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A P P U N T I

STUDENTE: Sannipoli

MATERIA: Teoria e Progetto dei Ponti Eserc.

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Consulenza: wednesday 14.00-15.00

BRIDGE DESIGN

Italian design codes for bridges

EN 1991-2 September 2003
Norme tecniche per le costruzioni luglio-07

Glossary: *Road bridges*

carriageway → "carreggiata". Traffic lane is "corsia" ⇒ the group of traffic lanes is carriageway.
for application of sections 4 and 5, the part of the road surface, supported by a single structure (deck, pier, etc.), which includes all physical traffic lanes (*i.e.* as may be marked on the road surface), hard shoulders, hard strips and marker strips (see 4.2.3(1)) → "manto stradale"

central reservation → "spartitraffico"
area separating the physical traffic lanes of a dual-carriageway road. It generally includes a median strip (→ "striscia") and lateral hard strips separated from the median strip by safety barriers.

hard shoulder → "corsia di emergenza" (emergency traffic load) → "più esterna"
surfaced strip ("striscia asfaltata"), usually of one traffic lane width, adjacent to the outermost physical traffic lane, intended for use by vehicles in the event of difficulty or during obstruction of the physical traffic lanes

hard strip → "banchina". Portion of carriageway where there is no traffic and it's not hard shoulder
surfaced strip, usually less than or equal to 2 m wide, located alongside ("accanto a", "di fianco a") a physical traffic lane, and between this traffic lane and a safety barrier or vehicle parapet

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Glossary: *Road bridges - loads*

abnormal load → "carico eccezionale". In Italy we have 3 types of abnormal load:
vehicle load which may not be carried on a route ("lungo il percorso") without permission from the relevant authority (→ to be able to carry this abnormal load)
- the smallest: you ask road authority and then you can carry the load alone
- you ask permission to road authority, but the load is really heavy → you can travel only with the help of police cars
- like 2nd, but also part of traffic, or total traffic, is stopped (police cars) → the load is very very heavy ⇒

(without any help (like police cars))

notional lane — "corsie", but are only theorics. They are used only to bridge design. Traffic lanes are instead related to the traffic
strip of the carriageway, parallel to ~~an~~ edge of the carriageway, which in section 4 is deemed to carry a line of cars ~~and~~/or lorries

remaining area → it's a free space that remains
difference, where relevant, between the total area of the carriageway and the sum of the areas of the notional lanes (see Figure 4.1)

tandem system
assembly of two consecutive axles ("assali") considered to be simultaneously loaded
the car could be a tandem system

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BRIDGE DESIGN

Actions on road bridges

EN 1991-2 September 2003
Norme tecniche per le costruzioni luglio-07

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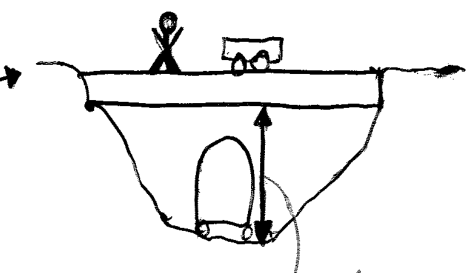
Clearance



- **5 m** Standard traffic
- **4 m** Restricted traffic
- **3.2 m** Only with Military and Fire Brigade permission

clearance: distance between the intrados of the bridge and the top of the pavement that is below the bridge

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Vertical traffic loads

- **Load Model 1 (LM1)**

  Tandem and distributed loads for general and local verification
- **Load Model 2 (LM2)**

  pneumatico

  Tandem or single tyre load for general and local verification
- **Load Model 3 (LM3)**

  Concentrated load for local verification (0.4x0.4m)
- **Load Model 4 (LM4)**

  Concentrated load for local verification (0.1x0.1m)
- **Load Model 5 (LM5)**

  "folla"

  Distributed Crowd load for general and local verification
- **Load Model 6 (LM6)** → it was invented for the "Messina Bridge" → never used

  Distributed loads for long span bridges

Road bridges category

$\alpha_Q Q_k$ **Concentrated loads** $\alpha_q q_k$ **Distributed loads**

Nowadays this distinction is not used anymore. It's only what we used in Italy.

(3) The values of adjustment factors α_{Qi}, α_{qi} and α_{qr} should be selected depending on the expected traffic and possibly on different classes of routes. In the absence of specification these factors should be taken equal to unity.

