029.96	---	---	---
10	OK	-46.50	414.40	-46.52	966.48	2.330	73.9	930.93	---	---	---
11	OK	42.10	-513.00	42.01	-1073.39	2.092	6.9	-1029.96	---	---	---
12	OK	33.50	-320.20	33.41	-1070.44	3.342	6.9	-1027.34	---	---	---

### ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01341	0.0	0.00079	5.5	-0.03345	74.9
2	0.00350	-0.01601	80.0	0.00060	74.9	-0.03889	5.5
3	0.00350	-0.01341	0.0	0.00079	5.5	-0.03345	74.9
4	0.00350	-0.01601	80.0	0.00060	74.9	-0.03889	5.5
5	0.00350	-0.01368	0.0	0.00074	5.5	-0.03403	74.9
6	0.00350	-0.01356	0.0	0.00076	5.5	-0.03378	74.9
7	0.00350	-0.01381	0.0	0.00072	5.5	-0.03432	74.9
8	0.00350	-0.01614	80.0	0.00058	74.9	-0.03919	5.5
9	0.00350	-0.01381	0.0	0.00072	5.5	-0.03432	74.9
10	0.00350	-0.01614	80.0	0.00058	74.9	-0.03919	5.5
11	0.00350	-0.01381	0.0	0.00072	5.5	-0.03432	74.9
12	0.00350	-0.01386	0.0	0.00072	5.5	-0.03442	74.9



 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	5.00	0.0	0.00	22.0	-180.6	5.5	12.8	1275	38.0	6.4
2	OK	0.42	0.0	0.00	20.7	-16.5	5.5	12.8	1275	38.0	6.4
3	OK	5.00	0.0	0.00	21.9	-182.0	5.5	12.8	1275	38.0	6.4
4	OK	1.04	80.0	0.00	59.8	-42.0	74.9	13.8	1375	35.3	9.9
5	OK	4.58	0.0	0.00	22.0	-165.1	5.5	12.8	1275	38.0	6.4
6	OK	5.00	0.0	0.00	22.0	-180.6	5.5	12.8	1275	38.0	6.4

### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm] cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00099	0	0.500	22 4.0	0.030	0.000544 (0.00054)	261	0.142(0.20)	-433.03
2	OK	-0.00009	0	0.500	22 4.0	0.030	0.000050 (0.00005)	261	0.013(0.20)	-417.99
3	OK	-0.00100	0	0.500	22 4.0	0.030	0.000552 (0.00055)	261	0.144(0.20)	-431.71
4	OK	-0.00023	0	0.500	30 4.0	0.026	0.000126 (0.00013)	334	0.042(0.20)	419.76
5	OK	-0.00090	0	0.500	22 4.0	0.030	0.000495 (0.00050)	261	0.129(0.20)	-433.23
6	OK	-0.00099	0	0.500	22 4.0	0.030	0.000544 (0.00054)	261	0.142(0.20)	-433.03

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	5.61	0.0	0.00	21.9	-203.2	5.5	12.8	1275	38.0	6.4
2	OK	0.53	0.0	0.00	21.1	-20.2	5.5	12.8	1275	38.0	6.4
3	OK	5.61	0.0	0.00	21.9	-203.2	5.5	12.8	1275	38.0	6.4
4	OK	0.93	80.0	0.00	59.7	-37.7	74.9	13.8	1375	35.3	9.9
5	OK	1.44	0.0	0.00	22.8	-49.5	5.5	12.8	1275	38.0	6.4

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

6 OK 2.45 0.0 0.00 22.5 -85.4 5.5 12.8 1275 38.0 6.4

#### SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00111	0	0.500	22 4.0	0.030	0.000657 (0.00061)	261	0.172 (0.20)	-432.43
2	OK	-0.00011	0	0.500	22 4.0	0.030	0.000060 (0.00006)	261	0.016 (0.20)	-422.48
3	OK	-0.00111	0	0.500	22 4.0	0.030	0.000657 (0.00061)	261	0.172 (0.20)	-432.43
4	OK	-0.00021	0	0.500	30 4.0	0.026	0.000113 (0.00011)	334	0.038 (0.20)	420.00
5	OK	-0.00027	0	0.500	22 4.0	0.030	0.000149 (0.00015)	261	0.039 (0.20)	-440.13
6	OK	-0.00047	0	0.500	22 4.0	0.030	0.000256 (0.00026)	261	0.067 (0.20)	-438.45

#### SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.50	0.0	0.00	21.5	-56.4	5.5	12.8	1275	38.0	6.4
2	OK	0.70	80.0	0.00	60.8	-30.3	74.9	13.8	1375	35.3	9.9
3	OK	1.51	0.0	0.00	21.5	-56.5	5.5	12.8	1275	38.0	6.4
4	OK	0.70	80.0	0.00	60.8	-30.3	74.9	13.8	1375	35.3	9.9
5	OK	0.48	80.0	0.00	60.5	-20.6	74.9	13.8	1375	35.3	9.9
6	OK	1.51	0.0	0.00	21.5	-56.5	5.5	12.8	1275	38.0	6.4

#### SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00031	0	0.500	22 4.0	0.030	0.000169 (0.00017)	261	0.044 (0.20)	-425.92
2	OK	-0.00017	0	0.500	30 4.0	0.026	0.000091 (0.00009)	334	0.030 (0.20)	409.61
3	OK	-0.00031	0	0.500	22 4.0	0.030	0.000169 (0.00017)	261	0.044 (0.20)	-425.92
4	OK	-0.00017	0	0.500	30 4.0	0.026	0.000091 (0.00009)	334	0.030 (0.20)	409.61
5	OK	-0.00011	0	0.500	30 4.0	0.026	0.000062 (0.00006)	334	0.021 (0.20)	412.00
6	OK	-0.00031	0	0.500	22 4.0	0.030	0.000169 (0.00017)	261	0.044 (0.20)	-425.92

#### SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.65	0.39	3900	---	---	---	-591.05	38.0	6.7
2	Frequent	Web	0.65	0.40	4000	---	---	---	-605.14	38.0	7.0
3	Frequent	Web	0.65	0.39	3900	---	---	---	-591.05	38.0	6.7
4	Frequent	Web	0.65	0.40	4050	---	---	---	-608.65	35.3	7.1
5	Frequent	Web	0.65	0.39	3850	---	---	---	-580.60	38.0	6.5
6	Frequent	Web	0.65	0.39	3850	---	---	---	-582.85	38.0	6.6
1	Quasi-perm.	Web	0.65	0.40	3950	---	---	---	-600.18	38.0	6.9
2	Quasi-perm.	Web	0.65	0.42	4150	---	---	---	-623.90	35.3	7.6

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

3	Quasi-perm.	Web	0.65	0.40	3950	---	---	---	-600.18	38.0	6.9
4	Quasi-perm.	Web	0.65	0.42	4150	---	---	---	-623.90	35.3	7.6
5	Quasi-perm.	Web	0.65	0.42	4100	---	---	---	-620.34	35.3	7.4
6	Quasi-perm.	Web	0.65	0.40	3950	---	---	---	-600.18	38.0	6.9

## Verifica a taglio

### VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5

#### • Caratteristiche della sezione

$b_w = 1000$	mm	larghezza	$f_{yk} = 450$	MPa	resist. caratteristica
$h = 800$	mm	altezza	$\gamma_s = 1.15$		coeff. sicurezza
$c = 40$	mm	copriferro	$f_{yd} = 391.3$	MPa	resist. di calcolo
$f_{ek} = 32$	MPa	resist. caratteristica	Armatura longitudinale tesa:		
$\gamma_c = 1.50$		coeff. sicurezza	$A_{sl,1} = 5$	Ø	$30 = 35.34$ cm <sup>2</sup>
$\alpha_{cc} = 0.85$		coeff. riduttivo	$A_{sl,2} = 0$	Ø	$0 = 0.00$ cm <sup>2</sup>
$d = 760$	mm	altezza utile	$A_{sl,3} = 0$	Ø	$0 = 0.00$ cm <sup>2</sup>
$f_{cd} = 18.13$	MPa	resist. di calcolo			$35.34$ cm <sup>2</sup>

#### • Sollecitazioni (compressione < 0, trazione > 0, taglio in valore assoluto)

$N_{ed} = 0.0$	kN	$V_{ed} = 678.0$	kN
----------------	----	------------------	----

#### • Elementi senza armature trasversali resistenti a taglio

$k = 1 + (200/d)^{1/2} < 2$	$k = 1.513$	< 2
$v_{min} = 0,035 k^{3/2} f_{ek}^{1/2}$	$v_{min} = 0.368$	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 = 0.005$	< 0.02
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} = 0.00$	MPa fcd

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ek})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

$$V_{Rd} = 339.4 \text{ kN}; \quad (\text{con } (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 280.0 \text{ kN})$$

$$V_{Rd} = 339.4 \text{ kN} \quad \text{valore di calcolo}$$

**la sezione NON è verificata in assenza di armature per il taglio**

#### • Elementi con armature trasversali resistenti a taglio

$$\theta = 22.0^\circ \quad \text{inclinaz. bielle cls} \quad \text{angolo ammissibile}$$

$$\alpha = 90.0^\circ \quad \text{inclinaz. staffe}$$

Armatura a taglio (staffatura):

$$A_{sw/s} = \text{staffe } \emptyset \quad 12 \text{ mm con n}^\circ \text{ bracci (trasv)} \quad 2 \quad \text{passo } 20 \text{ cm} = 0.113 \text{ cm}^2/\text{cm}$$

$$V_{Rsd} = 0.90 \times d \times (A_{sw/s}) \times f_{yd} \times (\cot \alpha + \cot \theta) \times \sin \alpha$$

$$f_{cd} = 9.07 \text{ MPa} \quad \text{resist. di calcolo ridotta}$$

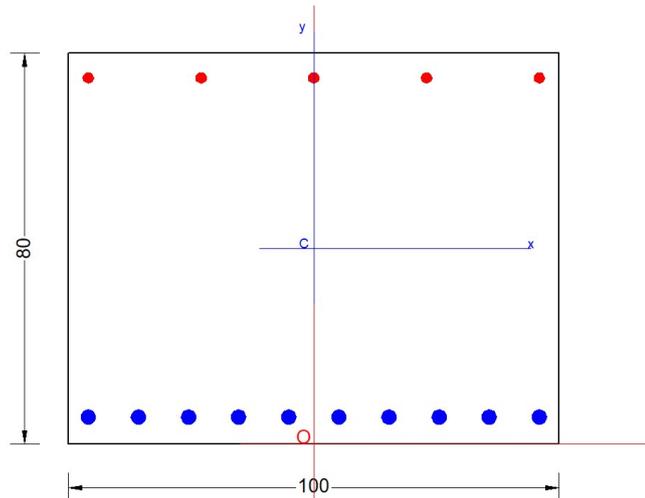
$$\alpha_c = 1.000 \quad \text{coeff. maggiorativo}$$

$$V_{Red} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \alpha + \cot \theta) / (1 + \cot^2 \alpha)$$

$$V_{Rd} = \min(V_{Red}, V_{Rsd}) \quad V_{Rd} = 749.2 > 678.0 \text{ kN} \quad \text{c.s.} = 1.1$$

la sezione armata a taglio risulta verificata.

### 10.5 S2 Soletta superiore – setto centrale



Carpenteria		
b	h	Acls
mm	mm	m <sup>2</sup>
1000	800	0.80

Copriferro	
lato	c [mm]
esterno	40
interno	40

Materiali			
Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C32/40
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	32.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	40.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rar	
rara		0.6 $f_{ck}$	19.2 N/mm <sup>2</sup>
0.8 $f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		0.45 $f_{ck}$	14.4 N/mm <sup>2</sup>

Armature				
Lato superiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
5	22	1900	51	
-	-	-	-	
Aa,tot =		1900		0.24

Lato inferiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
10	30	7070	55	
Aa,tot =		7070		0.88

### GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S2.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\RampalRC SEC Checks\S2\Soletta superiore S2.secEC)

Section description:  
Reference code: EN 1992-1-1 [EC2] Italian Annex  
Exposure Class: XC4 - Carbonation (cyclic wet and dry)  
Structural member of belonging: Beam (or slab) without stirrup [§6.2.1(4) EC2]  
Section type: Predefined section  
Cross-section shape: Rectangular  
Stress path: Constant axial force to achieve bending ULS

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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Reference of assigned forces:          Principal axes x,y of inertia

**MATERIALS DATA**

CONCRETE -	Class:	C32/40
	Design compressive strength fcd:	18.1 MPa
	Shear reduced compressive strength v1*fcd:	9.1 MPa (6.9)EC2
	Coeff. Crd,c:	0.1200 § 6.2.2(1) EC2
	Coeff. K1:	0.1500 § 6.2.2(1) EC2
	Coeff. v min:	0.0115 Mpa (§ 6.2.2(1) EC2
	Strain at max strength ec2:	0.0020
	Ultimate strain ecu:	0.0035
	Compression diagram stress-strain:	Parabola-Rectangle
	Mean elastic modulus Ecm:	33345.8 MPa
	Mean tensile strength fctm:	3.0 MPa
	Es/Ec in SLS combination:	15.00
	Stress limit Characteristic SLS:	19.2 MPa
	Stress limit Frequent SLS:	19.2 MPa
	wk lim =limit crack width Frequent SLS:	0.300 mm
	Coeff. k3:	3.4000 § 7.3.4(3) EC2
	Coeff. k4:	0.4250 § 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	14.4 MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200 mm
STEEL -	Longitudinal Bars:	B450C
	Characteristic yield stress fyk:	450.0 MPa
	Tensile strength ftk:	540.0 MPa
	Design yield stress fyd:	391.3 MPa
	Design strength ftd:	391.3 MPa
	Design ultimate strain esu:	0.068
	Mean elastic modulus Es:	200000.0 MPa
	Stirrups:	B500A
	Characteristic yield stress fyk:	500.0 MPa
	Tensile strength ftk:	540.00 MPa
	Design yield stress fyd:	434.8 MPa
	Design strength ftd:	434.8 MPa
	Mean elastic modulus Es:	200000.0 MPa
	SLS Charact. - Ss Limit:	360.00 MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	80.0	cm
Bottom bars:	10L30 + 5L10	(70.7 cm?)
Top bars:	5L22 + 5L10	(19.0 cm?)
Low Cover (from centroid of bar):	5.5	cm
Top Cover (from centroid of bar):	5.1	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
VY	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.          N          Mx          Vy          MT

**RELAZIONE DI CALCOLO**

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1	81.90	888.40	23.10	0.00
2	-22.20	442.50	5.50	0.00
3	14.50	-77.00	5.50	0.00
4	59.80	1200.30	23.10	0.00
5	-0.10	497.40	-61.90	0.00
6	46.90	772.70	92.10	0.00
7	2.50	18.80	-95.90	0.00
8	-34.90	194.90	-33.80	0.00
9	2.50	18.80	-95.90	0.00
10	-12.50	243.50	-28.80	0.00
11	2.50	18.80	-95.90	0.00
12	-1.80	31.10	95.90	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	60.30	630.60
2	-4.30	125.90
3	-0.80	111.50
4	42.00	890.40
5	-0.80	111.50
6	25.60	323.20

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	50.00	361.90
2	-1.70	109.90
3	-1.70	109.90
4	19.50	794.80
5	-1.70	109.90
6	32.70	681.80

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	3.00	14.40
2	-12.30	230.80
3	3.00	14.40
4	-12.30	230.80
5	-12.30	230.80
6	-2.60	36.10

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
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## CHECKS RESULTS

Min edge cover of longitudinal bars: 2.5 cm  
 Min edge distance between longitudinal bars: 3.6 cm

## ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	81.90	888.40	81.98	1930.03	2.168	65.7	1825.85	---	---	---
2	OK	-22.20	442.50	-21.90	1900.55	4.328	66.4	1798.59	---	---	---
3	OK	14.50	-77.00	14.50	-547.00	7.149	5.5	-526.30	---	---	---
4	OK	59.80	1200.30	59.58	1923.73	1.602	65.8	1820.13	---	---	---
5	OK	-0.10	497.40	0.09	1906.86	3.834	66.2	1804.44	---	---	---
6	OK	46.90	772.70	47.10	1920.19	2.482	65.9	1816.72	---	---	---
7	OK	2.50	18.80	2.46	1907.53	100.952	66.2	1805.07	---	---	---
8	OK	-34.90	194.90	-34.86	1896.82	10.053	66.5	1795.33	---	---	---
9	OK	2.50	18.80	2.46	1907.53	100.952	66.2	1805.07	---	---	---
10	OK	-12.50	243.50	-12.42	1903.28	7.886	66.3	1801.13	---	---	---
11	OK	2.50	18.80	2.46	1907.53	100.952	66.2	1805.07	---	---	---
12	OK	-1.80	31.10	-1.80	1906.32	61.995	66.2	1803.95	---	---	---

## ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.00487	80.0	0.00225	74.9	-0.01469	5.5
2	0.00350	-0.00531	80.0	0.00219	74.9	-0.01564	5.5
3	0.00350	-0.01851	0.0	-0.00003	5.5	-0.04458	74.9
4	0.00350	-0.00496	80.0	0.00224	74.9	-0.01488	5.5
5	0.00350	-0.00521	80.0	0.00220	74.9	-0.01542	5.5
6	0.00350	-0.00501	80.0	0.00223	74.9	-0.01500	5.5
7	0.00350	-0.00520	80.0	0.00221	74.9	-0.01540	5.5
8	0.00350	-0.00537	80.0	0.00218	74.9	-0.01576	5.5
9	0.00350	-0.00520	80.0	0.00221	74.9	-0.01540	5.5
10	0.00350	-0.00526	80.0	0.00220	74.9	-0.01554	5.5
11	0.00350	-0.00520	80.0	0.00221	74.9	-0.01540	5.5

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12      0.00350      -0.00522      80.0      0.00220      74.9      -0.01544      5.5

### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	5.82	80.0	0.00	50.4	-132.6	74.9	13.8	1375	70.7	6.4
2	OK	1.14	80.0	0.00	51.4	-27.4	74.9	13.8	1375	70.7	6.4
3	OK	1.01	80.0	0.00	51.2	-24.1	74.9	13.8	1375	70.7	6.4
4	OK	8.17	80.0	0.00	50.8	-189.8	74.9	13.8	1375	70.7	6.4
5	OK	1.01	80.0	0.00	51.2	-24.1	74.9	13.8	1375	70.7	6.4
6	OK	2.98	80.0	0.00	50.5	-68.3	74.9	13.8	1375	70.7	6.4

### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm] cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00074	0	0.500	30 4.0	0.051	0.000432 (0.00040)	235	0.102(0.20)	478.43
2	OK	-0.00015	0	0.500	30 4.0	0.051	0.000082 (0.00008)	235	0.019(0.20)	469.86
3	OK	-0.00014	0	0.500	30 4.0	0.051	0.000072 (0.00007)	235	0.017(0.20)	471.62
4	OK	-0.00106	0	0.500	30 4.0	0.051	0.000718 (0.00057)	235	0.169(0.20)	475.19
5	OK	-0.00014	0	0.500	30 4.0	0.051	0.000072 (0.00007)	235	0.017(0.20)	471.62
6	OK	-0.00038	0	0.500	30 4.0	0.051	0.000205 (0.00020)	235	0.048(0.20)	477.33

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	3.36	80.0	0.00	50.1	-75.2	74.9	13.8	1375	70.7	6.4
2	OK	1.00	80.0	0.00	51.2	-23.8	74.9	13.8	1375	70.7	6.4



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3	OK	1.00	80.0	0.00	51.2	-23.8	74.9	13.8	1375	70.7	6.4
4	OK	7.27	80.0	0.00	50.9	-170.5	74.9	13.8	1375	70.7	6.4
5	OK	1.00	80.0	0.00	51.2	-23.8	74.9	13.8	1375	70.7	6.4
6	OK	6.25	80.0	0.00	50.8	-145.3	74.9	13.8	1375	70.7	6.4

**SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00042	0	0.500	30 4.0	0.051	0.000226 (0.00023)	235	0.053 (0.20)	481.30
2	OK	-0.00013	0	0.500	30 4.0	0.051	0.000071 (0.00007)	235	0.017 (0.20)	471.08
3	OK	-0.00013	0	0.500	30 4.0	0.051	0.000071 (0.00007)	235	0.017 (0.20)	471.08
4	OK	-0.00096	0	0.500	30 4.0	0.051	0.000621 (0.00051)	235	0.146 (0.20)	473.70
5	OK	-0.00013	0	0.500	30 4.0	0.051	0.000071 (0.00007)	235	0.017 (0.20)	471.08
6	OK	-0.00081	0	0.500	30 4.0	0.051	0.000496 (0.00044)	235	0.117 (0.20)	475.24

**SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.14	80.0	0.00	47.9	-2.9	74.9	13.8	1375	70.7	6.4
2	OK	2.09	80.0	0.00	51.5	-50.6	74.9	13.8	1375	70.7	6.4
3	OK	0.14	80.0	0.00	47.9	-2.9	74.9	13.8	1375	70.7	6.4
4	OK	2.09	80.0	0.00	51.5	-50.6	74.9	13.8	1375	70.7	6.4
5	OK	2.09	80.0	0.00	51.5	-50.6	74.9	13.8	1375	70.7	6.4
6	OK	0.33	80.0	0.00	51.6	-8.0	74.9	13.8	1375	70.7	6.4

**SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00002	0	0.500	30 4.0	0.051	0.000009 (0.00001)	235	0.002 (0.20)	486.12
2	OK	-0.00028	0	0.500	30 4.0	0.051	0.000152 (0.00015)	235	0.036 (0.20)	468.63
3	OK	-0.00002	0	0.500	30 4.0	0.051	0.000009 (0.00001)	235	0.002 (0.20)	486.12
4	OK	-0.00028	0	0.500	30 4.0	0.051	0.000152 (0.00015)	235	0.036 (0.20)	468.63
5	OK	-0.00028	0	0.500	30 4.0	0.051	0.000152 (0.00015)	235	0.036 (0.20)	468.63
6	OK	-0.00004	0	0.500	30 4.0	0.051	0.000024 (0.00002)	235	0.006 (0.20)	467.42

**SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)**

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm?] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm?] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.65	0.37	3650	---	---	---	-548.75	70.7	5.9
2	Frequent	Web	0.65	0.38	3700	---	---	---	-563.11	70.7	6.1
3	Frequent	Web	0.65	0.38	3700	---	---	---	-563.11	70.7	6.1
4	Frequent	Web	0.65	0.38	3700	---	---	---	-559.38	70.7	6.1
5	Frequent	Web	0.65	0.38	3700	---	---	---	-563.11	70.7	6.1

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6	Frequent	Web	0.65	0.38	3700	---	---	---	-557.18	70.7	6.1
1	Quasi-perm.	Web	0.65	0.37	3600	---	---	---	-542.24	70.7	5.8
2	Quasi-perm.	Web	0.65	0.38	3750	---	---	---	-566.69	70.7	6.3
3	Quasi-perm.	Web	0.65	0.37	3600	---	---	---	-542.24	70.7	5.8
4	Quasi-perm.	Web	0.65	0.38	3750	---	---	---	-566.69	70.7	6.3
5	Quasi-perm.	Web	0.65	0.38	3750	---	---	---	-566.69	70.7	6.3
6	Quasi-perm.	Web	0.65	0.38	3750	---	---	---	-568.45	70.7	6.3

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

$b_w = 1000$ mm	larghezza	$f_{yk} = 450$ MPa	resist. caratteristica
$h = 800$ mm	altezza	$\gamma_s = 1.15$	coeff. sicurezza
$c = 40$ mm	copriferro	$f_{yd} = 391.3$ MPa	resist. di calcolo
$f_{ck} = 32$ MPa	resist. caratteristica	Armatura longitudinale tesa:	
$\gamma_c = 1.50$	coeff. sicurezza	$A_{sl,1} = 10$ Ø 30	= 70.69 cm <sup>2</sup>
$\alpha_{cc} = 0.85$	coeff. riduttivo	$A_{sl,2} = 0$ Ø 0	= 0.00 cm <sup>2</sup>
$d = 760$ mm	altezza utile	$A_{sl,3} = 0$ Ø 0	= 0.00 cm <sup>2</sup>
$f_{cd} = 18.13$ MPa	resist. di calcolo		70.69 cm <sup>2</sup>

**• Sollecitazioni** (compressione<0, trazione>0, taglio in valore assoluto)

$N_{ed} = 0.0$ kN	$V_{ed} = 96.0$ kN
-------------------	--------------------

**• Elementi senza armature trasversali resistenti a taglio**

$k = 1 + (200/d)^{1/2} < 2$	$k = 1.513$	<2
$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$	$v_{min} = 0.368$	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 = 0.009$	<0.02
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} = 0.00$ MPa	<0.2 f <sub>cd</sub>

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

$$V_{Rd} = 427.6 \text{ kN}; \quad (\text{con } (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 280.0 \text{ kN})$$

$$V_{Rd} = 427.6 \text{ kN} \quad \text{valore di calcolo}$$

la sezione è verificata in assenza di armature per il taglio

**• Elementi con armature trasversali resistenti a taglio**

$\theta = 22.0$ °	inclinaz. bielle cls	angolo ammissibile
$\alpha = 90.0$ °	inclinaz. staffe	

Armatura a taglio (staffatura):

$$A_{sw/s} = \text{staffe } \emptyset 12 \text{ mm con n}^\circ \text{ bracci (trasv)} \quad 2 \quad \text{passo } 40 \text{ cm} = 0.057 \text{ cm}^2/\text{cm}$$

$$V_{Rsd} = 0.90 \times d \times (A_{sw/s}) \times f_{yd} \times (\cot \alpha + \cot \theta) \times \sin \alpha$$

$$= 374.6 \text{ kN}$$

$$f_{cd} = 9.07 \text{ MPa} \quad \text{resist. di calcolo ridotta}$$

$$\alpha_c = 1.000 \quad \text{coeff. maggiorativo}$$

$$V_{Red} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \alpha + \cot \theta) / (1 + \cot^2 \alpha)$$

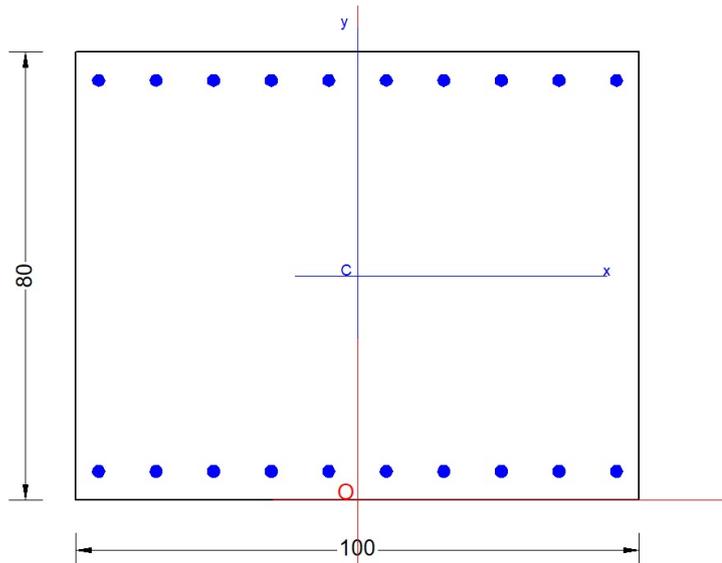
$$= 2154.0 \text{ kN}$$

$$V_{Rd} = \min(V_{Red}, V_{Rsd}) \quad V_{Rd} = 374.6 > 96.0 \text{ kN} \quad \text{c.s.} = 3.9$$

la sezione armata a taglio risulta verificata.

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

### 10.6 S3 Piedritto – testa



Carpenteria		
b	h	Acls
mm	mm	m <sup>2</sup>
1000	800	0.80

Copriferro	
lato	c [mm]
esterno	40
interno	40

Materiali			
Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C32/40
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	32.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	40.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rara	
rara		0.6 $f_{ck}$	19.2 N/mm <sup>2</sup>
0.8 $f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		0.45 $f_{ck}$	14.4 N/mm <sup>2</sup>

Armature				
Lato esterno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
10	22	3800	51	
-	-	-	-	
Aa,tot =		3800		0.48

Lato interno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
10	22	3800	51	
Aa,tot =		3800		0.48

### GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S3.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\Rampa\RC SEC Checks\S3\Soletta superiore S3.secEC)

Section description:	
Reference code:	EN 1992-1-1 [EC2] Italian Annex
Exposure Class:	XC4 - Carbonation (cyclic wet and dry)
Structural member of belonging:	Beam (or slab) without stirrup [§6.2.1(4) EC2]
Section type:	Predefined section
Cross-section shape:	Rectangular

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	92 di 155

Stress path: Constant axial force to achieve bending ULS  
Reference of assigned forces: Principal axes x,y of inertia

**MATERIALS DATA**

CONCRETE -	Class:	C32/40	
	Design compressive strength fcd:	18.1	MPa
	Shear reduced compressive strength v1*fcd:	9.1	MPa (6.9)EC2
	Coeff. Crd,c:	0.1200	§ 6.2.2(1) EC2
	Coeff. K1:	0.1500	§ 6.2.2(1) EC2
	Coeff. v min:	0.0115	Mpa (§ 6.2.2(1) EC2
	Strain at max strength ec2:	0.0020	
	Ultimate strain ecu:	0.0035	
	Compression diagram stress-strain:	Parabola-Rectangle	
	Mean elastic modulus Ecm:	33345.8	MPa
	Mean tensile strength fctm:	3.0	MPa
	Es/Ec in SLS combination:	15.00	
	Stress limit Characteristic SLS:	19.2	MPa
	Stress limit Frequent SLS:	19.2	MPa
	wk lim =limit crack width Frequent SLS:	0.300	mm
	Coeff. k3:	3.4000	§ 7.3.4(3) EC2
	Coeff. k4:	0.4250	§ 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	14.4	MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200	mm
STEEL -	Longitudinal Bars:	B450C	
	Characteristic yield stress fyk:	450.0	MPa
	Tensile strength ftk:	540.0	MPa
	Design yield stress fyd:	391.3	MPa
	Design strength ftd:	391.3	MPa
	Design ultimate strain esu:	0.068	
	Mean elastic modulus Es:	200000.0	MPa
	Stirrups:	B500A	
	Characteristic yield stress fyk:	500.0	MPa
	Tensile strength ftk:	540.00	MPa
	Design yield stress fyd:	434.8	MPa
	Design strength ftd:	434.8	MPa
	Mean elastic modulus Es:	200000.0	MPa
	SLS Charact. - Ss Limit:	360.00	MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	80.0	cm
Bottom bars:	10L22 + 5L10	(38.0 cm?)
Top bars:	10L22 + 5L10	(38.0 cm?)
Low Cover (from centroid of bar):	5.1	cm
Top Cover (from centroid of bar):	5.1	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
Vy	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.      N      Mx      Vy      MT

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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1	822.80	500.80	64.00	0.00
2	212.30	252.80	-0.10	0.00
3	661.90	-872.20	-120.50	0.00
4	822.70	775.80	78.00	0.00
5	661.90	-872.20	-120.50	0.00
6	822.70	775.80	78.00	0.00
7	243.60	516.20	71.00	0.00
8	41.40	430.10	75.50	0.00
9	233.30	-495.00	-66.70	0.00
10	243.60	516.20	71.00	0.00
11	51.80	-425.50	-74.50	0.00
12	41.40	430.10	75.50	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	591.10	454.00
2	138.60	-49.30
3	531.30	-603.10
4	591.10	580.20
5	531.30	-603.10
6	591.10	580.20

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	482.70	237.50
2	142.50	-45.30
3	443.40	-621.20
4	471.60	549.20
5	443.40	-621.20
6	471.60	549.20

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	142.60	-139.10
2	142.50	119.30
3	142.60	-139.10
4	142.50	138.80
5	142.50	-71.50
6	142.60	71.20

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

## CHECKS RESULTS

### Checks of section OK for all assigned combinations

Min edge cover of longitudinal bars: 2.9 cm  
 Min edge distance between longitudinal bars: 4.0 cm

### ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	822.80	500.80	822.59	1344.84	2.685	71.5	1266.48	---	---	---
2	OK	212.30	252.80	212.34	1134.94	4.489	73.1	1085.24	---	---	---
3	OK	661.90	-872.20	661.98	-1290.00	1.479	8.0	-1219.63	---	---	---
4	OK	822.70	775.80	822.59	1344.84	1.733	71.5	1266.48	---	---	---
5	OK	661.90	-872.20	661.98	-1290.00	1.479	8.0	-1219.63	---	---	---
6	OK	822.70	775.80	822.59	1344.84	1.733	71.5	1266.48	---	---	---
7	OK	243.60	516.20	243.39	1145.70	2.219	73.0	1094.88	---	---	---
8	OK	41.40	430.10	41.53	1075.58	2.501	73.5	1032.74	---	---	---
9	OK	233.30	-495.00	233.51	-1142.27	2.308	6.9	-1091.70	---	---	---
10	OK	243.60	516.20	243.39	1145.70	2.219	73.0	1094.88	---	---	---
11	OK	51.80	-425.50	52.10	-1079.26	2.536	6.5	-1036.05	---	---	---
12	OK	41.40	430.10	41.53	1075.58	2.501	73.5	1032.74	---	---	---

### ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01063	80.0	0.00140	74.9	-0.02737	5.1
2	0.00350	-0.01394	80.0	0.00091	74.9	-0.03459	5.1
3	0.00350	-0.01145	0.0	0.00128	5.1	-0.02916	74.9
4	0.00350	-0.01063	80.0	0.00140	74.9	-0.02737	5.1
5	0.00350	-0.01145	0.0	0.00128	5.1	-0.02916	74.9
6	0.00350	-0.01063	80.0	0.00140	74.9	-0.02737	5.1
7	0.00350	-0.01376	80.0	0.00093	74.9	-0.03420	5.1
8	0.00350	-0.01496	80.0	0.00075	74.9	-0.03682	5.1
9	0.00350	-0.01381	0.0	0.00092	5.1	-0.03432	74.9
10	0.00350	-0.01376	80.0	0.00093	74.9	-0.03420	5.1

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	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

11	0.00350	-0.01489	0.0	0.00076	5.1	-0.03668	74.9
12	0.00350	-0.01496	80.0	0.00075	74.9	-0.03682	5.1

### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	4.99	80.0	0.00	49.4	-108.1	74.9	12.8	1275	38.0	6.4
2	OK	0.56	0.0	0.00	47.7	-5.3	5.1	11.3	1134	38.0	6.4
3	OK	6.47	0.0	0.00	27.2	-170.5	5.1	12.8	1275	38.0	6.4
4	OK	6.28	80.0	0.00	51.7	-155.4	74.9	12.8	1275	38.0	6.4
5	OK	6.47	0.0	0.00	27.2	-170.5	5.1	12.8	1275	38.0	6.4
6	OK	6.28	80.0	0.00	51.7	-155.4	74.9	12.8	1275	38.0	6.4

### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm]   cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00060	0	0.500	22 4.0	0.030	0.000324 (0.00032)	261	0.085(0.20)	535.37
2	OK	-0.00003	0	0.500	22 4.0	0.034	0.000016 (0.00002)	248	0.004(0.20)	-756.56
3	OK	-0.00094	0	0.500	22 4.0	0.030	0.000512 (0.00051)	261	0.134(0.20)	-495.00
4	OK	-0.00086	0	0.500	22 4.0	0.030	0.000466 (0.00047)	261	0.122(0.20)	507.53
5	OK	-0.00094	0	0.500	22 4.0	0.030	0.000512 (0.00051)	261	0.134(0.20)	-495.00
6	OK	-0.00086	0	0.500	22 4.0	0.030	0.000466 (0.00047)	261	0.122(0.20)	507.53

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	2.68	80.0	0.00	42.5	-40.1	74.9	12.8	1275	38.0	6.4
2	OK	0.52	0.0	0.00	51.1	-4.0	5.1	10.1	1007	38.0	6.4

**RELAZIONE DI CALCOLO**

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3	OK	6.59	0.0	0.00	25.9	-187.1	5.1	12.8	1275	38.0	6.4
4	OK	5.89	80.0	0.00	53.0	-156.6	74.9	12.8	1275	38.0	6.4
5	OK	6.59	0.0	0.00	25.9	-187.1	5.1	12.8	1275	38.0	6.4
6	OK	5.89	80.0	0.00	53.0	-156.6	74.9	12.8	1275	38.0	6.4

**SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00023	0	0.500	22 4.0	0.030	0.000120 (0.00012)	261	0.031 (0.20)	623.60
2	OK	-0.00002	0	0.500	22 4.0	0.038	0.000012 (0.00001)	235	0.003 (0.20)	-832.78
3	OK	-0.00103	0	0.500	22 4.0	0.030	0.000577 (0.00056)	261	0.151 (0.20)	-480.61
4	OK	-0.00087	0	0.500	22 4.0	0.030	0.000470 (0.00047)	261	0.123 (0.20)	493.03
5	OK	-0.00103	0	0.500	22 4.0	0.030	0.000577 (0.00056)	261	0.151 (0.20)	-480.61
6	OK	-0.00087	0	0.500	22 4.0	0.030	0.000470 (0.00047)	261	0.123 (0.20)	493.03

**SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.51	0.0	0.00	28.5	-37.2	5.1	12.8	1275	38.0	6.4
2	OK	1.31	80.0	0.00	50.1	-29.7	74.9	12.8	1275	38.0	6.4
3	OK	1.51	0.0	0.00	28.5	-37.2	5.1	12.8	1275	38.0	6.4
4	OK	1.51	80.0	0.00	51.5	-37.1	74.9	12.8	1275	38.0	6.4
5	OK	0.80	0.0	0.00	39.8	-12.2	5.1	12.8	1275	38.0	6.4
6	OK	0.80	80.0	0.00	42.7	-12.2	74.9	12.8	1275	38.0	6.4

**SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00021	0	0.500	22 4.0	0.030	0.000111 (0.00011)	261	0.029 (0.20)	-508.12
2	OK	-0.00017	0	0.500	22 4.0	0.030	0.000089 (0.00009)	261	0.023 (0.20)	524.45
3	OK	-0.00021	0	0.500	22 4.0	0.030	0.000111 (0.00011)	261	0.029 (0.20)	-508.12
4	OK	-0.00021	0	0.500	22 4.0	0.030	0.000111 (0.00011)	261	0.029 (0.20)	508.26
5	OK	-0.00007	0	0.500	22 4.0	0.030	0.000037 (0.00004)	261	0.010 (0.20)	-618.10
6	OK	-0.00007	0	0.500	22 4.0	0.030	0.000037 (0.00004)	261	0.010 (0.20)	619.46

**SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)**

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.65	0.28	2750	---	---	---	-414.63	38.0	3.3
2	Frequent	Web	0.65	0.15	2050	---	---	---	-310.48	38.0	1.3
3	Frequent	Web	0.65	0.37	3550	---	---	---	-537.98	38.0	5.7
4	Frequent	Web	0.65	0.36	3450	---	---	---	-524.42	38.0	5.4
5	Frequent	Web	0.65	0.37	3550	---	---	---	-537.98	38.0	5.7



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6	Frequent	Web	0.65	0.36	3450	---	---	---	-524.42	38.0	5.4
1	Quasi-perm.	Web	0.65	0.35	3350	---	---	---	-508.85	38.0	5.1
2	Quasi-perm.	Web	0.65	0.34	3250	---	---	---	-493.01	38.0	4.8
3	Quasi-perm.	Web	0.65	0.35	3350	---	---	---	-508.85	38.0	5.1
4	Quasi-perm.	Web	0.65	0.35	3350	---	---	---	-508.71	38.0	5.1
5	Quasi-perm.	Web	0.65	0.28	2750	---	---	---	-418.30	38.0	3.4
6	Quasi-perm.	Web	0.65	0.28	2750	---	---	---	-417.39	38.0	3.4

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

$b_w = 1000$ mm	larghezza	$f_{yk} = 450$ MPa	resist. caratteristica
$h = 800$ mm	altezza	$\gamma_s = 1.15$	coeff. sicurezza
$c = 40$ mm	copriferro	$f_{yd} = 391.3$ MPa	resist. di calcolo
$f_{ck} = 32$ MPa	resist. caratteristica	Armatura longitudinale tesa:	
$\gamma_c = 1.50$	coeff. sicurezza	$A_{sl,1} = 10$ Ø 22	= 38.01 cm <sup>2</sup>
$\alpha_{cc} = 0.85$	coeff. riduttivo	$A_{sl,2} = 0$ Ø 0	= 0.00 cm <sup>2</sup>
$d = 760$ mm	altezza utile	$A_{sl,3} = 0$ Ø 0	= 0.00 cm <sup>2</sup>
$f_{cd} = 18.13$ MPa	resist. di calcolo		38.01 cm <sup>2</sup>

**• Sollecitazioni** (compressione<0, trazione>0, taglio in valore assoluto)

$N_{ed} = 0.0$ kN	$V_{ed} = 121.0$ kN
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**• Elementi senza armature trasversali resistenti a taglio**

$k = 1 + (200/d)^{1/2} < 2$	$k = 1.513$	<2
$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$	$v_{min} = 0.368$	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 = 0.005$	<0.02
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} = 0.00$ MPa	<0.2 fcd

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

$$V_{Rd} = 347.7 \text{ kN}; \quad (\text{con } (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 280.0 \text{ kN})$$

$$V_{Rd} = 347.7 \text{ kN} \quad \text{valore di calcolo}$$

la sezione è verificata in assenza di armature per il taglio

**• Elementi con armature trasversali resistenti a taglio**

$\theta = 22.0$ °	inclinaz. bielle cls	angolo ammissibile
$\alpha = 90.0$ °	inclinaz. staffe	

Armatura a taglio (staffatura):

$$A_{sw}/s = \text{staffe } \emptyset 12 \text{ mm con n}^\circ \text{ bracci (trasv)} \quad 2 \quad \text{passo } 40 \text{ cm} = 0.057 \text{ cm}^2/\text{cm}$$

$$V_{Rsd} = 0.90 \times d \times (A_{sw}/s) \times f_{yd} \times (\cot \alpha + \cot \theta) \times \sin \alpha$$

$$f_{cd} = 9.07 \text{ MPa} \quad \text{resist. di calcolo ridotta}$$

$$\alpha_c = 1.000 \quad \text{coeff. maggiorativo}$$

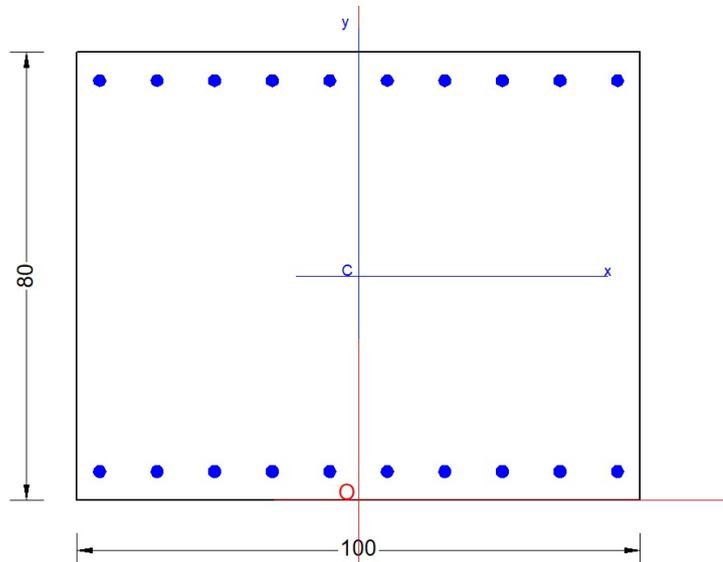
$$V_{Rd} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \alpha + \cot \theta) / (1 + \cot^2 \alpha)$$

$$V_{Rd} = \min(V_{Rcd}, V_{Rsd})$$

$$V_{Rd} = 374.6 > 121.0 \text{ kN} \quad \text{c.s.} = 3.1$$

la sezione armata a taglio risulta verificata.