NOTE 1 The values of α_{Qi}, α_{qi} and α_{qr} factors are given in the National Annex. In all cases, for bridges without road signs restricting vehicle weights, the following minimum values are recommended :

$\alpha_{Q1} \geq 0,8$ and (4.3)

for : $i \geq 2$, $\alpha_{qi} \geq 1$; this restriction being not applicable to α_{qr}. (4.4)

NOTE 2 Values of α factors may correspond, in the National Annex, to classes of traffic. When they are taken equal to 1, they correspond to a traffic for which a heavy industrial international traffic is expected, representing a large part of the total traffic of heavy vehicles. For more common traffic compositions (highways or motorways), a moderate reduction of α factors applied to tandems systems and the uniformly distributed loads on Lane 1 may be applied (10 to 20%).

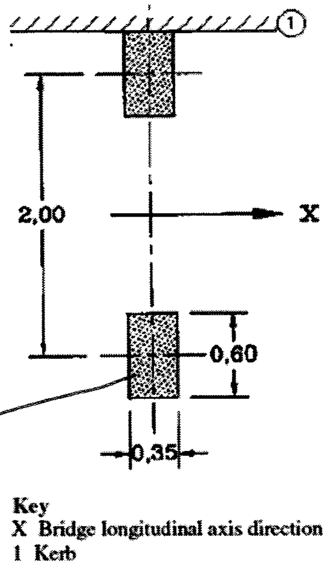
Load model 2 – LM2

400 kN axle load

For global and local verifications

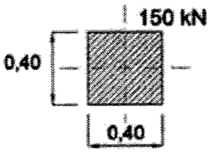
$\alpha_Q Q_k$

A single tyre (contact area) Of 200 kN may be used if more conservative



It's a contact surface that we have under the tyres of a lorry.

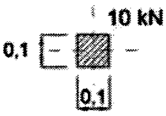
Load model 3 – LM3



N.B. For local verifications on NON protected footways and cycle tracks (see slide 42)

without a barrier (→ during maintenance, an heavy vehicle can go on the NON protected footways)

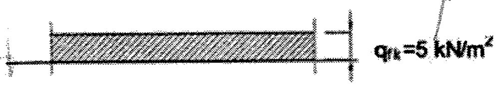
Load model 4 – LM4



N.B. For local verifications on protected footways and cycle tracks

is the load for our exercise (+ dead load)

Load model 5 – LM5

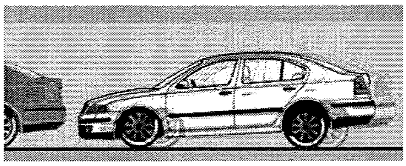


(1) Crowd loading, if relevant, should be represented by a Load Model consisting of a uniformly distributed load (which includes dynamic amplification) equal to 5 kN/m^2.

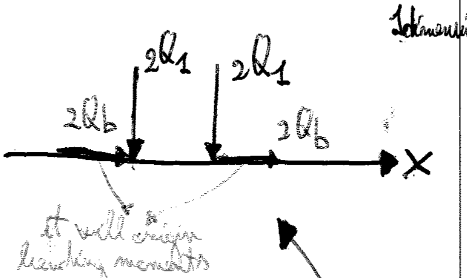
N.B. Combination value of crowd loading is 2.5 kN/m^2

Horizontal traffic loads

- Braking and acceleration forces
(that are longitudinal actions)



- Centrifugal forces
(are present only on curved bridges)
(the smaller the radius, the bigger the force).



- Braking and acceleration forces

(90 tons)

(2) The characteristic value of Q_{lk}, limited to 900 kN for the total width of the bridge, should be calculated as a fraction of the total maximum vertical loads corresponding to the Load Model 1 likely to be applied on Lane Number 1, as follows :

≅10% of distributed load coming from LM1

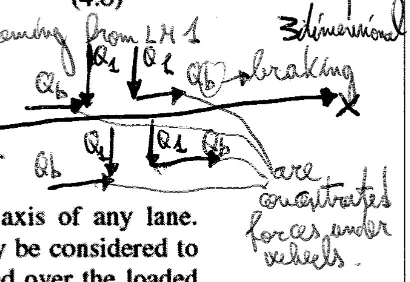
$$Q_{lk} = 0,6\alpha_{Q1}(2Q_{1k}) + 0,10\alpha_{q1}q_{1k}w_1L \qquad (4.6)$$

$$180\alpha_{Q1}\,(kN) \le Q_{lk} \le 900\,(kN)$$

≅60% of concentrated load coming from LM 1

where :

L is the length of the deck or of the part of it under consideration.



(4) This force should be taken into account as located along the axis of any lane. However, if the eccentricity effects are not significant, the force may be considered to be applied only along the carriageway axis, and uniformly distributed over the loaded length.

NOTE 1 For example, $Q_{lk} = 360 + 2,7L$ ($\le$ 900 kN) for a 3m wide lane and for a loaded length L>1,2 m, if α factors are equal to unity.

On the bridge we don't put load alone, but we group together.

Values of the multi component actions

	Loads on carriageway					Loads on footways
	Vertical			Horizontal		Vertical
Group of actions	Main action LM1-2-3-4-6	Special vehicles	Crowd	Braking Accel.	Centrifugal	Uniform
1	Characteristic value					2.5 kN/m²
2a	Frequent value			Characteristic value		
2b	Frequent value				Characteristic value	
3 (*)						5.0 kN/m²
4 (**)			5.0 kN/m²			5.0 kN/m²
5(***)	To be defined in design	Characteristic value				

(*) Only for footbridges

(**) Only for urban bridges (ex.: in the center of big cities)

(***) Only if special vehicles are taken into account

1: is intended to maximize vertical actions

2a: it's smaller than characteristic

2b: maximize horizontal transverse actions

4: no traffic on the bridge, but big crowd on the bridge (Marathon of New York).

2a: this model is not consistent (brak. acc. is calculated with charact. value while vertical loads are frequent)

2a maximize the effects of horizontal longitudinal actions

3: it's intended only for foot bridges

5 has 2 cases to take into account
- during construction (construction machinary is moving on the bridge while there is no traffic)
- abnormal loads (we have to ask road authority).

Fatigue load models

Cycling loads (like traffic)

5 Fatigue loads models

a) Fatigue Load Models 1, 2 and 3 are intended to be used to determine the maximum and minimum stresses resulting from the possible load arrangements on the bridge of any of these models ; in many cases, only the algebraic difference between these stresses is used in EN1992 to EN1999.

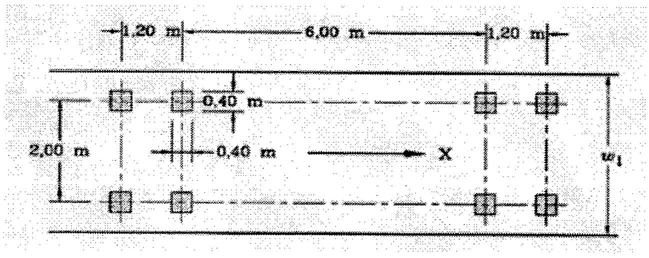
b) Fatigue Load Models 4 and 5 are intended to be used to determine stress range spectra resulting from the passage of lorries on the bridge.

c) Fatigue Load Models 1 and 2 are intended to be used to check whether the fatigue life may be considered as unlimited when a constant stress amplitude fatigue limit is given. Therefore, they are appropriate for steel constructions and may be inappropriate for other materials. Fatigue Load Model 1 is generally conservative and covers multi-lane effects automatically. Fatigue Load Model 2 is more accurate than Fatigue Load Model 1 when the simultaneous presence of several lorries on the bridge can be neglected for fatigue verifications. If that is not the case, it should be used only if it is supplemented by additional data. The National Annex may give the conditions of use of fatigue load models 1 and 2.

Fatigue load models

Fatigue load model 3 → consists of ideal lorry (not real)

(1) This model consists of four axles, each of them having two identical wheels. The geometry is shown in Figure 4.8. The weight of each axle is equal to 120 kN, and the contact surface of each wheel is a square of side 0,40 m.



Key
w_l : Lane width
X : Bridge longitudinal axis

(3) Where relevant, two vehicles in the same lane should be taken into account.