### 10.7 S4 Piedritto – piede



Carpenteria		
b	h	Acls
mm	mm	m <sup>2</sup>
1000	800	0.80

Copriferro	
lato	c [mm]
esterno	40
interno	40

Materiali			
Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C32/40
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	32.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	40.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rar	
rara		0.6 $f_{ck}$	19.2 N/mm <sup>2</sup>
0.8 $f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		0.45 $f_{ck}$	14.4 N/mm <sup>2</sup>

Armature				
Lato esterno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
10	22	3800	51	
-	-	-	-	
Aa,tot =		3800		0.48

Lato interno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
10	22	3800	51	
Aa,tot =		3800		0.48

### GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S4.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\Rampa\RC SEC Checks\I4\Soletta superiore S4.secEC)

Section description:  
 Reference code: EN 1992-1-1 [EC2] Italian Annex  
 Exposure Class: XC4 - Carbonation (cyclic wet and dry)  
 Structural member of belonging: Beam (or slab) without stirrup [§6.2.1(4) EC2]  
 Section type: Predefined section  
 Cross-section shape: Rectangular  
 Stress path: Constant axial force to achieve bending ULS  
 Reference of assigned forces: Principal axes x,y of inertia

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	99 di 155

**MATERIALS DATA**

CONCRETE -	Class:	C32/40
	Design compressive strength fcd:	18.1 MPa
	Shear reduced compressive strength v1*fcd:	9.1 MPa (6.9)EC2
	Coeff. Crd.c:	0.1200 § 6.2.2(1) EC2
	Coeff. K1:	0.1500 § 6.2.2(1) EC2
	Coeff. v min:	0.0115 Mpa (§ 6.2.2(1) EC2
	Strain at max strength ec2:	0.0020
	Ultimate strain ecu:	0.0035
	Compression diagram stress-strain:	Parabola-Rectangle
	Mean elastic modulus Ecm:	33345.8 MPa
	Mean tensile strength fctm:	3.0 MPa
	Es/Ec in SLS combination:	15.00
	Stress limit Characteristic SLS:	19.2 MPa
	Stress limit Frequent SLS:	19.2 MPa
	wk lim =limit crack width Frequent SLS:	0.300 mm
	Coeff. k3:	3.4000 § 7.3.4(3) EC2
	Coeff. k4:	0.4250 § 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	14.4 MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200 mm
	STEEL -	Longitudinal Bars:
Characteristic yield stress fyk:		450.0 MPa
Tensile strength ftk:		540.0 MPa
Design yield stress fyd:		391.3 MPa
Design strength ftd:		391.3 MPa
Design ultimate strain esu:		0.068
Mean elastic modulus Es:		200000.0 MPa
Stirrups:		B500A
Characteristic yield stress fyk:		500.0 MPa
Tensile strength ftk:		540.00 MPa
Design yield stress fyd:		434.8 MPa
Design strength ftd:		434.8 MPa
Mean elastic modulus Es:		200000.0 MPa
SLS Charact. - Ss Limit:		360.00 MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	80.0	cm
Bottom bars:	10U22	(38.0 cm?)
Top bars:	10U22	(38.0 cm?)
Low Cover (from centroid of bar):	5.1	cm
Top Cover (from centroid of bar):	5.1	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
Vy	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.	N	Mx	Vy	MT
1	1049.50	3.20	28.30	0.00

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	100 di 155

2	439.10	307.30	-38.60	0.00
3	450.00	-246.40	16.20	0.00
4	813.70	466.10	-43.70	0.00
5	888.70	148.30	-114.80	0.00
6	1049.50	161.30	40.60	0.00
7	417.20	-386.80	134.30	0.00
8	203.80	-523.10	170.90	0.00
9	225.30	-637.10	161.40	0.00
10	203.80	661.70	-177.50	0.00
11	225.40	528.50	-181.70	0.00
12	203.80	-523.10	170.90	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	759.00	-8.90
2	306.60	-55.80
3	464.50	-182.90
4	566.70	232.80
5	699.30	-76.30
6	759.00	122.90

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	650.70	1.20
2	310.50	-65.00
3	465.40	-197.10
4	620.00	283.10
5	611.30	-58.20
6	639.60	152.60

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	310.50	-150.30
2	310.50	152.20
3	310.50	-150.30
4	310.50	152.20
5	310.50	42.40
6	310.50	-40.50

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

## CHECKS RESULTS

### Checks of section OK for all assigned combinations

Min edge cover of longitudinal bars: 2.9 cm

Min edge distance between longitudinal bars: 8.0 cm

### ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	1049.50	3.20	1049.30	1421.66	444.267	70.8	1331.44	---	---	---
2	OK	439.10	307.30	439.07	1213.37	3.948	72.6	1153.77	---	---	---
3	OK	450.00	-246.40	449.80	-1217.07	4.939	7.5	-1156.99	---	---	---
4	OK	813.70	466.10	813.42	1341.72	2.879	71.5	1263.83	---	---	---
5	OK	888.70	148.30	888.88	1367.41	9.221	71.3	1285.49	---	---	---
6	OK	1049.50	161.30	1049.30	1421.66	8.814	70.8	1331.44	---	---	---
7	OK	417.20	-386.80	417.16	-1205.82	3.117	7.4	-1147.15	---	---	---
8	OK	203.80	-523.10	203.70	-1131.95	2.164	6.9	-1082.67	---	---	---
9	OK	225.30	-637.10	225.58	-1139.53	1.789	6.9	-1089.27	---	---	---
10	OK	203.80	661.70	203.70	1131.95	1.711	73.1	1082.67	---	---	---
11	OK	225.40	528.50	225.58	1139.53	2.156	73.1	1089.27	---	---	---
12	OK	203.80	-523.10	203.70	-1131.95	2.164	6.9	-1082.67	---	---	---

### ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.00954	80.0	0.00156	74.9	-0.02498	5.1
2	0.00350	-0.01265	80.0	0.00110	74.9	-0.03177	5.1
3	0.00350	-0.01259	0.0	0.00111	5.1	-0.03165	74.9
4	0.00350	-0.01068	80.0	0.00139	74.9	-0.02747	5.1
5	0.00350	-0.01030	80.0	0.00145	74.9	-0.02664	5.1
6	0.00350	-0.00954	80.0	0.00156	74.9	-0.02498	5.1
7	0.00350	-0.01277	0.0	0.00108	5.1	-0.03203	74.9
8	0.00350	-0.01398	0.0	0.00090	5.1	-0.03470	74.9
9	0.00350	-0.01386	0.0	0.00092	5.1	-0.03442	74.9
10	0.00350	-0.01398	80.0	0.00090	74.9	-0.03470	5.1
11	0.00350	-0.01386	80.0	0.00092	74.9	-0.03442	5.1

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

12      0.00350      -0.01398      0.0      0.00090      5.1      -0.03470      74.9

### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.89	0.0	0.77	80.0	11.6	5.1	0.0	1275	0.0	0.0
2	OK	0.73	0.0	0.00	74.0	-0.2	5.1	2.3	225	38.0	10.0
3	OK	2.09	0.0	0.00	43.0	-23.4	5.1	12.4	1237	38.0	10.0
4	OK	2.65	80.0	0.00	38.2	-31.6	74.9	12.7	1275	38.0	10.0
5	OK	1.30	0.0	0.23	80.0	4.4	5.1	0.0	1275	0.0	0.0
6	OK	1.70	80.0	0.00	1.8	1.1	74.9	0.6	62	38.0	10.0

### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm]   cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	0.00007	0	---	---	---	---	---	---	0.00
2	OK	-0.00001	0	0.500	22 4.0	0.169	0.000001 (0.00000)	158	0.000(0.20)	-2849.50
3	OK	-0.00014	0	0.500	22 4.0	0.031	0.000070 (0.00007)	258	0.018(0.20)	-704.19
4	OK	-0.00018	0	0.500	22 4.0	0.030	0.000095 (0.00009)	261	0.025(0.20)	685.78
5	OK	0.00010	0	---	---	---	---	---	---	0.00
6	OK	0.00000	0	0.500	22 4.0	0.616	0.000003 (0.00000)	142	0.000(0.20)	9569.46

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.72	80.0	0.70	0.0	10.6	74.9	0.0	1275	0.0	0.0
2	OK	0.81	0.0	0.00	67.2	-1.4	5.1	4.3	430	38.0	10.0
3	OK	2.24	0.0	0.00	41.0	-27.9	5.1	12.8	1275	38.0	10.0

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	103 di 155

4	OK	3.21	80.0	0.00	40.9	-44.0	74.9	12.8	1275	38.0	10.0
5	OK	1.08	0.0	0.26	80.0	4.6	5.1	0.0	1275	0.0	0.0
6	OK	1.85	80.0	0.00	18.7	-6.1	74.9	6.2	622	38.0	10.0

**SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	0.00005	0	---	---	---	---	---	---	0.00
2	OK	-0.00001	0	0.500	22 4.0	0.088	0.000004 (0.00000)	178	0.001 (0.20)	-1637.71
3	OK	-0.00016	0	0.500	22 4.0	0.030	0.000084 (0.00008)	261	0.022 (0.20)	-673.57
4	OK	-0.00025	0	0.500	22 4.0	0.030	0.000132 (0.00013)	261	0.034 (0.20)	646.59
5	OK	0.00008	0	---	---	---	---	---	---	0.00
6	OK	-0.00004	0	0.500	22 4.0	0.061	0.000018 (0.00002)	197	0.004 (0.20)	1215.84

**SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.70	0.0	0.00	38.0	-24.9	5.1	12.8	1275	38.0	10.0
2	OK	1.72	80.0	0.00	42.3	-25.6	74.9	12.8	1275	38.0	10.0
3	OK	1.70	0.0	0.00	38.0	-24.9	5.1	12.8	1275	38.0	10.0
4	OK	1.72	80.0	0.00	42.3	-25.6	74.9	12.8	1275	38.0	10.0
5	OK	0.64	80.0	0.04	0.0	1.2	74.9	0.0	1275	0.0	0.0
6	OK	0.63	0.0	0.05	80.0	1.3	5.1	0.0	1275	0.0	0.0

**SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00014	0	0.500	22 4.0	0.030	0.000075 (0.00007)	261	0.020 (0.20)	-628.34
2	OK	-0.00015	0	0.500	22 4.0	0.030	0.000077 (0.00008)	261	0.020 (0.20)	624.68
3	OK	-0.00014	0	0.500	22 4.0	0.030	0.000075 (0.00007)	261	0.020 (0.20)	-628.34
4	OK	-0.00015	0	0.500	22 4.0	0.030	0.000077 (0.00008)	261	0.020 (0.20)	624.68
5	OK	0.00000	0	---	---	---	---	---	---	0.00
6	OK	0.00005	0	---	---	---	---	---	---	0.00

**SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL [§ 7.3.2 EC2]**

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web			0	---	---	---	0.00	0.0	0.0
2	Frequent	Web	0.65	-0.35	1050	---	---	---	-157.88	38.0	-1.6
3	Frequent	Web	0.65	0.25	2550	---	---	---	-383.86	38.0	2.7
4	Frequent	Web	0.65	0.26	2650	---	---	---	-399.88	38.0	3.0
5	Frequent	Web			0	---	---	---	0.00	0.0	0.0
6	Frequent	Web	0.65	-0.09	1400	---	---	---	-212.66	38.0	-0.6

**RELAZIONE DI CALCOLO**

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1	Quasi-perm.	Web	0.65	0.27	2700	---	---	---	-411.47	38.0	3.2
2	Quasi-perm.	Web	0.65	0.28	2750	---	---	---	-413.90	38.0	3.3
3	Quasi-perm.	Web	0.65	0.27	2700	---	---	---	-411.47	38.0	3.2
4	Quasi-perm.	Web	0.65	0.28	2750	---	---	---	-413.90	38.0	3.3
5	Quasi-perm.	Web			0	---	---	---	0.00	0.0	0.0
6	Quasi-perm.	Web			0	---	---	---	0.00	0.0	0.0

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

$b_w = 1000$ mm	larghezza	$f_{yk} = 450$ MPa	resist. caratteristica
$h = 800$ mm	altezza	$\gamma_s = 1.15$	coeff. sicurezza
$c = 40$ mm	copriferro	$f_{yd} = 391.3$ MPa	resist. di calcolo
$f_{ck} = 32$ MPa	resist. caratteristica	Armatura longitudinale tesa:	
$\gamma_c = 1.50$	coeff. sicurezza	$A_{sl,1} = 10$ Ø	$22 = 38.01$ cm <sup>2</sup>
$\alpha_{cc} = 0.85$	coeff. riduttivo	$A_{sl,2} = 0$ Ø	$0 = 0.00$ cm <sup>2</sup>
$d = 760$ mm	altezza utile	$A_{sl,3} = 0$ Ø	$0 = 0.00$ cm <sup>2</sup>
$f_{cd} = 18.13$ MPa	resist. di calcolo		$38.01$ cm <sup>2</sup>

**• Sollecitazioni** (compressione<0, trazione>0, taglio in valore assoluto)

$N_{ed} = 0.0$  kN       $V_{ed} = 178.0$  kN

**• Elementi senza armature trasversali resistenti a taglio**

$k = 1 + (200/d)^{1/2} < 2$        $k = 1.513$        $< 2$

$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$        $v_{min} = 0.368$

$\rho_1 = A_{sl}/(b_w \times d) < 0.02$        $\rho_1 = 0.005$        $< 0.02$

$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$        $\sigma_{cp} = 0.00$  MPa       $f_{cd}$        $< 0.2$

$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$

$V_{Rd} = 347.7$  kN;      (con  $(v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 280.0$  kN)

$V_{Rd} = 347.7$  kN      valore di calcolo

la sezione è verificata in assenza di armature per il taglio

**• Elementi con armature trasversali resistenti a taglio**

$\theta = 22.0$  °      inclinaz. bielle cls      angolo ammissibile

$\alpha = 90.0$  °      inclinaz. staffe

Armatura a taglio (staffatura):

$A_{sw}/s =$  staffe Ø  $12$  mm con n° bracci (trasv)       $2$       passo  $40$  cm =  $0.057$  cm<sup>2</sup>/cm

$V_{Rsd} = 0.90 \times d \times (A_{sw}/s) \times f_{yd} \times (\cot \gamma \alpha + \cot \theta) \times \sin \alpha$

$f_{cd} = 9.07$  MPa      resist. di calcolo ridotta

$\alpha_c = 1.000$       coeff. maggiorativo

$V_{Rsd} = 374.6$  kN

$V_{Rcd} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \gamma \alpha + \cot \theta) / (1 + \cot \gamma^2 \alpha)$

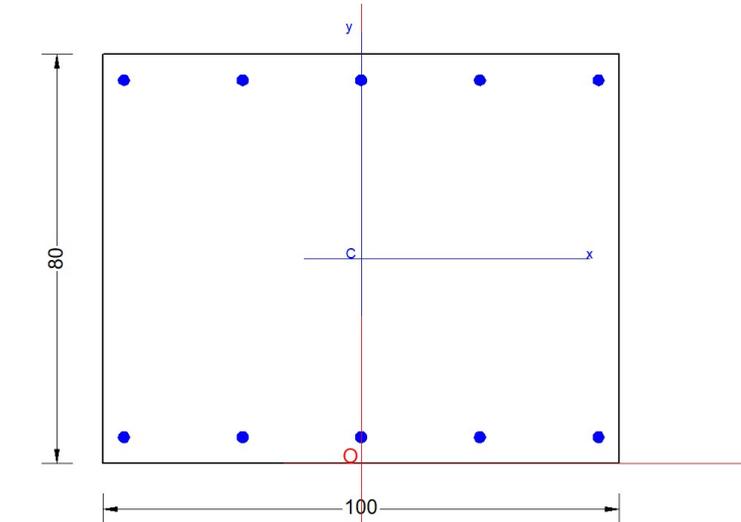
$V_{Rcd} = 2154.0$  kN

$V_{Rd} = \min(V_{Rcd}, V_{Rsd})$        $V_{Rd} = 374.6$  >  $178.0$  kN      c.s. =  $2.1$

la sezione armata a taglio risulta verificata.



### 10.8 S5 Piedritto – mezzeria



Carpenteria		
b	h	Acls
mm	mm	m <sup>2</sup>
1000	800	0.80

Copriferro	
lato	c [mm]
esterno	40
interno	40

Materiali			
Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C32/40
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	32.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	40.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rar	
rara		0.6 $f_{ck}$	19.2 N/mm <sup>2</sup>
0.8 $f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		0.45 $f_{ck}$	14.4 N/mm <sup>2</sup>

Armature				
Lato esterno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
5	22	1900	51	
-	-	-	-	
Aa,tot =		1900		0.24

Lato interno				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
5	22	1900	51	
Aa,tot =		1900		0.24

### GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S5.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\Rampa\RC SEC Checks\S5\Soletta superiore S5.secEC)

Section description:  
Reference code: EN 1992-1-1 [EC2] Italian Annex  
Exposure Class: XC4 - Carbonation (cyclic wet and dry)  
Structural member of belonging: Beam (or slab) without stirrup [§6.2.1(4) EC2]  
Section type: Predefined section  
Cross-section shape: Rectangular  
Stress path: Constant axial force to achieve bending ULS  
Reference of assigned forces: Principal axes x,y of inertia

**RELAZIONE DI CALCOLO**

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**MATERIALS DATA**

CONCRETE -	Class:	C32/40
	Design compressive strength fcd:	18.1 MPa
	Shear reduced compressive strength $v1 \cdot fcd$ :	9.1 MPa (6.9)EC2
	Coeff. Crd.c:	0.1200 § 6.2.2(1) EC2
	Coeff. K1:	0.1500 § 6.2.2(1) EC2
	Coeff. v min:	0.0115 Mpa (§ 6.2.2(1) EC2)
	Strain at max strength ec2:	0.0020
	Ultimate strain ecu:	0.0035
	Compression diagram stress-strain:	Parabola-Rectangle
	Mean elastic modulus Ecm:	33345.8 MPa
	Mean tensile strength fctm:	3.0 MPa
	Es/Ec in SLS combination:	15.00
	Stress limit Characteristic SLS:	19.2 MPa
	Stress limit Frequent SLS:	19.2 MPa
	wk lim =limit crack width Frequent SLS:	0.300 mm
	Coeff. k3:	3.4000 § 7.3.4(3) EC2
	Coeff. k4:	0.4250 § 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	14.4 MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200 mm
	STEEL -	Longitudinal Bars:
Characteristic yield stress fyk:		450.0 MPa
Tensile strength ftk:		540.0 MPa
Design yield stress fyd:		391.3 MPa
Design strength ftd:		391.3 MPa
Design ultimate strain esu:		0.068
Mean elastic modulus Es:		200000.0 MPa
Stirrups:		B500A
Characteristic yield stress fyk:		500.0 MPa
Tensile strength ftk:		540.00 MPa
Design yield stress fyd:		434.8 MPa
Design strength ftd:		434.8 MPa
Mean elastic modulus Es:		200000.0 MPa
SLS Charact. - Ss Limit:		360.00 MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	80.0	cm
Bottom bars:	5LJ22	(19.0 cm?)
Top bars:	5LJ22	(19.0 cm?)
Low Cover (from centroid of bar):	5.1	cm
Top Cover (from centroid of bar):	5.1	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
VY	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.	N	Mx	Vy	MT
1	936.10	244.10	52.90	0.00
2	325.70	261.60	-5.80	0.00

**RELAZIONE DI CALCOLO**

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3	856.50	-454.70	-78.80	0.00
4	739.90	464.20	25.10	0.00
5	775.30	-363.00	-122.00	0.00
6	936.10	454.30	75.00	0.00
7	330.40	140.70	107.70	0.00
8	122.60	36.10	112.20	0.00
9	314.50	-137.90	-103.40	0.00
10	330.40	140.70	107.70	0.00
11	138.60	-35.50	-111.20	0.00
12	122.60	36.10	112.20	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	675.00	254.10
2	222.60	-52.60
3	506.90	-348.80
4	549.90	351.80
5	615.30	-337.40
6	675.00	341.80

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	566.70	109.60
2	226.50	-52.30
3	507.80	-348.50
4	536.10	356.40
5	527.40	-336.00
6	555.60	344.00

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	226.50	-130.50
2	226.50	130.30
3	226.50	-130.50
4	226.50	130.30
5	226.50	-20.00
6	226.50	19.80

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

## CHECKS RESULTS

### Checks of section OK for all assigned combinations

Min edge cover of longitudinal bars: 2.9 cm  
 Min edge distance between longitudinal bars: 20.8 cm

## ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	936.10	244.10	936.20	866.43	3.549	71.9	808.31	---	---	---
2	OK	325.70	261.60	325.78	655.73	2.507	74.0	623.18	---	---	---
3	OK	856.50	-454.70	856.30	-839.26	1.846	7.8	-784.83	---	---	---
4	OK	739.90	464.20	740.00	799.42	1.722	72.7	750.09	---	---	---
5	OK	775.30	-363.00	775.16	-811.47	2.235	7.5	-760.78	---	---	---
6	OK	936.10	454.30	936.20	866.43	1.907	71.9	808.31	---	---	---
7	OK	330.40	140.70	330.37	657.33	4.672	74.0	624.67	---	---	---
8	OK	122.60	36.10	122.76	584.57	16.193	74.6	558.60	---	---	---
9	OK	314.50	-137.90	314.27	-651.71	4.726	5.9	-619.69	---	---	---
10	OK	330.40	140.70	330.37	657.33	4.672	74.0	624.67	---	---	---
11	OK	138.60	-35.50	138.52	-590.10	16.623	5.4	-563.77	---	---	---
12	OK	122.60	36.10	122.76	584.57	16.193	74.6	558.60	---	---	---

## ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01133	80.0	0.00129	74.9	-0.02890	5.1
2	0.00350	-0.01662	80.0	0.00051	74.9	-0.04045	5.1
3	0.00350	-0.01192	0.0	0.00121	5.1	-0.03018	74.9
4	0.00350	-0.01285	80.0	0.00107	74.9	-0.03221	5.1
5	0.00350	-0.01257	0.0	0.00111	5.1	-0.03160	74.9
6	0.00350	-0.01133	80.0	0.00129	74.9	-0.02890	5.1
7	0.00350	-0.01657	80.0	0.00051	74.9	-0.04035	5.1
8	0.00350	-0.01873	80.0	0.00019	74.9	-0.04506	5.1
9	0.00350	-0.01673	0.0	0.00049	5.1	-0.04070	74.9

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

10	0.00350	-0.01657	80.0	0.00051	74.9	-0.04035	5.1
11	0.00350	-0.01856	0.0	0.00022	5.1	-0.04470	74.9
12	0.00350	-0.01873	80.0	0.00019	74.9	-0.04506	5.1

### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	3.62	80.0	0.00	42.1	-53.0	74.9	12.8	1275	19.0	22.5
2	OK	0.73	0.0	0.00	58.4	-3.3	5.1	7.5	751	19.0	22.5
3	OK	5.17	0.0	0.00	25.9	-147.3	5.1	12.8	1275	19.0	22.5
4	OK	5.21	80.0	0.00	53.2	-140.6	74.9	12.8	1275	19.0	22.5
5	OK	4.97	0.0	0.00	29.1	-117.4	5.1	12.8	1275	19.0	22.5
6	OK	5.01	80.0	0.00	49.5	-109.3	74.9	12.8	1275	19.0	22.5

### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm]   cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00030	0	0.500	22 4.0	0.015	0.000159 (0.00016)	387	0.061(0.20)	609.27
2	OK	-0.00002	0	0.500	22 4.0	0.025	0.000010 (0.00001)	284	0.003(0.20)	-967.73
3	OK	-0.00081	0	0.500	22 4.0	0.015	0.000442 (0.00044)	387	0.171(0.20)	-474.93
4	OK	-0.00078	0	0.500	22 4.0	0.015	0.000422 (0.00042)	387	0.163(0.20)	484.68
5	OK	-0.00065	0	0.500	22 4.0	0.015	0.000352 (0.00035)	387	0.136(0.20)	-509.51
6	OK	-0.00061	0	0.500	22 4.0	0.015	0.000328 (0.00033)	387	0.127(0.20)	525.12

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.58	80.0	0.00	12.6	-2.7	74.9	4.3	426	19.0	22.5
2	OK	0.72	0.0	0.00	59.4	-3.0	5.1	7.1	713	19.0	22.5

**RELAZIONE DI CALCOLO**

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3	OK	5.17	0.0	0.00	25.9	-146.9	5.1	12.8	1275	19.0	22.5
4	OK	5.28	80.0	0.00	53.7	-146.7	74.9	12.8	1275	19.0	22.5
5	OK	4.97	0.0	0.00	26.8	-133.8	5.1	12.8	1275	19.0	22.5
6	OK	5.09	80.0	0.00	52.8	-133.8	74.9	12.8	1275	19.0	22.5

**SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00002	0	0.500	22 4.0	0.045	0.000008 (0.00001)	220	0.002 (0.20)	1490.04
2	OK	-0.00002	0	0.500	22 4.0	0.027	0.000009 (0.00001)	276	0.003 (0.20)	-1004.82
3	OK	-0.00081	0	0.500	22 4.0	0.015	0.000441 (0.00044)	387	0.170 (0.20)	-475.26
4	OK	-0.00081	0	0.500	22 4.0	0.015	0.000440 (0.00044)	387	0.170 (0.20)	479.40
5	OK	-0.00074	0	0.500	22 4.0	0.015	0.000401 (0.00040)	387	0.155 (0.20)	-485.28
6	OK	-0.00074	0	0.500	22 4.0	0.015	0.000402 (0.00040)	387	0.155 (0.20)	489.45

**SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.93	0.0	0.00	28.8	-47.6	5.1	12.8	1275	19.0	22.5
2	OK	1.92	80.0	0.00	51.7	-47.5	74.9	12.8	1275	19.0	22.5
3	OK	1.93	0.0	0.00	28.8	-47.6	5.1	12.8	1275	19.0	22.5
4	OK	1.92	80.0	0.00	51.7	-47.5	74.9	12.8	1275	19.0	22.5
5	OK	0.43	0.0	0.10	80.0	1.9	5.1	0.0	1275	0.0	0.0
6	OK	0.42	80.0	0.10	0.0	1.9	74.9	0.0	1275	0.0	0.0

**SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00026	0	0.500	22 4.0	0.015	0.000143 (0.00014)	387	0.055 (0.20)	-500.85
2	OK	-0.00026	0	0.500	22 4.0	0.015	0.000143 (0.00014)	387	0.055 (0.20)	501.10
3	OK	-0.00026	0	0.500	22 4.0	0.015	0.000143 (0.00014)	387	0.055 (0.20)	-500.85
4	OK	-0.00026	0	0.500	22 4.0	0.015	0.000143 (0.00014)	387	0.055 (0.20)	501.10
5	OK	0.00003	0	----	----	----	----	----	----	0.00
6	OK	0.00001	0	----	----	----	----	----	----	0.00

**SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)**

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.65	-0.39	1000	---	---	---	-152.21	19.0	-1.7
2	Frequent	Web	0.65	-0.05	1500	---	---	---	-225.72	19.0	-0.3
3	Frequent	Web	0.65	0.33	3150	---	---	---	-477.23	19.0	4.5
4	Frequent	Web	0.65	0.33	3150	---	---	---	-473.09	19.0	4.5
5	Frequent	Web	0.65	0.32	3100	---	---	---	-467.38	19.0	4.4

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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6	Frequent	Web	0.65	0.32	3050	---	---	---	-463.39	19.0	4.2
1	Quasi-perm.	Web	0.65	0.31	3000	---	---	---	-452.85	19.0	4.1
2	Quasi-perm.	Web	0.65	0.31	3000	---	---	---	-452.62	19.0	4.1
3	Quasi-perm.	Web	0.65	0.31	3000	---	---	---	-452.85	19.0	4.1
4	Quasi-perm.	Web	0.65	0.31	3000	---	---	---	-452.62	19.0	4.1
5	Quasi-perm.	Web			0	---	---	---	0.00	0.0	0.0
6	Quasi-perm.	Web			0	---	---	---	0.00	0.0	0.0

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

$b_w = 1000$ mm	larghezza	$f_{yk} = 450$ MPa	resist. caratteristica
$h = 800$ mm	altezza	$\gamma_s = 1.15$	coeff. sicurezza
$c = 40$ mm	copriferro	$f_{yd} = 391.3$ MPa	resist. di calcolo
$f_{ck} = 32$ MPa	resist. caratteristica	Armatura longitudinale tesa:	
$\gamma_c = 1.50$	coeff. sicurezza	$A_{sl,1} = 5 \text{ } \emptyset 22$	$= 19.01 \text{ cm}^2$
$\alpha_{cc} = 0.85$	coeff. riduttivo	$A_{sl,2} = 0 \text{ } \emptyset 0$	$= 0.00 \text{ cm}^2$
$d = 760$ mm	altezza utile	$A_{sl,3} = 0 \text{ } \emptyset 0$	$= 0.00 \text{ cm}^2$
$f_{cd} = 18.13$ MPa	resist. di calcolo		$19.01 \text{ cm}^2$

**• Sollecitazioni (compressione<0, trazione>0, taglio in valore assoluto)**

$N_{ed} = 0.0$ kN	$V_{ed} = 122.0$ kN
-------------------	---------------------

**• Elementi senza armature trasversali resistenti a taglio**

$k = 1 + (200/d)^{1/2} < 2$	$k = 1.513$	$< 2$
$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$	$v_{min} = 0.368$	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 = 0.003$	$< 0.02$
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} = 0.00$ MPa	$f_{cd}$

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

$$V_{Rd} = 276.0 \text{ kN}; \quad (\text{con } (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 280.0 \text{ kN})$$

$$V_{Rd} = 280.0 \text{ kN} \quad \text{assunto pari alla resistenza minima}$$

la sezione è verificata in assenza di armature per il taglio

**• Elementi con armature trasversali resistenti a taglio**

$\theta = 22.0$ °	inclinaz. bielle cls	angolo ammissibile
$\alpha = 90.0$ °	inclinaz. staffe	

Armatura a taglio (staffatura):

$A_{sw}/s$ staffe	$= \emptyset 12$ mm con n° bracci (trasv)	$2$	passo	$40$ cm	$= 0.057$ cm <sup>2</sup> /cm
-------------------	---	-----	-------	---------	-------------------------------

$$V_{Rsd} = 0.90 \times d \times (A_{sw}/s) \times f_{yd} \times (\cot \alpha + \cot \theta) \times \sin \alpha$$

$$f_{cd} = 9.07 \text{ MPa} \quad \text{resist. di calcolo ridotta}$$

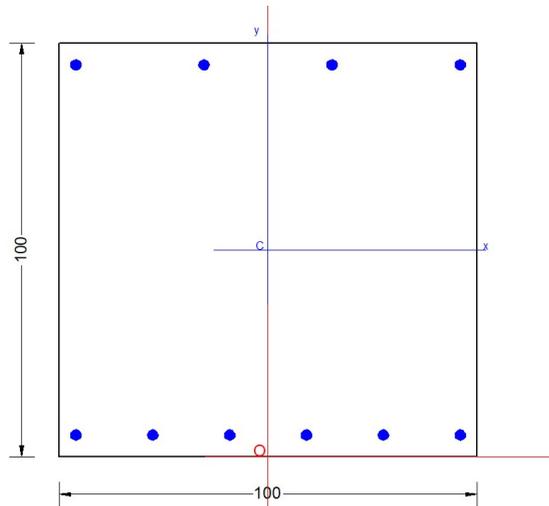
$$\alpha_c = 1.000 \quad \text{coeff. maggiorativo}$$

$$V_{Red} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \alpha + \cot \theta) / (1 + \cot^2 \alpha)$$

$$V_{Rd} = \min(V_{Red}, V_{Rsd}) \quad V_{Rd} = 374.6 > 122.0 \text{ kN} \quad \text{c.s.} = 3.1$$

la sezione armata a taglio risulta verificata.

## 10.9 S6 Fondazione – appoggio



Sezione

### Carpenteria

b	h	Acls
mm	mm	m <sup>2</sup>
1000	1000	1.00

### Copriferro

lato	c [mm]
esterno	40
interno	40

### Materiali

Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C25/30
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	25.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	30.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rara	
rara		$0.6 f_{ck}$	15.0 N/mm <sup>2</sup>
$0.8 f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		$0.45 f_{ck}$	11.25 N/mm <sup>2</sup>

### Armature

Lato inferiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
6	26	3186	60	
-	-	-	-	
Aa,tot =			3186	0.35

Lato superiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
4	26	2124	60	
		0		
Aa,tot =			2124	0.22

## GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S6.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\RampalRC SEC Checks\S6\Soletta superiore S6.secEC)

Section description:

Reference code:

Exposure Class:

Structural member of belonging:

Section type:

Cross-section shape:

Stress path:

Reference of assigned forces:

EN 1992-1-1 [EC2] Italian Annex

XC4 - Carbonation (cyclic wet and dry)

Beam (or slab) without stirrup [§6.2.1(4) EC2]

Predefined section

Rectangular

Constant axial force to achieve bending ULS

Principal axes x,y of inertia

## MATERIALS DATA



**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	113 di 155

CONCRETE -	Class:	C25/30	
	Design compressive strength fcd:	14.2	MPa
	Shear reduced compressive strength v1*fcd:	7.1	MPa (6.9)EC2
	Coeff. Crd,c:	0.1200	§ 6.2.2(1) EC2
	Coeff. K1:	0.1500	§ 6.2.2(1) EC2
	Coeff. v min:	0.0102	Mpa (§ 6.2.2(1) EC2
	Strain at max strength ec2:	0.0020	
	Ultimate strain ecu:	0.0035	
	Compression diagram stress-strain:	Parabola-Rectangle	
	Mean elastic modulus Ecm:	31475.8	MPa
	Mean tensile strength fctm:	2.6	MPa
	Es/Ec in SLS combination:	15.00	
	Stress limit Characteristic SLS:	15.0	MPa
	Stress limit Frequent SLS:	15.0	MPa
	wk lim =limit crack width Frequent SLS:	0.300	mm
	Coeff. k3:	3.4000	§ 7.3.4(3) EC2
	Coeff. k4:	0.4250	§ 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	11.3	MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200	mm
STEEL -	Longitudinal Bars:	B450C	
	Characteristic yield stress fyk:	450.0	MPa
	Tensile strength ftk:	540.0	MPa
	Design yield stress fyd:	391.3	MPa
	Design strength ftd:	391.3	MPa
	Design ultimate strain esu:	0.068	
	Mean elastic modulus Es:	200000.0	MPa
	Stirrups:	B500A	
	Characteristic yield stress fyk:	500.0	MPa
	Tensile strength ftk:	540.00	MPa
	Design yield stress fyd:	434.8	MPa
	Design strength ftd:	434.8	MPa
	Mean elastic modulus Es:	200000.0	MPa
	SLS Charact. - Ss Limit:	360.00	MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	100.0	cm
Bottom bars:	6L26	(31.9 cm?)
Top bars:	4L26 + 5L10	(21.2 cm?)
Low Cover (from centroid of bar):	5.3	cm
Top Cover (from centroid of bar):	5.3	cm
Web Cover (from centroid of bar):	4.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
Mx	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
Vy	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.	N	Mx	Vy	MT
1	85.20	92.80	245.70	0.00
2	-108.20	-84.60	-415.00	0.00
3	-68.40	-211.20	-403.60	0.00

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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4	67.10	465.70	500.90	0.00
5	-38.00	164.70	-519.40	0.00
6	-10.60	250.40	559.60	0.00
7	207.10	505.60	240.80	0.00
8	-125.50	-314.10	117.80	0.00
9	-116.00	-413.80	97.60	0.00
10	202.90	621.30	240.10	0.00
11	182.80	592.70	-249.00	0.00
12	207.10	505.60	240.80	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	45.60	84.50
2	-33.40	98.30
3	-4.70	-54.70
4	24.10	261.70
5	-26.70	131.00
6	-7.70	190.70

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	41.90	74.40
2	-37.00	83.30
3	-17.70	-104.70
4	19.40	294.70
5	-8.90	200.60
6	19.40	294.70

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
Mx Moment force [kNm] around x axis  
with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	51.40	73.90
2	28.40	137.80
3	38.10	43.80
4	41.70	167.90
5	34.10	165.10
6	41.70	167.90

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

## CHECKS RESULTS

Min edge cover of longitudinal bars: 2.7 cm  
 Min edge distance between longitudinal bars: 8.9 cm

## ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is $\geq 1.0$
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	85.20	92.80	85.18	1171.05	12.498	92.4	1116.13	---	---	---
2	OK	-108.20	-84.60	-108.16	-713.58	7.672	5.5	-690.71	---	---	---
3	OK	-68.40	-211.20	-68.41	-731.42	3.394	5.6	-707.34	---	---	---
4	OK	67.10	465.70	67.25	1163.11	2.495	92.4	1109.34	---	---	---
5	OK	-38.00	164.70	-38.21	1116.37	6.900	92.8	1068.80	---	---	---
6	OK	-10.60	250.40	-10.73	1128.58	4.520	92.7	1079.32	---	---	---
7	OK	207.10	505.60	206.81	1224.69	2.416	91.9	1162.56	---	---	---
8	OK	-125.50	-314.10	-125.67	-705.73	2.204	5.5	-683.46	---	---	---
9	OK	-116.00	-413.80	-115.77	-710.17	1.699	5.5	-687.42	---	---	---
10	OK	202.90	621.30	203.09	1223.05	1.965	91.9	1160.87	---	---	---
11	OK	182.80	592.70	182.85	1214.13	2.045	92.0	1153.43	---	---	---
12	OK	207.10	505.60	206.81	1224.69	2.416	91.9	1162.56	---	---	---

## ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at $ec_2/ec_u$ of depth (if $ec^* > 0$ then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01613	100.0	0.00107	94.7	-0.03988	5.3
2	0.00350	-0.02367	0.0	0.00014	5.3	-0.05654	94.7
3	0.00350	-0.02328	0.0	0.00019	5.3	-0.05568	94.7
4	0.00350	-0.01632	100.0	0.00105	94.7	-0.04030	5.3
5	0.00350	-0.01744	100.0	0.00091	94.7	-0.04276	5.3
6	0.00350	-0.01713	100.0	0.00095	94.7	-0.04208	5.3
7	0.00350	-0.01492	100.0	0.00122	94.7	-0.03719	5.3
8	0.00350	-0.02385	0.0	0.00012	5.3	-0.05692	94.7
9	0.00350	-0.02375	0.0	0.00013	5.3	-0.05671	94.7
10	0.00350	-0.01495	100.0	0.00122	94.7	-0.03728	5.3