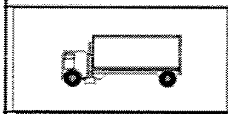
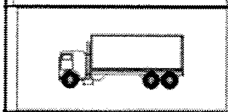
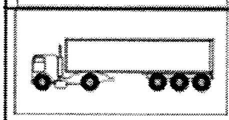
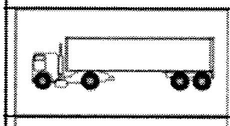
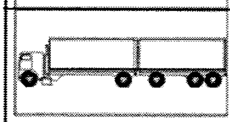
highway (like the Torino-Bardonecchia)

Fatigue load models

Fatigue load model 4

It consists of sets of Standard lorries which together produce effects Equivalent to those of typical traffic on european roads

the dimension are the same of load model 2, but the loads are smaller.

VEHICLE TYPE			TRAFFIC TYPE			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
LORRY	Axle spacing (m)	Equivalent axle loads (kN)	Lorry percentage %	Lorry percentage	Lorry percentage	Wheel type
	4,5	70 130	20,0 %	40,0	80,0	A B
	4,20 1,30	70 120 120	5,0 %	10,0	5,0	A B B
	3,20 5,20 1,30 1,30	70 150 90 90 90	50,0 %	30,0	5,0	A B C C C
	3,40 6,00 1,80	70 140 90 90	15,0 %	15,0	5,0	A B B B
	4,80 3,60 4,40 1,30	70 130 90 80 80	10,0 %	5,0	5,0	A B C C C

Actions for accidental design situations

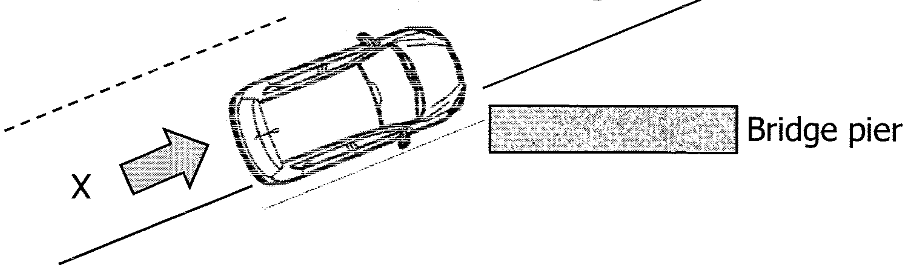
Collision forces from vehicles that are moving UNDER the bridge

- Impact of vehicles on vertical elements
  (piers, walls, abutments, etc...)

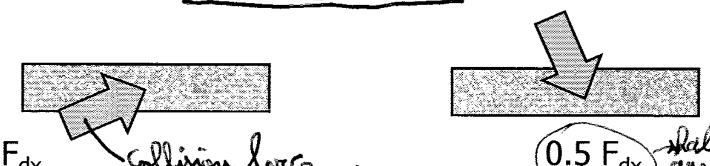
- Impact of vehicles on horizontal elements
  (e.g. lorries on deck intrados)

Impact of vehicles on vertical elements

X = vehicle driving direction
= direction of the road under the bridge



The collision effect is simulated by two different forces acting non simultaneously on the element

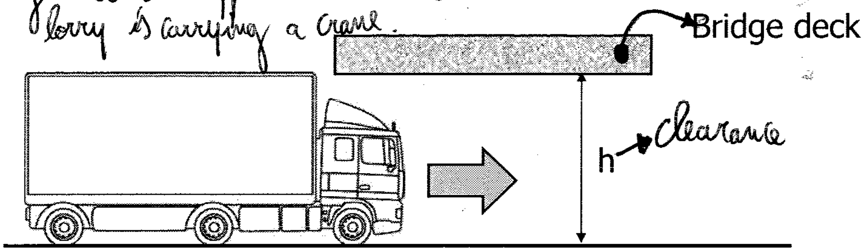


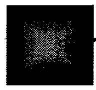
F_{dx} collision force

$0.5\ F_{dx}$ Half of the collision force applicated on the horizontal directions of the traffic.

Impact of vehicles on horizontal elements

General it happens when the lorry is carrying a crane.

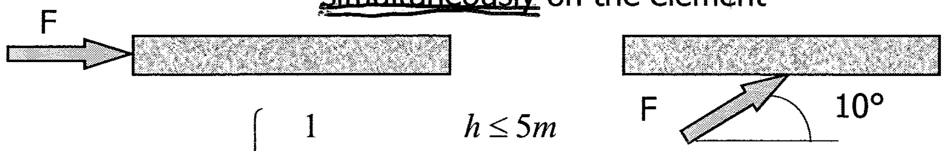




it's the arm of the crane that impact the deck.