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	116 di 155

11	0.00350	-0.01516	100.0	0.00119	94.7	-0.03772	5.3
12	0.00350	-0.01492	100.0	0.00122	94.7	-0.03719	5.3

**SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.71	100.0	0.00	70.9	-24.1	94.7	13.3	1325	31.9	17.9
2	OK	0.75	100.0	0.00	79.4	-40.2	94.7	13.3	1325	31.9	17.9
3	OK	0.48	0.0	0.00	18.2	-30.1	5.3	13.3	1325	21.2	11.2
4	OK	2.12	100.0	0.00	75.4	-90.7	94.7	13.3	1325	31.9	17.9
5	OK	1.02	100.0	0.00	78.2	-51.0	94.7	13.3	1325	31.9	17.9
6	OK	1.52	100.0	0.00	76.7	-69.7	94.7	13.3	1325	31.9	17.9

**SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; = (e1 + e2)/(2*e1) for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm] cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio As eff/Ac eff [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00013	0	0.500	26 4.0	0.024	0.000072 (0.00007)	320	0.023(0.20)	571.04
2	OK	-0.00022	0	0.500	26 4.0	0.024	0.000121 (0.00012)	320	0.039(0.20)	486.02
3	OK	-0.00016	0	0.500	26 4.0	0.016	0.000090 (0.00009)	412	0.037(0.20)	-494.28
4	OK	-0.00049	0	0.500	26 4.0	0.024	0.000272 (0.00027)	320	0.087(0.20)	524.36
5	OK	-0.00027	0	0.500	26 4.0	0.024	0.000153 (0.00015)	320	0.049(0.20)	497.47
6	OK	-0.00037	0	0.500	26 4.0	0.024	0.000209 (0.00021)	320	0.067(0.20)	511.97

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.63	100.0	0.00	70.6	-21.0	94.7	13.3	1325	31.9	17.9
2	OK	0.62	100.0	0.00	80.3	-35.3	94.7	13.3	1325	31.9	17.9
3	OK	0.90	0.0	0.00	17.5	-59.5	5.3	13.3	1325	21.2	11.2
4	OK	2.38	100.0	0.00	75.7	-103.3	94.7	13.3	1325	31.9	17.9
5	OK	1.60	100.0	0.00	76.7	-73.4	94.7	13.3	1325	31.9	17.9
6	OK	2.38	100.0	0.00	75.7	-103.3	94.7	13.3	1325	31.9	17.9

### SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00011	0	0.500	26 4.0	0.024	0.000063 (0.00006)	320	0.020 (0.20)	573.72
2	OK	-0.00019	0	0.500	26 4.0	0.024	0.000106 (0.00011)	320	0.034 (0.20)	477.58
3	OK	-0.00032	0	0.500	26 4.0	0.016	0.000179 (0.00018)	412	0.074 (0.20)	-486.79
4	OK	-0.00056	0	0.500	26 4.0	0.024	0.000310 (0.00031)	320	0.099 (0.20)	521.86
5	OK	-0.00039	0	0.500	26 4.0	0.024	0.000220 (0.00022)	320	0.070 (0.20)	511.61
6	OK	-0.00056	0	0.500	26 4.0	0.024	0.000310 (0.00031)	320	0.099 (0.20)	521.86

### SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	0.63	100.0	0.00	69.2	-19.6	94.7	13.3	1325	31.9	17.9
2	OK	1.13	100.0	0.00	74.3	-45.6	94.7	13.3	1325	31.9	17.9
3	OK	0.38	100.0	0.00	66.7	-10.6	94.7	13.3	1325	31.9	17.9
4	OK	1.38	100.0	0.00	73.9	-54.6	94.7	13.3	1325	31.9	17.9
5	OK	1.35	100.0	0.00	74.3	-54.6	94.7	13.3	1325	31.9	17.9
6	OK	1.38	100.0	0.00	73.9	-54.6	94.7	13.3	1325	31.9	17.9

### SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00011	0	0.500	26 4.0	0.024	0.000059 (0.00006)	320	0.019 (0.20)	589.31
2	OK	-0.00025	0	0.500	26 4.0	0.024	0.000137 (0.00014)	320	0.044 (0.20)	535.51
3	OK	-0.00006	0	0.500	26 4.0	0.024	0.000032 (0.00003)	320	0.010 (0.20)	611.18
4	OK	-0.00029	0	0.500	26 4.0	0.024	0.000164 (0.00016)	320	0.052 (0.20)	539.77
5	OK	-0.00029	0	0.500	26 4.0	0.024	0.000164 (0.00016)	320	0.052 (0.20)	535.55
6	OK	-0.00029	0	0.500	26 4.0	0.024	0.000164 (0.00016)	320	0.052 (0.20)	539.77

### SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Frc	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
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**RELAZIONE DI CALCOLO**

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1	Frequent	Web	0.65	0.36	4400	337.55	3.38	1.50	---	31.9	5.9
2	Frequent	Web	0.65	0.44	5350	-164.83	-1.65	0.67	---	31.9	8.7
3	Frequent	Web	0.65	0.43	5200	-108.62	-1.09	0.67	---	21.2	8.2
4	Frequent	Web	0.65	0.39	4850	66.53	0.67	1.50	---	31.9	7.1
5	Frequent	Web	0.65	0.40	4950	12.98	0.13	1.50	---	31.9	7.3
6	Frequent	Web	0.65	0.39	4850	66.53	0.67	1.50	---	31.9	7.1
1	Quasi-perm.	Web	0.65	0.36	4300	419.01	4.19	1.50	---	31.9	5.7
2	Quasi-perm.	Web	0.65	0.39	4750	137.87	1.38	1.50	---	31.9	6.8
3	Quasi-perm.	Web	0.65	0.34	4150	533.29	5.33	1.50	---	31.9	5.3
4	Quasi-perm.	Web	0.65	0.38	4700	160.11	1.60	1.50	---	31.9	6.7
5	Quasi-perm.	Web	0.65	0.39	4750	138.11	1.38	1.50	---	31.9	6.8
6	Quasi-perm.	Web	0.65	0.38	4700	160.11	1.60	1.50	---	31.9	6.7

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

$b_w = 1000$ mm	larghezza	$f_{yk} = 450$ MPa	resist. caratteristica
$h = 1000$ mm	altezza	$\gamma_s = 1.15$	coeff. sicurezza
$c = 40$ mm	copriferro	$f_{yd} = 391.3$ MPa	resist. di calcolo
$f_{ck} = 25$ MPa	resist. caratteristica	Armatura longitudinale tesa:	
$\gamma_c = 1.50$	coeff. sicurezza	$A_{sl,1} = 6 \text{ } \emptyset 26$	$= 31.86 \text{ cm}^2$
$\alpha_{cc} = 0.85$	coeff. riduttivo	$A_{sl,2} = 0 \text{ } \emptyset 0$	$= 0.00 \text{ cm}^2$
$d = 960$ mm	altezza utile	$A_{sl,3} = 0 \text{ } \emptyset 0$	$= 0.00 \text{ cm}^2$
$f_{cd} = 14.17$ MPa	resist. di calcolo		$31.86 \text{ cm}^2$

**• Sollecitazioni** (compressione<0, trazione>0, taglio in valore assoluto)

$N_{ed} = 0.0$ kN	$V_{ed} = 560.0$ kN
-------------------	---------------------

**• Elementi senza armature trasversali resistenti a taglio**

$k = 1 + (200/d)^{1/2} < 2$	$k = 1.456$	$< 2$
$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$	$v_{min} = 0.308$	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 = 0.003$	$< 0.02$
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} = 0.00$ MPa	$< 0.2 f_{cd}$

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / \gamma_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d \quad (\text{con } (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 295.3 \text{ kN})$$

$$V_{Rd} = 339.6 \text{ kN}; \quad \text{valore di calcolo}$$

**la sezione NON è verificata in assenza di armature per il taglio**

**• Elementi con armature trasversali resistenti a taglio**

$\theta = 22.0$ °	inclinaz. bielle cls	angolo ammissibile
$\alpha = 90.0$ °	inclinaz. staffe	

Armatura a taglio (staffatura):

$A_{sw}/s$ staffe	$= \emptyset 12$ mm con n° bracci (trasv)	$2$	passo	$30$ cm	$= 0.075 \text{ cm}^2/\text{cm}$
$V_{Rsd}$	$= 630.9$ kN				

$$V_{Rsd} = 0.90 \times d \times (A_{sw}/s) \times f_{yd} \times (\cotg \alpha + \cotg \theta) \times \text{sen} \alpha$$

$$f_{cd} = 7.08 \text{ MPa} \text{ resist. di calcolo ridotta}$$

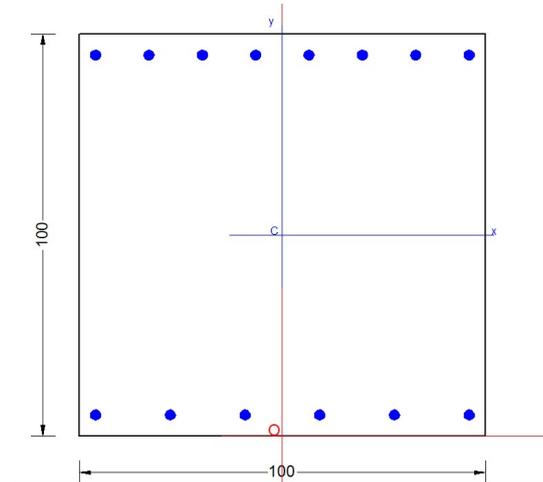
$$\alpha_c = 1.000 \text{ coeff. maggiorativo}$$

$$V_{Rcd} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cotg \alpha + \cotg \theta) / (1 + \cotg^2 \alpha)$$

$$V_{Rd} = \min(V_{Rcd}, V_{Rsd}) \quad V_{Rd} = 630.9 > 560.0 \text{ kN} \quad \text{c.s.} = 1.1$$

la sezione armata a taglio risulta verificata.

10.10 S7 Soletta di fondazione – Mezzeria



Sezione

**Carpenteria**

b	h	Acls
mm	mm	m <sup>2</sup>
1000	1000	1.00

**Copriferro**

lato	c [mm]
esterno	40
interno	40

**Materiali**

Barre acciaio		Calcestruzzo	
Tipo	B450C	Tipo	C25/30
$f_{yk}$	450.0 N/mm <sup>2</sup>	$f_{ck}$	25.0 N/mm <sup>2</sup>
$\gamma_M$	1.15 -	$R_{ck}$	30.0 N/mm <sup>2</sup>
$f_{yd}$	391.3 N/mm <sup>2</sup>	rara	
rara		0.6 $f_{ck}$	15.0 N/mm <sup>2</sup>
0.8 $f_{yk}$	360.0 N/mm <sup>2</sup>	qperm	
		0.45 $f_{ck}$	11.25 N/mm <sup>2</sup>

**Armature**

Lato superiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
8	26	4250	53	
Aa,tot =			4250	0.43

Lato inferiore				
n	$\phi$	Aa	c'	Aa/Acls
-	mm	mm <sup>2</sup>	mm	%
6	26	3190	53	
Aa,tot =			3190	0.40

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<b>RELAZIONE DI CALCOLO</b>	COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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## GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S7.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\Rampa\RC SEC Checks\S7\Soletta superiore S7.secEC)

Section description:  
 Reference code: EN 1992-1-1 [EC2] Italian Annex  
 Exposure Class: XC4 - Carbonation (cyclic wet and dry)  
 Structural member of belonging: Beam (or slab) without stirrup [§6.2.1(4) EC2]  
 Section type: Predefined section  
 Cross-section shape: Rectangular  
 Stress path: Constant axial force to achieve bending ULS  
 Reference of assigned forces: Principal axes x,y of inertia

## MATERIALS DATA

CONCRETE -

Class:	C25/30
Design compressive strength fcd:	14.2 MPa
Shear reduced compressive strength $v1*fcd$ :	7.1 MPa (6.9)EC2
Coeff. Crd.c:	0.1200 § 6.2.2(1) EC2
Coeff. K1:	0.1500 § 6.2.2(1) EC2
Coeff. v min:	0.0102 Mpa (§ 6.2.2(1) EC2)
Strain at max strength ec2:	0.0020
Ultimate strain ecu:	0.0035
Compression diagram stress-strain:	Parabola-Rectangle
Mean elastic modulus Ecm:	31475.8 MPa
Mean tensile strength fctm:	2.6 MPa
Es/Ec in SLS combination:	15.00
Stress limit Characteristic SLS:	15.0 MPa
Stress limit Frequent SLS:	15.0 MPa
wk lim =limit crack width Frequent SLS:	0.300 mm
Coeff. k3:	3.4000 § 7.3.4(3) EC2
Coeff. k4:	0.4250 § 7.3.4(3) EC2
Stress limit Quasi-Permanent SLS:	11.3 MPa
wk lim =limit crack-width Quasi-Permanent SLS:	0.200 mm

STEEL -

Longitudinal Bars:	B450C
Characteristic yield stress fyk:	450.0 MPa
Tensile strength ftk:	540.0 MPa
Design yield stress fyd:	391.3 MPa
Design strength ftd:	391.3 MPa
Design ultimate strain esu:	0.068
Mean elastic modulus Es:	200000.0 MPa
Stirrups:	B500A
Characteristic yield stress fyk:	500.0 MPa
Tensile strength ftk:	540.00 MPa
Design yield stress fyd:	434.8 MPa
Design strength ftd:	434.8 MPa
Mean elastic modulus Es:	200000.0 MPa
SLS Charact. - Ss Limit:	360.00 MPa

## GEOMETRIC AND REINFORCEMENT DATA

Width:	100.0	cm
Depth:	100.0	cm
Bottom bars:	6L26 + 5L10	(31.9 cm?)
Top bars:	8L26 + 5L10	(42.5 cm?)
Low Cover (from centroid of bar):	5.3	cm
Top Cover (from centroid of bar):	5.3	cm



 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b>  <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA <b>NR1J</b>	LOTTO <b>01 D 29</b>	CODIFICA <b>CL</b>	DOCUMENTO <b>IV0100 001</b>	REV. <b>A</b>

Web Cover (from centroid of bar): 4.0 cm

#### ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION

N Design axial force [kN] applied in the concrete centroidal point (+ if compressive)  
 Mx Design moment force [kNm] around the x axis  
 with positive direction if such as to compress the upper edge of the section  
 Vy Design shear [kN] in a direction parallel to the axis Y  
 T Design torsional Moment [kN m]

Comb.N.	N	Mx	Vy	MT
1	-85.16	-326.45	32.86	0.00
2	108.22	-582.17	34.27	0.00
3	4.84	-818.28	53.74	0.00
4	32.25	-818.28	-39.14	0.00
5	38.05	-680.73	-41.62	0.00
6	-67.14	-544.34	119.01	0.00
7	-207.12	-231.38	158.69	0.00
8	125.47	-270.10	-130.60	0.00
9	101.18	-338.34	131.53	0.00
10	-202.95	8.39	151.09	0.00
11	-182.84	-163.93	-160.25	0.00
12	-202.95	-122.05	159.49	0.00

#### SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
 Mx Moment force [kNm] around x axis  
 with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	-45.65	-396.61
2	33.43	-508.96
3	2.82	-597.79
4	21.85	-597.79
5	26.69	-483.16
6	7.65	-483.16

#### SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
 Mx Moment force [kNm] around x axis  
 with positive direction if such as to compress the upper edge of the section

Comb.N.	N	Mx
1	-41.93	-398.25
2	36.99	-388.85
3	-10.63	-536.98
4	0.81	-536.98
5	8.87	-345.93
6	-30.09	-498.32

#### SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION

N Axial force [kN] applied in the concrete centroidal (+ if compressive)  
 Mx Moment force [kNm] around x axis  
 with positive direction if such as to compress the upper edge of the section

**RELAZIONE DI CALCOLO**

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Comb.N.	N	Mx
1	-51.35	-207.24
2	-28.45	-154.71
3	-51.35	-207.24
4	-28.45	-142.97
5	-28.45	-154.71
6	-51.35	-207.24

**CHECKS RESULTS**

Min edge cover of longitudinal bars: 2.7 cm  
Min edge distance between longitudinal bars: 5.3 cm

**ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE**

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is >=1.0
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	-85.16	-326.45	-85.19	-1466.30	4.551	7.7	-1403.60	---	---	---
2	OK	108.22	-582.17	108.06	-1551.66	2.662	8.3	-1476.71	---	---	---
3	OK	4.84	-818.28	4.96	-1506.16	1.841	8.0	-1437.76	---	---	---
4	OK	32.25	-818.28	32.07	-1518.14	1.855	8.1	-1448.05	---	---	---
5	OK	38.05	-680.73	38.21	-1520.86	2.233	8.1	-1450.34	---	---	---
6	OK	-67.14	-544.34	-67.19	-1474.29	2.722	7.8	-1410.45	---	---	---
7	OK	-207.12	-231.38	-207.05	-1412.23	6.413	7.3	-1356.69	---	---	---
8	OK	125.47	-270.10	125.34	-1559.25	5.749	8.4	-1483.13	---	---	---
9	OK	101.18	-338.34	101.42	-1548.74	4.566	8.3	-1473.97	---	---	---
10	OK	-202.95	8.39	-202.88	1042.99	49.455	94.0	1011.88	---	---	---
11	OK	-182.84	-163.93	-183.01	-1422.90	9.269	7.4	-1366.02	---	---	---
12	OK	-202.95	-122.05	-202.88	-1414.08	12.844	7.3	-1358.28	---	---	---

**ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE**

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.01599	0.0	0.00109	5.3	-0.03956	94.7

**RELAZIONE DI CALCOLO**

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2	0.00350	-0.01448	0.0	0.00128	5.3	-0.03623	94.7
3	0.00350	-0.01528	0.0	0.00118	5.3	-0.03800	94.7
4	0.00350	-0.01507	0.0	0.00120	5.3	-0.03752	94.7
5	0.00350	-0.01502	0.0	0.00121	5.3	-0.03742	94.7
6	0.00350	-0.01584	0.0	0.00111	5.3	-0.03923	94.7
7	0.00350	-0.01699	0.0	0.00097	5.3	-0.04177	94.7
8	0.00350	-0.01435	0.0	0.00129	5.3	-0.03595	94.7
9	0.00350	-0.01453	0.0	0.00127	5.3	-0.03633	94.7
10	0.00350	-0.02144	100.0	0.00042	94.7	-0.05161	5.3
11	0.00350	-0.01679	0.0	0.00099	5.3	-0.04134	94.7
12	0.00350	-0.01695	0.0	0.00097	5.3	-0.04170	94.7

**SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing > 5(c+O/2) and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	2.67	0.0	0.00	24.9	-112.4	5.3	13.3	1325	42.5	7.5
2	OK	3.52	0.0	0.00	26.7	-134.6	5.3	13.3	1325	42.5	7.5
3	OK	4.10	0.0	0.00	26.1	-161.9	5.3	13.3	1325	42.5	7.5
4	OK	4.12	0.0	0.00	26.4	-159.9	5.3	13.3	1325	42.5	7.5
5	OK	3.34	0.0	0.00	26.6	-128.3	5.3	13.3	1325	42.5	7.5
6	OK	3.32	0.0	0.00	26.2	-130.3	5.3	13.3	1325	42.5	7.5

**SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; $=(e1 + e2)/(2*e1)$ for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
W   c	Equivalent bar diameter [mm] cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio $As\ eff / Ac\ eff$ [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = 0.6 Smax / Es
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = sr max*(e sm - e cm)[eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00060	0	0.500	26 4.0	0.032	0.000337 (0.00034)	274	0.092(0.20)	-537.07
2	OK	-0.00073	0	0.500	26 4.0	0.032	0.000404 (0.00040)	274	0.111(0.20)	-555.36
3	OK	-0.00087	0	0.500	26 4.0	0.032	0.000520 (0.00049)	274	0.143(0.20)	-549.05

**RELAZIONE DI CALCOLO**

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4	OK	-0.00086	0	0.500	26 4.0	0.032	0.000510 (0.00048)	274	0.140(0.20)	-552.33
5	OK	-0.00069	0	0.500	26 4.0	0.032	0.000385 (0.00038)	274	0.105(0.20)	-554.27
6	OK	-0.00070	0	0.500	26 4.0	0.032	0.000391 (0.00039)	274	0.107(0.20)	-550.19

**SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	2.68	0.0	0.00	25.0	-112.5	5.3	13.3	1325	42.5	7.5
2	OK	2.70	0.0	0.00	27.0	-101.6	5.3	13.3	1325	42.5	7.5
3	OK	3.67	0.0	0.00	25.8	-146.8	5.3	13.3	1325	42.5	7.5
4	OK	3.68	0.0	0.00	26.0	-145.6	5.3	13.3	1325	42.5	7.5
5	OK	2.38	0.0	0.00	26.3	-92.9	5.3	13.3	1325	42.5	7.5
6	OK	3.38	0.0	0.00	25.4	-138.4	5.3	13.3	1325	42.5	7.5

**SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00061	0	0.500	26 4.0	0.032	0.000337 (0.00034)	274	0.092 (0.20)	-538.04
2	OK	-0.00055	0	0.500	26 4.0	0.032	0.000305 (0.00030)	274	0.083 (0.20)	-558.45
3	OK	-0.00079	0	0.500	26 4.0	0.032	0.000445 (0.00044)	274	0.122 (0.20)	-546.56
4	OK	-0.00078	0	0.500	26 4.0	0.032	0.000439 (0.00044)	274	0.120 (0.20)	-548.72
5	OK	-0.00050	0	0.500	26 4.0	0.032	0.000279 (0.00028)	274	0.076 (0.20)	-551.20
6	OK	-0.00074	0	0.500	26 4.0	0.032	0.000415 (0.00042)	274	0.114 (0.20)	-542.48

**SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]**

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	1.36	0.0	0.00	23.6	-61.7	5.3	13.3	1325	42.5	7.5
2	OK	1.04	0.0	0.00	24.5	-45.0	5.3	13.3	1325	42.5	7.5
3	OK	1.36	0.0	0.00	23.6	-61.7	5.3	13.3	1325	42.5	7.5
4	OK	0.96	0.0	0.00	24.3	-41.8	5.3	13.3	1325	42.5	7.5
5	OK	1.04	0.0	0.00	24.5	-45.0	5.3	13.3	1325	42.5	7.5
6	OK	1.36	0.0	0.00	23.6	-61.7	5.3	13.3	1325	42.5	7.5

**SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]**

Comb.N.	Check	e1	e2	k2	W   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00033	0	0.500	26 4.0	0.032	0.000185 (0.00019)	274	0.051 (0.20)	-524.40
2	OK	-0.00024	0	0.500	26 4.0	0.032	0.000135 (0.00013)	274	0.037 (0.20)	-530.43
3	OK	-0.00033	0	0.500	26 4.0	0.032	0.000185 (0.00019)	274	0.051 (0.20)	-524.40
4	OK	-0.00022	0	0.500	26 4.0	0.032	0.000125 (0.00013)	274	0.034 (0.20)	-528.99
5	OK	-0.00024	0	0.500	26 4.0	0.032	0.000135 (0.00013)	274	0.037 (0.20)	-530.43
6	OK	-0.00033	0	0.500	26 4.0	0.032	0.000185 (0.00019)	274	0.051 (0.20)	-524.40

**SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL [§ 7.3.2 EC2]**

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]

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k1 Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]  
 As reg Area [cm?] of reinforcing steel within the tensile zone of region under consideration  
 As min Minimum Area [cm?] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	0.65	0.40	5050	-18.33	-0.18	0.67	---	42.5	7.6
2	Frequent	Web	0.65	0.39	4850	81.71	0.82	1.50	---	42.5	7.0
3	Frequent	Web	0.65	0.40	4950	23.43	0.23	1.50	---	42.5	7.3
4	Frequent	Web	0.65	0.40	4950	34.04	0.34	1.50	---	42.5	7.3
5	Frequent	Web	0.65	0.40	4900	46.16	0.46	1.50	---	42.5	7.2
6	Frequent	Web	0.65	0.40	5000	3.44	0.03	0.67	---	42.5	7.4
1	Quasi-perm.	Web	0.65	0.42	5150	-85.14	-0.85	0.67	---	42.5	8.0
2	Quasi-perm.	Web	0.65	0.41	5100	-55.60	-0.56	0.67	---	42.5	7.8
3	Quasi-perm.	Web	0.65	0.42	5150	-85.14	-0.85	0.67	---	42.5	8.0
4	Quasi-perm.	Web	0.65	0.41	5150	-62.64	-0.63	0.67	---	42.5	7.9
5	Quasi-perm.	Web	0.65	0.41	5100	-55.60	-0.56	0.67	---	42.5	7.8
6	Quasi-perm.	Web	0.65	0.42	5150	-85.14	-0.85	0.67	---	42.5	8.0

**Verifica a taglio**

**VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5**

**• Caratteristiche della sezione**

b <sub>w</sub> = 1000	mm	larghezza	f <sub>yk</sub> = 450	MPa	resist. caratteristica
h = 1000	mm	altezza	γ <sub>s</sub> = 1.15		coeff. sicurezza
c = 40	mm	copriferro	f <sub>yd</sub> = 391.3	MPa	resist. di calcolo
f <sub>ck</sub> = 25	MPa	resist. caratteristica	Armatura longitudinale tesa:		
γ <sub>c</sub> = 1.50		coeff. sicurezza	A <sub>sl,1</sub> = 8	Ø 26	= 42.47 cm <sup>2</sup>
α <sub>cc</sub> = 0.85		coeff. riduttivo	A <sub>sl,2</sub> = 0	Ø 0	= 0.00 cm <sup>2</sup>
d = 960	mm	altezza utile	A <sub>sl,3</sub> = 0	Ø 0	= 0.00 cm <sup>2</sup>
f <sub>cd</sub> = 14.17	MPa	resist. di calcolo			42.47 cm <sup>2</sup>

**• Sollecitazioni** (compressione<0, trazione>0, taglio in valore assoluto)

N <sub>ed</sub> = 0.0	kN	V <sub>ed</sub> = 160.0	kN
-----------------------	----	-------------------------	----

**• Elementi senza armature trasversali resistenti a taglio**

k = 1+(200/d) <sup>1/2</sup> <2	k = 1.456	<2
v <sub>min</sub> = 0,035 k <sup>3/2</sup> f <sub>ck</sub> <sup>1/2</sup>	v <sub>min</sub> = 0.308	
ρ <sub>1</sub> = A <sub>sl</sub> /(b <sub>w</sub> ×d) <0.02	ρ <sub>1</sub> = 0.004	<0.02
σ <sub>cp</sub> = N <sub>Ed</sub> /A <sub>c</sub> <0.2f <sub>cd</sub>	σ <sub>cp</sub> = 0.00	<0.2

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ck})^{1/3} / g_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

V<sub>Rd</sub> = 373.8 kN; (con (v<sub>min</sub>+0,15×σ<sub>cp</sub>)×b<sub>w</sub>×d = 295.3 kN)

V<sub>Rd</sub> = 373.8 kN valore di calcolo

la sezione è verificata in assenza di armature per il taglio

**• Elementi con armature trasversali resistenti a taglio**

θ = 22.0	°	inclinaz. bielle cls	angolo ammissibile
α = 90.0	°	inclinaz. staffe	

Armatura a taglio (staffatura):

A <sub>sw</sub> /s = staffe Ø 12	mm con n° bracci (trasv)	2	passo	30	cm	=	0.075	cm <sup>2</sup> /cm
----------------------------------	--------------------------	---	-------	----	----	---	-------	---------------------

$$V_{Rsd} = 0.90 \times d \times (A_{sw}/s) \times f_{yd} \times (\cot \alpha + \cot \theta) \times \sin \alpha$$

f <sub>cd</sub> = 7.08	MPa	resist. di calcolo ridotta
α <sub>c</sub> = 1.000		coeff. maggiorativo

$$V_{Rsd} = 630.9 \text{ kN}$$

$$V_{Red} = 0.90 \times d \times b_w \times \alpha_c \times f_{cd} \times (\cot \alpha + \cot \theta) / (1 + \cot^2 \alpha)$$

V <sub>Rd</sub> = min(V <sub>Red</sub> , V <sub>Rsd</sub> )	V <sub>Rd</sub> = 630.9	>	160.0	kN	c.s.=	3.9
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la sezione armata a taglio risulta verificata.

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

### 10.11 Sbalzo soletta

Per la verifica della soletta a sbalzo è stata considerata una sezione 1.75m x 0.30m.

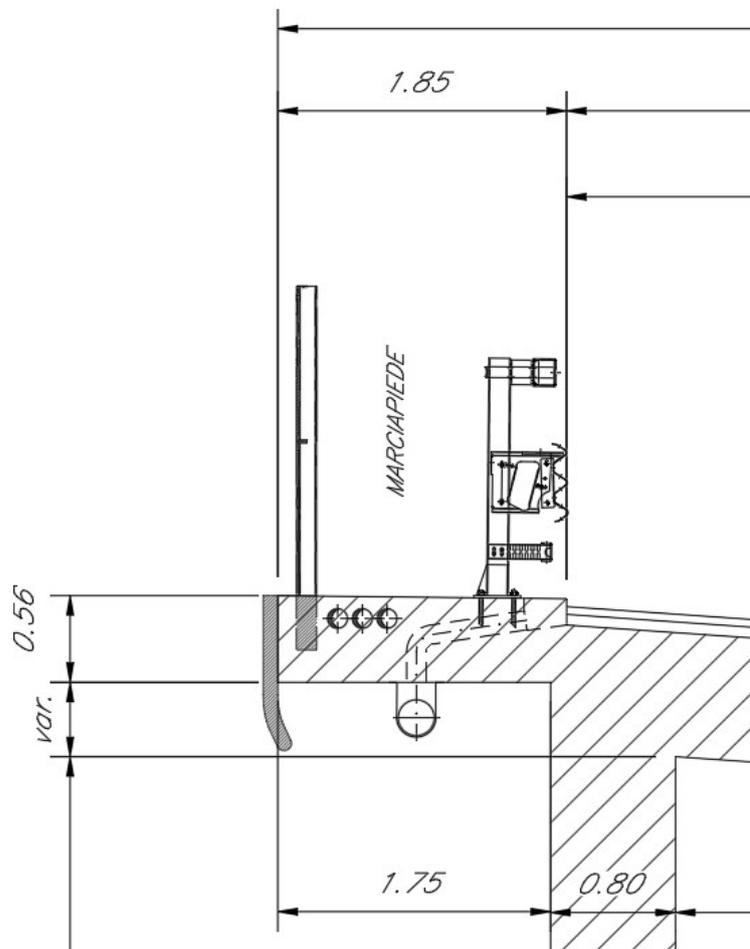


Fig. 43 – Sbalzo soletta

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

All'elemento frame d'estremità sono state applicati i seguenti carichi nominali.

Frame	Station	OutputCase	CaseType	P	V2	V3	T	M2	M3
Text	m	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
21	1.75	DEAD	LinStatic	0.0	26.25	0.0	0.0	0.0	-23.00
21	1.75	G2 parapetto	LinStatic	0.0	1.0	0.0	0.0	0.0	-1.75
21	1.75	G2 sicurvvia	LinStatic	0.0	0.0	0.0	0.0	0.0	0.0
21	1.75	G2 veletta	LinStatic	0.0	2.5	0.0	0.0	0.0	-4.4
21	1.75	Q_tr S5_folla	LinStatic	0.0	8.75	0.0	0.0	0.0	-7.7
21	1.75	Q_tr S4_ruota	LinStatic	0.0	10.0	0.0	0.0	0.0	-17.5
21	1.75	Sisma SLV_Z	LinStatic	0.0	4.6	0.0	0.0	0.0	-4.01
21	1.75	Svio	LinStatic	-50	0.0	0.0	0.0	0.0	-75

E combinati con i seguenti coefficienti si ottengono le sollecitazioni di verifica.

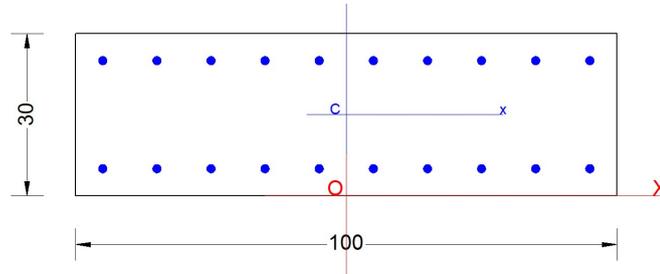
SLU_1	SLU_2	SLE	SLV_1	SVIO
1.35	1.35	1.00	1.00	1.00
1.50	1.50	1.00	1.00	1.00
1.50	1.50	1.00	1.00	1.00
1.50	1.50	1.00	1.00	1.00
1.50	0.00	1.00	0.00	0.00
0.00	1.50	0.00	0.00	0.00
0.00	0.00	0.00	1.00	0.00
0.00	0.00	0.00	0.00	1.00

## Verifiche

TABLE: Element Forces - Frames									
Frame	Station	OutputCase	CaseType	P	V2	V3	T	M2	M3
Text	m	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
21	1.75	SLU_1	Combination	0	53.82	0	0	0	-51.83
21	1.75	SLU_2	Combination	0	55.69	0	0	0	-66.53
21	1.75	SLE	Combination	0	38.5	0	0	0	-36.85
21	1.75	SLV_1	Combination	0	34.35	0	0	0	-33.16
21	1.75	SVIO	Combination	-50	29.75	0	0	0	-104.75

 <b>ITALFERR</b> GRUPPO FERROVIE DELLO STATO ITALIANE	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

La sbalzo è armato con  $\Phi 16/10$



## GENERAL DATA OF PREDEFINED RC CROSS-SECTION

Section Name: Soletta superiore S10\_16\_10.secEC

(File path: S:\Projects\18005\_0549D07\_Cesano Vigna di Valle\Work\01 Cavalcaferrovia\IV01\_km 29+500\04\_Rampa\RC SEC Checks\Sbalzo soletta\Soletta superiore S10\_16\_10.secEC)

Section description:	
Reference code:	EN 1992-1-1 [EC2] Italian Annex
Exposure Class:	XC4 - Carbonation (cyclic wet and dry)
Structural member of belonging:	Beam (or slab) without stirrup [§6.2.1(4) EC2]
Section type:	Predefined section
Cross-section shape:	Rectangular
Stress path:	Constant axial force to achieve bending ULS
Reference of assigned forces:	Principal axes x,y of inertia

## MATERIALS DATA

CONCRETE -	Class:	C32/40
	Design compressive strength fcd:	18.1 MPa
	Shear reduced compressive strength $v1*fcd$ :	9.1 MPa (6.9)EC2
	Coeff. Crd.c:	0.1200 § 6.2.2(1) EC2
	Coeff. K1:	0.1500 § 6.2.2(1) EC2
	Coeff. v min:	0.0115 Mpa (§ 6.2.2(1) EC2)
	Strain at max strength ec2:	0.0020
	Ultimate strain ecu:	0.0035
	Compression diagram stress-strain:	Parabola-Rectangle
	Mean elastic modulus Ecm:	33345.8 MPa
	Mean tensile strength fctm:	3.0 MPa
	Es/Ec in SLS combination:	15.00
	Stress limit Characteristic SLS:	19.2 MPa
	Stress limit Frequent SLS:	19.2 MPa
	wk lim =limit crack width Frequent SLS:	0.300 mm
	Coeff. k3:	3.4000 § 7.3.4(3) EC2
	Coeff. k4:	0.4250 § 7.3.4(3) EC2
	Stress limit Quasi-Permanent SLS:	14.4 MPa
	wk lim =limit crack-width Quasi-Permanent SLS:	0.200 mm
STEEL -	Longitudinal Bars:	B450C
	Characteristic yield stress fyk:	450.0 MPa
	Tensile strength ftk:	540.0 MPa
	Design yield stress fyd:	391.3 MPa
	Design strength ftd:	391.3 MPa
	Design ultimate strain esu:	0.068
	Mean elastic modulus Es:	200000.0 MPa
	Stirrups:	B500A



**RELAZIONE DI CALCOLO**

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Characteristic yield stress $f_{yk}$ :	500.0	MPa
Tensile strength $f_{tk}$ :	540.00	MPa
Design yield stress $f_{yd}$ :	434.8	MPa
Design strength $f_{td}$ :	434.8	MPa
Mean elastic modulus $E_s$ :	200000.0	MPa
SLS Charact. - $S_s$ Limit:	360.00	MPa

**GEOMETRIC AND REINFORCEMENT DATA**

Width:	100.0	cm
Depth:	30.0	cm
Bottom bars:	10 $\bar{\text{U}}$ 16	(20.1 cm?)
Top bars:	10 $\bar{\text{U}}$ 16	(20.1 cm?)
Low Cover (from centroid of bar):	5.0	cm
Top Cover (from centroid of bar):	5.0	cm
Web Cover (from centroid of bar):	5.0	cm

**ULTIMATE LIMIT STATE - DESIGN INTERNAL FORCES ASSIGNED FOR EACH COMBINATION**

N	Design axial force [kN] applied in the concrete centroidal point (+ if compressive)
M <sub>x</sub>	Design moment force [kNm] around the x axis with positive direction if such as to compress the upper edge of the section
V <sub>y</sub>	Design shear [kN] in a direction parallel to the axis Y
T	Design torsional Moment [kN m]

Comb.N.	N	M <sub>x</sub>	V <sub>y</sub>	MT
1	0.00	-51.83	53.82	0.00
2	0.00	-66.53	55.69	0.00
3	0.00	-33.16	34.35	0.00
4	50.00	-104.75	29.75	0.00
5	0.00	-104.75	29.75	0.00

**SERVICEABILITY LIMIT STATE - CHARACTERISTIC COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N	Axial force [kN] applied in the concrete centroidal (+ if compressive)
M <sub>x</sub>	Moment force [kNm] around x axis with positive direction if such as to compress the upper edge of the section

Comb.N.	N	M <sub>x</sub>
1	0.00	-36.85

**SERVICEABILITY LIMIT STATE - FREQUENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N	Axial force [kN] applied in the concrete centroidal (+ if compressive)
M <sub>x</sub>	Moment force [kNm] around x axis with positive direction if such as to compress the upper edge of the section

Comb.N.	N	M <sub>x</sub>
1	0.00	-36.85

**SERVICEABILITY LIMIT STATE - QUASI-PERMANENT COMB. - ASSIGNED INTERNAL FORCES FOR EACH COMBINATION**

N	Axial force [kN] applied in the concrete centroidal (+ if compressive)
M <sub>x</sub>	Moment force [kNm] around x axis with positive direction if such as to compress the upper edge of the section

Comb.N.	N	M <sub>x</sub>
---------	---	----------------

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1                    0.00                    -36.85

**CHECKS RESULTS**

**Checks of section OK for all assigned combinations**

Min edge cover of longitudinal bars:                    4.2 cm  
Min edge distance between longitudinal bars:                    8.4 cm  
Min edge cover of stirrups:                    4.2 cm

**ULTIMATE LIMIT STATE - CHECKS FOR BENDING AND AXIAL FORCE**

Check	Result of check
N	Design axial force applied in the concrete centroid [kN] (+ if compressive)
Mx	Design bending moment force [kNm] around x axis of reference
N ult	Axial capacity [kN] of section (+ if compressive)
Mx ult	Bending moment capacity [kNm] around x axis of reference
S.F.	Safety factor = vectorial ratio of (N ult, Mx ult) to (N, Mx). Check OK if such ratio is $\geq 1.0$
Yneutral	Ordinate Y [cm] of neutral axis in the reference system
Mx sn.	Yielding bending moment [kNm]
x/d	Ratio of the depth of neutral axis to the effective depth of the section
D	Ratio of redistributed moment to the elastic moment in continuous beams [(5.10)EC2]
As Tension	Area bars in tension zone [cm <sup>2</sup> ] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

As Tension Area bars in tension zone [cm<sup>2</sup>] (beam). [Between brackets is shown the min. area from eq.(9.1N) EC2]

1	OK	0.00	-51.83	0.05	-178.92	3.452	5.1	-170.30	---	---	---
2	OK	0.00	-66.53	0.05	-178.92	2.689	5.1	-170.30	---	---	---
3	OK	0.00	-33.16	0.05	-178.92	5.396	5.1	-170.30	---	---	---
4	OK	50.00	-104.75	50.04	-184.05	1.757	5.3	-175.47	---	---	---
5	OK	0.00	-104.75	0.05	-178.92	1.708	5.1	-170.30	---	---	---

**ULTIMATE LIMIT STATE - STRAINS AT ULTIMATE STATE**

ec max	Ultimate compressive strain in concrete
ec*	Strain in the concrete fiber at ec2/ecu of depth (if ec*>0 then the section is all compressed)
Yc max	Ordinate in [cm] of the fiber corresponding to ec max
es max	Max strain in steel bars (+ if compressive)
Ys max	Ordinate [cm] of the bar corresponding to es max
es min	Min strain in steel bars (+ if compressive)
Ys min	Ordinate [cm] of the bar corresponding to es min

Comb.N.	ec max	ec *	Yc max	es max	Ys max	es min	Ys min
1	0.00350	-0.00528	0.0	0.00009	5.0	-0.01357	25.0
2	0.00350	-0.00528	0.0	0.00009	5.0	-0.01357	25.0
3	0.00350	-0.00528	0.0	0.00009	5.0	-0.01357	25.0
4	0.00350	-0.00507	0.0	0.00017	5.0	-0.01315	25.0
5	0.00350	-0.00528	0.0	0.00009	5.0	-0.01357	25.0

**ULTIMATE LIMIT STATE - SHEAR CHECKS WITH NO TRANSVERSE REINFORCEMENT (§ 6.2.2 EC2)**

Check	Result of check
Ved	Design value [daN] of applied shear force Vy (uniaxial shear)
Vwct	Design shear resistance [kN] without shear reinforcement [eq.(6.2)EC2]

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d	Effective depth of section [cm]
bw	Effective min width of the cross-section [cm]
Ro	Geometrical ratio of longitudinal tension longitudinal reinforcement = $Asl/(bw d)[<0.02]$
Scp	Mean compression stress (NEd/Ac)[Mpa] [ $< 0.2 fcd$ ]
k	Coeff. k in the equation (6.2.a) EC2

Comb.N.	Check	Ved	Vwct	d	bw	Ro	Scp	k
1	OK	53.82	167.80	25.0	100.0	0.0000	0.00	1.894
2	OK	55.69	167.80	25.0	100.0	0.0000	0.00	1.894
3	OK	34.35	167.80	25.0	100.0	0.0000	0.00	1.894
4	OK	29.75	174.05	25.0	100.0	0.0000	0.02	1.894
5	OK	29.75	167.80	25.0	100.0	0.0000	0.00	1.894

#### SLS CHARACTERISTIC COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Check	Result of Check
Sc max	Max compressive stress (+) in concrete [Mpa]
Yc max	y-coordinate [cm] of concrete fiber corresponding to Sc max (reference X,Y,O)
Sc min	Min compression stress (+) in concrete [Mpa]
Yc min	y-coordinate [cm] of concrete fiber corresponding to Sc min (reference X,Y,O)
Ss min	Min tension stress (-) of steel bars [Mpa]
Ys min	y-coordinate [cm] of centroid bar corresponding to Ss min (reference X,Y,O)
hc eff.	Concrete depth [cm] in concrete tension area surrounding tension steel bars
Ac eff	Effective tension area [cm <sup>2</sup> ] that is the area of concrete surrounding the tension reinforcement (for crack width control)
As eff	Area of tension steel bars [cm <sup>2</sup> ] within Ac eff (for crack width control)
D bars	Mean spacing [cm] of bonded reinforcement [eq.(7.11) §7.3.4(3)EC2] (D bars = 0 indicates spacing $> 5(c+O/2)$ and eq.(7.14) is pertinent)

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	3.03	0.0	0.00	8.7	-84.8	5.0	7.1	710	20.1	10.0

#### SLS CHARACTERISTIC COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Check	Result of check
e1	Greater concrete tensile strain (tension is -) within the entire section assessed in cracked section
e2	Lesser concrete tensile strain (tension is -) within the entire section assessed in cracked section
k1	= 0.8 high bond bars assigned [see eq.(7.11) EC2]
kt	= 0.6 for charact. and frequent SLS; = 0.4 for quasi-perm. SLS (Factor dependent on the duration of the load [see eq.(7.9) EC2])
k2	= 0.5 for bending; $=(e1 + e2)/(2*e1)$ for eccentric tension [see eq.(7.13) EC2]
k3	= 3.400 Coeff. in eq. (7.11) EC2 according to national annex
k4	= 0.425 Coeff. in eq. (7.11) EC2 according to national annex
Ш   c	Equivalent bar diameter [mm]   cover [cm] to the tensile reinforcement [see eq.(7.11)(7.12) EC2]
r eff	Geometrical ratio $As\ eff/Ac\ eff$ [eq.(7.10) EC2]
e sm - e cm	Difference between the mean strain of tensile steel and concrete under the relevant combination [eq.(7.8)] Between brackets: Minimum value of eq.(7.9) = $0.6 Smax / Es$
sr max	Max final crack [mm] spacing
wk	Calculated value [mm] of crack width = $sr\ max*(e\ sm - e\ cm)$ [eq.(7.8)]. Limit value of wk is shown between brackets
M crack	First cracking bending moment [kNm]

Comb.N.	Check	e1	e2	k2	Ш   c	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00055	0	0.500	16 4.2	0.028	0.000254 (0.00025)	239	0.061(0.20)	-57.52

#### SLS FREQUENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	3.03	0.0	0.00	8.7	-84.8	5.0	7.1	710	20.1	10.0

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### SLS FREQUENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	$\Delta   c$	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00055	0	0.500	16 4.2	0.028	0.000254 (0.00025)	239	0.061 (0.20)	-57.52

### SLS QUASI-PERMANENT COMBINATIONS - STRESS LIMITATION [§ 7.2 EC2]

Comb.N.	Check	Sc max	Yc max	Sc min	Yc min	Ss min	Ys min	hc eff	Ac eff	As Eff	D bars
1	OK	3.03	0.0	0.00	8.7	-84.8	5.0	7.1	710	20.1	10.0

### SLS QUASI-PERMANENT COMBINATIONS - CRACK WIDTH [§ 7.3.4 EC2]

Comb.N.	Check	e1	e2	k2	$\Delta   c$	r eff	e sm - e cm	sr max	wk	M crack
1	OK	-0.00055	0	0.500	16 4.2	0.028	0.000254 (0.00025)	239	0.061 (0.20)	-57.52

### SLS - MINIMUM REINFORCEMENT AREAS FOR CRACK CONTROL (§ 7.3.2 EC2)

N.Comb.	SLS combination number
Comb.Type	Frequent or Quasi-Permanent
Region	Part of concrete section under consideration (Web/Flange)
k	Coeff. wich take account of self-equilibrating stresses [see eq.(7.1) EC2]
kc	Coeff. wich take account of stress distribution at cracking[see eq.(7.1) EC2]
Ac eff	Area of concrete within tensile zone of the region under consideration just before cracking [(7.1)]
Ned	SLS axial force (+ if compressive) acting on the region under consideration [kN]
Sc	=Ned/Ac mean stress in the region of area Ac (web or rectangular section) [eq.(7.1)]
Fcr	Absolute value of the tensile force [kN] within the flange at cracking due to the cracking moment [eq.(7.3)]
k1	Coeff. considering the effect of axial forces on the stress (web or rectangular section) [eq.(7.2)]
As reg	Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration
As min	Minimum Area [cm <sup>2</sup> ] of reinforcing steel within the tensile zone of region under consideration required by eq. (7.1)

Comb.N.	Comb.Type	Region	k	kc	Ac eff	Ned	Sc	k1	Frc	As reg	As min
1	Frequent	Web	1.00	0.40	1500	---	---	---	-226.79	20.1	4.0
1	Quasi-perm.	Web	1.00	0.40	1500	---	---	---	-226.79	20.1	4.0

### VERIFICA A TAGLIO DELLA SEZIONE IN C.A. SECONDO D.M. 17/01/2018 § 4.1.2.3.5

#### • Caratteristiche della sezione

$b_w =$	1000	mm	larghezza	$f_{yk} =$	450	MPa	resist. caratteristica
$h =$	300	mm	altezza	$\gamma_s =$	1.15		coeff. sicurezza
$c =$	40	mm	copriferro	$f_{yd} =$	391.3	MPa	resist. di calcolo
$f_{ck} =$	32	MPa	resist. caratteristica	Armatura longitudinale tesa:			
$\gamma_c =$	1.50		coeff. sicurezza	$A_{sl,1} =$	10	Ø	16 = 12.57 cm <sup>2</sup>
$\alpha_{cc} =$	0.85		coeff. riduttivo	$A_{sl,2} =$	0	Ø	0 = 0.00 cm <sup>2</sup>
$d =$	260	mm	altezza utile	$A_{sl,3} =$	0	Ø	0 = 0.00 cm <sup>2</sup>
$f_{cd} =$	18.13	MPa	resist. di calcolo				12.57 cm <sup>2</sup>

#### • Sollecitazioni (compressione<0, trazione>0, taglio in valore assoluto)

$N_{ed} =$	0.0	kN	$V_{ed} =$	56.0	kN
------------	-----	----	------------	------	----

#### • Elementi senza armature trasversali resistenti a taglio

$k = 1 + (200/d)^{1/2} < 2$	$k =$	1.877	< 2
$v_{min} = 0,035 k^{3/2} f_{ck}^{1/2}$	$v_{min} =$	0.509	
$\rho_1 = A_{sl}/(b_w \times d) < 0.02$	$\rho_1 =$	0.005	< 0.02
$\sigma_{cp} = N_{Ed}/A_c < 0.2 f_{cd}$	$\sigma_{cp} =$	0.00	MPa < 0.2

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fcd

$$V_{Rd} = (0,18 \times k \times (100 \times \rho_1 \times f_{ctk})^{1/3} / g_c + 0,15 \times \sigma_{cp}) \times b_w \times d > (v_{\min} + 0,15 \times \sigma_{cp}) \times b_w \times d$$

$$V_{Rd} = 170.7 \text{ kN}; \quad (\text{con } (v_{\min} + 0,15 \times \sigma_{cp}) \times b_w \times d = 132.4 \text{ kN})$$

$$V_{Rd} = 170.7 \text{ kN} \quad \text{valore di calcolo}$$

la sezione è verificata in assenza di armature per il taglio

## 10.12 Capacità portante

I carichi considerati nella verifica di capacità portante sono stimati dalla tabella sottostante a partire dall'involuppo delle combinazioni SLU/SLV.

OutputCase	CaseType Text	GlobalFX KN	GlobalFY KN	GlobalFZ KN	GlobalMX KN-m	GlobalMY KN-m	GlobalMZ KN-m	GlobalX m	GlobalY m	GlobalZ m
SLV_5	Combination	308.304	0	808.296	0	1980.6486	0	0	0	0
SLV_6	Combination	308.304	0	763.87	0	1980.6486	0	0	0	0
SLU_GR_2B...	Combination	-10.314	0	1393.941	0	844.5473	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1895.453	0	779.7797	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1893.703	0	779.7797	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1893.703	0	779.7797	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1893.703	0	779.7797	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1893.703	0	779.7797	0	0	0	0
SLV_4	Combination	92.491	0	860.125	0	594.1848	0	0	0	0
SLV_8	Combination	92.491	0	712.041	0	594.1848	0	0	0	0
SVID_1	Combination	50	0	786.083	0	539.986	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1663.733	0	489.4192	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1663.733	0	489.4192	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1663.733	0	489.4192	0	0	0	0
SLU_GR_1_...	Combination	-27.405	0	1663.733	0	489.4192	0	0	0	0

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$$q_{lim} = c \cdot N_c \cdot s_c \cdot d_c \cdot i_c \cdot b_c \cdot g_c + q \cdot N_q \cdot s_q \cdot d_q \cdot i_q \cdot b_q \cdot g_q + 0,5 \cdot \gamma \cdot B^* \cdot N_{\gamma} \cdot s_{\gamma} \cdot d_{\gamma} \cdot i_{\gamma} \cdot b_{\gamma} \cdot g_{\gamma}$$

D = Profondità del piano di appoggio

$e_B$  = Eccentricità in direzione B ( $e_B = Mb/N$ )

$e_L$  = Eccentricità in direzione L ( $e_L = MI/N$ ) (per fondazione nastriforme  $e_L = 0$ ;  $L^* = L$ )

$B^*$  = Larghezza fittizia della fondazione ( $B^* = B - 2 \cdot e_B$ )

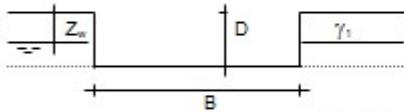
$L^*$  = Lunghezza fittizia della fondazione ( $L^* = L - 2 \cdot e_L$ )

(per fondazione nastriforme le sollecitazioni agenti sono riferite all'unità di lunghezza)

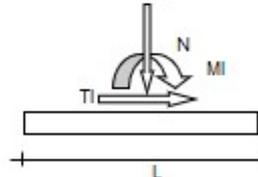
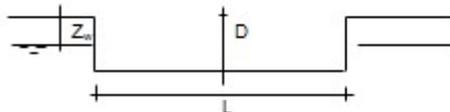
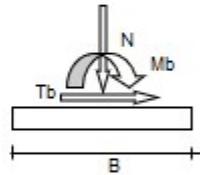
coefficienti parziali

Metodo di calcolo		azioni		proprietà del terreno	
		permanenti	temporanee variabili	$\tan \varphi'$	$c'$
Stato limite ultimo	○	1.00	1.30	1.25	1.60
Tensioni ammissibili	○	1.00	1.00	1.00	1.00
definiti dall'utente	⊗	1.00	1.00	1.25	1.60

valori suggeriti dall'EC7

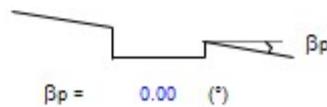
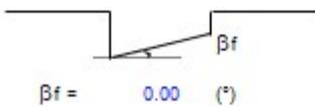


$\gamma, c', \varphi'$



(Per fondazione nastriforme L = 100 m)

B = 1.00 (m)  
L = 12.00 (m)  
D = 2.40 (m)



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	AZIONI		Valori di calcolo
	valori di input		
	permanenti	temporanee	
N [kN]	809.00	0.00	809.00
Mb [kNm]	0.00	0.00	0.00
Ml [kNm]	1981.00	0.00	1981.00
Tb [kN]	0.00	0.00	0.00
Tl [kN]	0.00	309.00	309.00
H [kN]	0.00	309.00	309.00

*Peso unità di volume del terreno*

$\gamma_1 = 15.00$  (kN/mc)  
 $\gamma = 18.00$  (kN/mc)

*Valori caratteristici di resistenza del terreno*

$c' = 10.00$  (kN/mq)  
 $\varphi' = 28.00$  (°)

*Valori di progetto*

$c' = 8.25$  (kN/mq)  
 $\varphi' = 23.04$  (°)

*Profondità della falda*

$Z_w = 17.00$  (m)

$e_u = 0.00$  (m)  
 $e_c = 2.45$  (m)

$B^* = 1.00$  (m)  
 $L^* = 7.10$  (m)

**q** : sovraccarico alla profondità D

$q = 38.00$  (kN/mq)

**$\gamma$**  : peso di volume del terreno di fondazione

$\gamma = 18.00$  (kN/mc)

**$N_c, N_q, N\gamma$**  : coefficienti di capacità portante

$N_q = \tan^2(45 + \varphi'/2) \cdot e^{(\pi \cdot \tan \varphi' \cdot c' / \gamma)}$

$N_q = 8.70$

$N_c = (N_q - 1) / \tan \varphi'$

$N_c = 18.10$

$N\gamma = 2 \cdot (N_q + 1) \cdot \tan \varphi'$

$N\gamma = 8.25$

**$s_c, s_q, s_\gamma$**  : fattori di forma

$s_c = 1 + B \cdot N_q / (L^* \cdot N_c)$

$s_c = 1.07$

$s_q = 1 + B \cdot \tan \varphi' / L^*$

$s_q = 1.08$

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$$s_y = 1 - 0,4 \cdot B^* / L^*$$

$$s_y = 0,94$$

$i_c, i_q, i_r$  : fattori di inclinazione del carico

$$m_b = (2 + B^* / L^*) / (1 + B^* / L^*) = 1,88 \quad \theta = \arctg(T_b/T_l) = 0,00 \quad (^\circ)$$

$$m_l = (2 + L^* / B^*) / (1 + L^* / B^*) = 1,12 \quad m = 1,12 \quad (-)$$

( $m=2$  nel caso di fondazione nastroforme e  $m=(m_b \sin^2 \theta + m_l \cos^2 \theta)$  in tutti gli altri)

$$i_c = (1 - H / (N + B^* L^* c' \cotg \varphi))^m$$

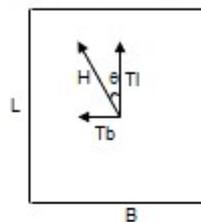
$$i_c = 0,63$$

$$i_q = i_c - (1 - i_c) / (N_q - 1)$$

$$i_q = 0,58$$

$$i_r = (1 - H / (N + B^* L^* c' \cotg \varphi))^{(m+1)}$$

$$i_r = 0,42$$



$d_c, d_q, d_r$  : fattori di profondità del piano di appoggio

$$\text{per } D/B^* \leq 1; d_c = 1 + 2 D \tan \varphi' (1 - \sin \varphi)^2 / B^*$$

$$\text{per } D/B^* > 1; d_c = 1 + (2 \tan \varphi' (1 - \sin \varphi)^2) \cdot \arctan (D / B^*)$$

$$d_c = 1,37$$

$$d_q = d_c - (1 - d_c) / (N_c \tan \varphi)$$

$$d_q = 1,42$$

$$d_r = 1$$

$$d_r = 1,00$$

$b_c, b_q, b_r$  : fattori di inclinazione base della fondazione

$$b_c = (1 - \beta_l \tan \varphi)^2 \quad \beta_l + \beta_p = 0,00 \quad \beta_l + \beta_p < 45^\circ$$

$$b_c = 1,00$$

$$b_q = b_c - (1 - b_c) / (N_c \tan \varphi)$$

$$b_q = 1,00$$

$$b_r = b_c$$

$$b_r = 1,00$$





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$g_1, g_2, g_3$  : fattori di inclinazione piano di campagna

$$g_1 = (1 - \tan\beta_p)^2 \quad \beta_r + \beta_p = 0.00 \quad \beta_r + \beta_p < 45^\circ$$

$$g_1 = 1.00$$

$$g_2 = g_1 - (1 - g_1) / (N_c \tan\varphi)$$

$$g_2 = 1.00$$

$$g_3 = g_1$$

$$g_3 = 1.00$$

Carico limite unitario

$$q_{lim} = 414.73 \quad (\text{kN/m}^2) \quad R3 \quad 2.30$$

$$q_{rd} = 180 \quad (\text{kN/m}^2)$$

Pressione massima agente

$$q = N / B \cdot L^*$$

$$q = 113.90 \quad (\text{kN/m}^2)$$

Coefficiente di sicurezza

$$F_s = q_{lim} / q = 3.64 \quad \text{OK}$$

VERIFICA A SCORRIMENTO

$$H_d = 309.00 \quad (\text{kN})$$

$$S_d = N \cdot \tan(\varphi) + c' \cdot B \cdot L^*$$

$$S_d = 388.51 \quad (\text{kN})$$

Coefficiente di sicurezza allo scorrimento

$$F_{scorr} = 1.26 \quad \text{OK}$$

### 10.13 Armatura longitudinale

Per l'armatura longitudinale è stato adottato un valore minimo pari al 20% dell'armatura trasversale.

	<b>RADDOPPIO CESANO VIGNA DI VALLE</b> <b>PROGETTO DEFINITIVO</b>					
	<b>RELAZIONE DI CALCOLO</b>	COMMESSA NR1J	LOTTO 01 D 29	CODIFICA CL	DOCUMENTO IV0100 001	REV. A

#### 10.14 Incidenza armatura

I valori delle incidenze di armatura lenta sono indicati nella seguente tabella:

- Soletta = **170 kg/m<sup>3</sup>**;
- Sbalzo soletta = **150 kg/m<sup>3</sup>**;
- Piedritti = **120 kg/m<sup>3</sup>** ;
- Fondazione = **100 kg/m<sup>3</sup>**.

Come previsto dall' Eurocodice (UNI EN 1992-1-1) per le piastre a portanza unidirezionale si raccomanda di prevedere un'armatura secondaria in quantità non minore del 20% dell'armatura principale.

Pertanto nel calcolo è stato considerato un incremento del 15% per tener conto della presenza di legature, spille e sovrapposizioni.

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<b>Soletta</b>					<b>Piedritti</b>				
H <sub>sez1</sub>	0.8 m				H <sub>sez1</sub>	0.8 m			
B <sub>sez</sub>	1 m				B <sub>sez</sub>	1 m			
L <sub>sez1</sub>	9 m				L <sub>sez1</sub>	10.2 m			
Vol. per m di muro	7.2 m <sup>3</sup>				Vol. per m di muro	8.16 m <sup>3</sup>			
<b>Sbalzo soletta</b>					<b>Fondazione</b>				
H <sub>sez</sub>	0.5 m				H <sub>sez</sub>	1 m			
B <sub>sez</sub>	1 m				B <sub>sez</sub>	1 m			
L <sub>sez</sub>	1.85 m				L <sub>sez</sub>	12 m			
Vol. per m di muro	0.925 m <sup>3</sup>				Vol. per m di muro	12 m <sup>3</sup>			
Peso acciaio	7850 kg/m <sup>3</sup>				Peso acciaio	7850 kg/m <sup>3</sup>			
Idennità di legature	0.15				Idennità di legature	0.15			
<b>Ferri di soletta (per m)</b>					<b>Ferri di piedritti (per m)</b>				
	φ	A	n° barre	peso		φ	A	n° barre	peso
	mm	m <sup>2</sup>	-	kg/m		mm	m <sup>2</sup>	-	kg/m
Zona tesa	30	0.000707	10	499.395	Zona tesa	22	0.00038	10	268.564
Zona compressa	22	0.00038	10	268.564	Zona compressa	22	0.00038	10	268.564
Zona tesa, arm. orizzontale	12	0.000113	45	39.952	Zona tesa, arm. orizzontale	12	0.000113	58	51.493
Zona compressa, arm. Orizzontale	12	0.000113	45	39.952	Zona compressa, arm. Orizzontale	12	0.000113	58	51.493
Bracci	12	0.000113	72	64	Bracci	12	0.000113	74	66
Incidenza paramento	145.64 kg/m <sup>3</sup>				Incidenza paramento	99.51 kg/m <sup>3</sup>			
<b>Ferri di sbalzo soletta (per m)</b>					<b>Ferri di fondazione (per m)</b>				
	φ	A	n° barre	peso		φ	A	n° barre	peso
	mm	m <sup>2</sup>	-	kg/m		mm	m <sup>2</sup>	-	kg/m
Zona tesa	16	0.000201	10	29.199	Zona tesa	26	0.000531	8	400.108
Zona compressa	16	0.000201	10	29.199	Zona compressa	26	0.000531	6	300.081
Zona tesa, arm. dir. long.	12	0.000113	10	8.878	Zona tesa, arm. dir. long.	12	0.000113	62	55.044
Zona compressa, arm. dir. long.	12	0.000113	10	8.878	Zona compressa, arm. dir. long.	12	0.000113	62	55.044
Bracci					Bracci	12	0.000113	70	63
Incidenza Fondazione	94.68 kg/m <sup>3</sup>				Incidenza Fondazione	83.69 kg/m <sup>3</sup>			















### 11.3 ALLEGATO 1 – Combinazioni SLV e Svio

CaseName	SLV_1	SLV_2	SLV_3	SLV_4	SLV_5	SLV_6	SLV_7	SLV_8	Svio_1
DEAD	1	1	1	1	1	1	1	1	1
G2 impianti	1	1	1	1	1	1	1	1	1
G2 parapetto	1	1	1	1	1	1	1	1	1
G2 pavimentazione	1	1	1	1	1	1	1	1	1
G2 sicurvia	1	1	1	1	1	1	1	1	1
G2 sopraelevazioni	1	1	1	1	1	1	1	1	1
G2 veletta	1	1	1	1	1	1	1	1	1
G3 Falda H	1	1	1	1	1	1	1	1	1
G3 Falda V	1	1	1	1	1	1	1	1	1
G3 Terra Dx	1	1	1	1	1	1	1	1	1
G3 Terra Sx	1	1	1	1	1	1	1	1	1
Q_inc	0	0	0	0	0	0	0	0	0
Avv_frenatura	1	1	1	1	1	1	1	1	1
Q_ritiro	0	0	0	0	0	0	0	0	1
Q_svio	0,5	0	0	0,5	0	0,5	0	0	0,5
Q_temp farfalla	0	0,5	0	0	0,5	0	0,5	0	0
Q_temp farfalla -	0,5	0	0	0	0,5	0,5	0	0	0,5
Q_temp unif	0	0,5	0	0,5	0	0	0,5	0	0
Q_temp unif -	0	0	0	0	0	0	0	0	0
Q_tr	0	0	0	0	0	0	0	0	0
Avv_frenatura	0	0	0	0	0	0	0	0	0
Q_tr Fcentr	0	0	0	0	0	0	0	0	0
Q_tr S1_zona rim	0	0	0	0	0	0	0	0	0
Q_tr S1_zona rim_INC	0	0	0	0	0	0	0	0	0
Q_tr S1-C1 distr	0	0	0	0	0	0	0	0	0
Q_tr S1-C1 distr_INC	0	0	0	0	0	0	0	0	0
Q_tr S1- C1 ruota	0	0	0	0	0	0	0	0	0
Q_tr S1- C1 ruota_INC	0	0	0	0	0	0	0	0	0
Q_tr S1-C2 distr	0	0	0	0	0	0	0	0	0
Q_tr S1-C2 distr_INC	0	0	0	0	0	0	0	0	0
Q_tr S1- C2 ruota	0	0	0	0	0	0	0	0	0
Q_tr S1- C2 ruota_INC	0	0	0	0	0	0	0	0	0
Q_tr S1-C3 distr	0	0	0	0	0	0	0	0	0
Q_tr S1-C3 distr_INC	0	0	0	0	0	0	0	0	0
Q_tr S1- C3 ruota	0	0	0	0	0	0	0	0	0
Q_tr S1- C3 ruota_INC	0	0	0	0	0	0	0	0	0
Q_tr S2_ruota	0	0	0	0	0	0	0	0	0
Q_tr S2_ruota_INC	0	0	0	0	0	0	0	0	0
Q_tr S4_ruota	0	0	0	0	0	0	0	0	0
Q_tr S5_folla	0	0	0	0	0	0	0	0	0
Q_vento parete sopravento	0	0	0	0	0	0	0	0	0
Q_vento parete sottovento	0	0	0	0	0	0	0	0	0
Q_vento soletta	0	0	0	0	0	0	0	0	0
Q_vento veicoli	0	0	0	0	0	0	0	0	0
Sisma SLD	0	0	0	0	0	0	0	0	0
Sisma SLD_Z	0	0	0	0	0	0	0	0	0
Sisma SLV	1	1	0,3	-0,3	-1	-1	0,3	-0,3	0
Sisma SLV_Z	0,3	-0,3	1	1	0,3	-0,3	-1	-1	0

#### 11.4 ALLEGATO 1 – Combinazioni involuppo

CaseName	Inviluppo	INVSISMA	INVSLEFR2A	INVSLEFR2B	INVSLEQP	INVSLERR
SLEFR_Gr_2A_S1_1			1			
SLEFR_Gr_2A_S1_10			1			
SLEFR_Gr_2A_S1_11			1			
SLEFR_Gr_2A_S1_12			1			
SLEFR_Gr_2A_S1_13			1			
SLEFR_Gr_2A_S1_14			1			
SLEFR_Gr_2A_S1_15			1			
SLEFR_Gr_2A_S1_16			1			
SLEFR_Gr_2A_S1_17			1			
SLEFR_Gr_2A_S1_18			1			
SLEFR_Gr_2A_S1_19			1			
SLEFR_Gr_2A_S1_2			1			
SLEFR_Gr_2A_S1_20			1			
SLEFR_Gr_2A_S1_21			1			
SLEFR_Gr_2A_S1_22			1			
SLEFR_Gr_2A_S1_23			1			
SLEFR_Gr_2A_S1_24			1			
SLEFR_Gr_2A_S1_25			1			
SLEFR_Gr_2A_S1_26			1			
SLEFR_Gr_2A_S1_27			1			
SLEFR_Gr_2A_S1_28			1			
SLEFR_Gr_2A_S1_29			1			
SLEFR_Gr_2A_S1_3			1			
SLEFR_Gr_2A_S1_30			1			
SLEFR_Gr_2A_S1_31			1			
SLEFR_Gr_2A_S1_32			1			
SLEFR_Gr_2A_S1_33			1			
SLEFR_Gr_2A_S1_34			1			
SLEFR_Gr_2A_S1_35			1			
SLEFR_Gr_2A_S1_36			1			
SLEFR_Gr_2A_S1_37			1			
SLEFR_Gr_2A_S1_38			1			
SLEFR_Gr_2A_S1_39			1			
SLEFR_Gr_2A_S1_4			1			
SLEFR_Gr_2A_S1_40			1			
SLEFR_Gr_2A_S1_5			1			
SLEFR_Gr_2A_S1_6			1			
SLEFR_Gr_2A_S1_7			1			

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SLEFR_Gr_2A_S1_8			1		
SLEFR_Gr_2A_S1_9			1		
SLEFR_Gr_2B_S1_1				1	
SLEFR_Gr_2B_S1_10				1	
SLEFR_Gr_2B_S1_11				1	
SLEFR_Gr_2B_S1_12				1	
SLEFR_Gr_2B_S1_13				1	
SLEFR_Gr_2B_S1_14				1	
SLEFR_Gr_2B_S1_15				1	
SLEFR_Gr_2B_S1_16				1	
SLEFR_Gr_2B_S1_17				1	
SLEFR_Gr_2B_S1_18				1	
SLEFR_Gr_2B_S1_19				1	
SLEFR_Gr_2B_S1_2				1	
SLEFR_Gr_2B_S1_20				1	
SLEFR_Gr_2B_S1_21				1	
SLEFR_Gr_2B_S1_22				1	
SLEFR_Gr_2B_S1_23				1	
SLEFR_Gr_2B_S1_24				1	
SLEFR_Gr_2B_S1_25				1	
SLEFR_Gr_2B_S1_26				1	
SLEFR_Gr_2B_S1_27				1	
SLEFR_Gr_2B_S1_28				1	
SLEFR_Gr_2B_S1_29				1	
SLEFR_Gr_2B_S1_3				1	
SLEFR_Gr_2B_S1_30				1	
SLEFR_Gr_2B_S1_31				1	
SLEFR_Gr_2B_S1_32				1	
SLEFR_Gr_2B_S1_33				1	
SLEFR_Gr_2B_S1_34				1	
SLEFR_Gr_2B_S1_35				1	
SLEFR_Gr_2B_S1_36				1	
SLEFR_Gr_2B_S1_37				1	
SLEFR_Gr_2B_S1_38				1	
SLEFR_Gr_2B_S1_39				1	
SLEFR_Gr_2B_S1_4				1	
SLEFR_Gr_2B_S1_40				1	
SLEFR_Gr_2B_S1_5				1	
SLEFR_Gr_2B_S1_6				1	
SLEFR_Gr_2B_S1_7				1	

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COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	149 di 155

SLEFR_Gr_2B_S1_8				1	
SLEFR_Gr_2B_S1_9				1	
SLEQP_Gr_1_S1_1					1
SLEQP_Gr_1_S1_10					1
SLEQP_Gr_1_S1_2					1
SLEQP_Gr_1_S1_3					1
SLEQP_Gr_1_S1_4					1
SLEQP_Gr_1_S1_5					1
SLEQP_Gr_1_S1_6					1
SLEQP_Gr_1_S1_7					1
SLEQP_Gr_1_S1_8					1
SLEQP_Gr_1_S1_9					1
SLERR_Gr_1_S1_1					1
SLERR_Gr_1_S1_10					1
SLERR_Gr_1_S1_11					1
SLERR_Gr_1_S1_12					1
SLERR_Gr_1_S1_13					1
SLERR_Gr_1_S1_14					1
SLERR_Gr_1_S1_15					1
SLERR_Gr_1_S1_16					1
SLERR_Gr_1_S1_17					1
SLERR_Gr_1_S1_18					1
SLERR_Gr_1_S1_19					1
SLERR_Gr_1_S1_2					1
SLERR_Gr_1_S1_20					1
SLERR_Gr_1_S1_21					1
SLERR_Gr_1_S1_22					1
SLERR_Gr_1_S1_23					1
SLERR_Gr_1_S1_24					1
SLERR_Gr_1_S1_25					1
SLERR_Gr_1_S1_26					1
SLERR_Gr_1_S1_27					1
SLERR_Gr_1_S1_28					1
SLERR_Gr_1_S1_29					1
SLERR_Gr_1_S1_3					1
SLERR_Gr_1_S1_30					1
SLERR_Gr_1_S1_31					1
SLERR_Gr_1_S1_32					1
SLERR_Gr_1_S1_33					1
SLERR_Gr_1_S1_34					1

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COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	150 di 155

SLERR_Gr_1_S1_35					1
SLERR_Gr_1_S1_36					1
SLERR_Gr_1_S1_37					1
SLERR_Gr_1_S1_38					1
SLERR_Gr_1_S1_39					1
SLERR_Gr_1_S1_4					1
SLERR_Gr_1_S1_40					1
SLERR_Gr_1_S1_5					1
SLERR_Gr_1_S1_6					1
SLERR_Gr_1_S1_7					1
SLERR_Gr_1_S1_8					1
SLERR_Gr_1_S1_9					1
SLU_Gr_1_S1_1	1				
SLU_Gr_1_S1_10	1				
SLU_Gr_1_S1_11	1				
SLU_Gr_1_S1_12	1				
SLU_Gr_1_S1_13	1				
SLU_Gr_1_S1_14	1				
SLU_Gr_1_S1_15	1				
SLU_Gr_1_S1_16	1				
SLU_Gr_1_S1_17	1				
SLU_Gr_1_S1_18	1				
SLU_Gr_1_S1_19	1				
SLU_Gr_1_S1_2	1				
SLU_Gr_1_S1_20	1				
SLU_Gr_1_S1_21	1				
SLU_Gr_1_S1_22	1				
SLU_Gr_1_S1_23	1				
SLU_Gr_1_S1_24	1				
SLU_Gr_1_S1_25	1				
SLU_Gr_1_S1_26	1				
SLU_Gr_1_S1_27	1				
SLU_Gr_1_S1_28	1				
SLU_Gr_1_S1_29	1				
SLU_Gr_1_S1_3	1				
SLU_Gr_1_S1_30	1				
SLU_Gr_1_S1_31	1				
SLU_Gr_1_S1_32	1				
SLU_Gr_1_S1_33	1				
SLU_Gr_1_S1_34	1				

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COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	151 di 155

SLU_Gr_1_S1_35	1				
SLU_Gr_1_S1_36	1				
SLU_Gr_1_S1_37	1				
SLU_Gr_1_S1_38	1				
SLU_Gr_1_S1_39	1				
SLU_Gr_1_S1_4	1				
SLU_Gr_1_S1_40	1				
SLU_Gr_1_S1_5	1				
SLU_Gr_1_S1_6	1				
SLU_Gr_1_S1_7	1				
SLU_Gr_1_S1_8	1				
SLU_Gr_1_S1_9	1				
SLU_Gr_1_S2_1	1				
SLU_Gr_1_S2_10	1				
SLU_Gr_1_S2_11	1				
SLU_Gr_1_S2_12	1				
SLU_Gr_1_S2_13	1				
SLU_Gr_1_S2_14	1				
SLU_Gr_1_S2_15	1				
SLU_Gr_1_S2_16	1				
SLU_Gr_1_S2_17	1				
SLU_Gr_1_S2_18	1				
SLU_Gr_1_S2_19	1				
SLU_Gr_1_S2_2	1				
SLU_Gr_1_S2_20	1				
SLU_Gr_1_S2_21	1				
SLU_Gr_1_S2_22	1				
SLU_Gr_1_S2_23	1				
SLU_Gr_1_S2_24	1				
SLU_Gr_1_S2_25	1				
SLU_Gr_1_S2_26	1				
SLU_Gr_1_S2_27	1				
SLU_Gr_1_S2_28	1				
SLU_Gr_1_S2_29	1				
SLU_Gr_1_S2_3	1				
SLU_Gr_1_S2_30	1				
SLU_Gr_1_S2_31	1				
SLU_Gr_1_S2_32	1				
SLU_Gr_1_S2_33	1				
SLU_Gr_1_S2_34	1				

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COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
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SLU_Gr_1_S2_35	1				
SLU_Gr_1_S2_36	1				
SLU_Gr_1_S2_37	1				
SLU_Gr_1_S2_38	1				
SLU_Gr_1_S2_39	1				
SLU_Gr_1_S2_4	1				
SLU_Gr_1_S2_40	1				
SLU_Gr_1_S2_5	1				
SLU_Gr_1_S2_6	1				
SLU_Gr_1_S2_7	1				
SLU_Gr_1_S2_8	1				
SLU_Gr_1_S2_9	1				
SLU_Gr_1_S4_1	1				
SLU_Gr_1_S4_10	1				
SLU_Gr_1_S4_11	1				
SLU_Gr_1_S4_12	1				
SLU_Gr_1_S4_13	1				
SLU_Gr_1_S4_14	1				
SLU_Gr_1_S4_15	1				
SLU_Gr_1_S4_16	1				
SLU_Gr_1_S4_17	1				
SLU_Gr_1_S4_18	1				
SLU_Gr_1_S4_19	1				
SLU_Gr_1_S4_2	1				
SLU_Gr_1_S4_20	1				
SLU_Gr_1_S4_3	1				
SLU_Gr_1_S4_4	1				
SLU_Gr_1_S4_5	1				
SLU_Gr_1_S4_6	1				
SLU_Gr_1_S4_7	1				
SLU_Gr_1_S4_8	1				
SLU_Gr_1_S4_9	1				
SLU_Gr_2A_S1_1	1				
SLU_Gr_2A_S1_10	1				
SLU_Gr_2A_S1_11	1				
SLU_Gr_2A_S1_12	1				
SLU_Gr_2A_S1_13	1				
SLU_Gr_2A_S1_14	1				
SLU_Gr_2A_S1_15	1				
SLU_Gr_2A_S1_16	1				



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COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	153 di 155

SLU_Gr_2A_S1_17	1				
SLU_Gr_2A_S1_18	1				
SLU_Gr_2A_S1_19	1				
SLU_Gr_2A_S1_2	1				
SLU_Gr_2A_S1_20	1				
SLU_Gr_2A_S1_21	1				
SLU_Gr_2A_S1_22	1				
SLU_Gr_2A_S1_23	1				
SLU_Gr_2A_S1_24	1				
SLU_Gr_2A_S1_25	1				
SLU_Gr_2A_S1_26	1				
SLU_Gr_2A_S1_27	1				
SLU_Gr_2A_S1_28	1				
SLU_Gr_2A_S1_29	1				
SLU_Gr_2A_S1_3	1				
SLU_Gr_2A_S1_30	1				
SLU_Gr_2A_S1_31	1				
SLU_Gr_2A_S1_32	1				
SLU_Gr_2A_S1_33	1				
SLU_Gr_2A_S1_34	1				
SLU_Gr_2A_S1_35	1				
SLU_Gr_2A_S1_36	1				
SLU_Gr_2A_S1_37	1				
SLU_Gr_2A_S1_38	1				
SLU_Gr_2A_S1_39	1				
SLU_Gr_2A_S1_4	1				
SLU_Gr_2A_S1_40	1				
SLU_Gr_2A_S1_5	1				
SLU_Gr_2A_S1_6	1				
SLU_Gr_2A_S1_7	1				
SLU_Gr_2A_S1_8	1				
SLU_Gr_2A_S1_9	1				
SLU_Gr_2B_S1_1	1				
SLU_Gr_2B_S1_10	1				
SLU_Gr_2B_S1_11	1				
SLU_Gr_2B_S1_12	1				
SLU_Gr_2B_S1_13	1				
SLU_Gr_2B_S1_14	1				
SLU_Gr_2B_S1_15	1				
SLU_Gr_2B_S1_16	1				

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	154 di 155

SLU_Gr_2B_S1_17	1				
SLU_Gr_2B_S1_18	1				
SLU_Gr_2B_S1_19	1				
SLU_Gr_2B_S1_2	1				
SLU_Gr_2B_S1_20	1				
SLU_Gr_2B_S1_21	1				
SLU_Gr_2B_S1_22	1				
SLU_Gr_2B_S1_23	1				
SLU_Gr_2B_S1_24	1				
SLU_Gr_2B_S1_25	1				
SLU_Gr_2B_S1_26	1				
SLU_Gr_2B_S1_27	1				
SLU_Gr_2B_S1_28	1				
SLU_Gr_2B_S1_29	1				
SLU_Gr_2B_S1_3	1				
SLU_Gr_2B_S1_30	1				
SLU_Gr_2B_S1_31	1				
SLU_Gr_2B_S1_32	1				
SLU_Gr_2B_S1_33	1				
SLU_Gr_2B_S1_34	1				
SLU_Gr_2B_S1_35	1				
SLU_Gr_2B_S1_36	1				
SLU_Gr_2B_S1_37	1				
SLU_Gr_2B_S1_38	1				
SLU_Gr_2B_S1_39	1				
SLU_Gr_2B_S1_4	1				
SLU_Gr_2B_S1_40	1				
SLU_Gr_2B_S1_5	1				
SLU_Gr_2B_S1_6	1				
SLU_Gr_2B_S1_7	1				
SLU_Gr_2B_S1_8	1				
SLU_Gr_2B_S1_9	1				
SLV_1		1			
SLV_2		1			
SLV_3		1			
SLV_4		1			
SLV_5		1			
SLV_6		1			
SLV_7		1			
SLV_8		1			



**RADDOPPIO CESANO VIGNA DI VALLE**  
**PROGETTO DEFINITIVO**

**RELAZIONE DI CALCOLO**

COMMESSA	LOTTO	CODIFICA	DOCUMENTO	REV.	FOGLIO
NR1J	01 D 29	CL	IV0100 001	A	155 di 155

Svio_1		1				
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