0.25 x 0.25m
Impact area

The collision effect is simulated by two forces acting non simultaneously on the element



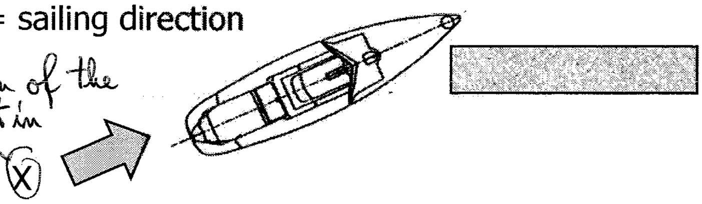
$$F = r \cdot F_{dx} \qquad r = \begin{cases} 1 & h \le 5m \\ 1-(h-5) & 5m < h \le 6m \\ 0 & h > 6m \end{cases}$$

is related to the clearance

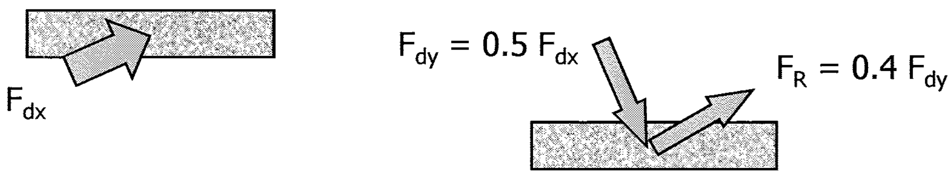
Impact of boats on piers

X = sailing direction

it's the direction of the flow of the current in the river



The collision effect is simulated by two different systems of forces acting non simultaneously on the pier



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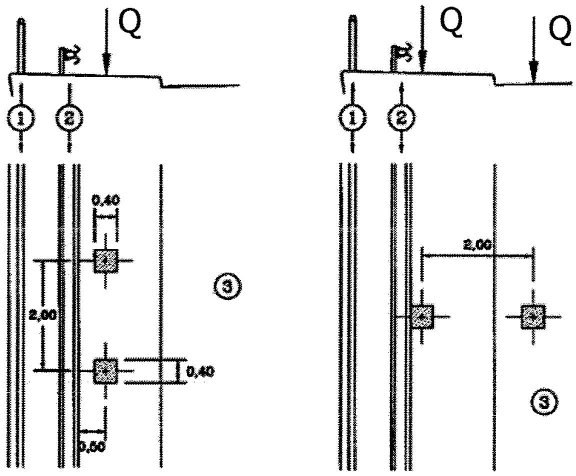
Actions for accidental design situations

Vehicles on footways and cycle tracks



Q = 150 kN

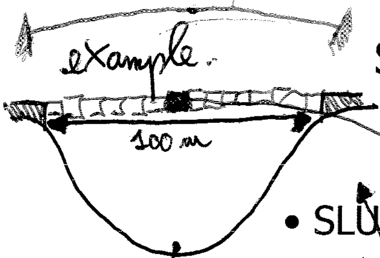
Other variable actions

- Hydraulic actions
- Impact of ice on the piers (north of Europe)
- Impact of flying vehicles (elicopters, etc..) on very tall piers

Nowadays is simple to create an incastro between the 2 cantilever

SLU and SLE Verifications

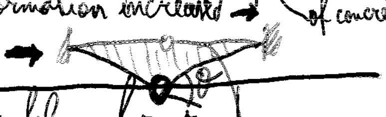
example.



when I come here, I pass from isostatic to iperstatic scheme.

In the past we put a hinge:

but, because of creep of concrete the deformation increased →



- SLU verification
- SLE deformation → it's not a big problem for road bridge (because the suspension of the cars make the deformation negligible) it's a problem for pedestrian bridge.

  deformation shouldn't cause trouble to traffic
- SLE stress limitation
  (see slide n°48)
- SLE cracking control
  (see slide n°49 to 50)

it's difficult to get here for construction → we built the bridge cantilevering from each abutments.

of material (linee tratteggiate a matita)

there was also replacement to repeat the pavement horizontally but then we had more load → more creep → more deformation

SLE Stress limitation

Material	SLE combination	Stress limit
Concrete	Characteristic	0.60 f_{ck}
Concrete	Quasi permanent	0.45 f_{ck}
Steel	Characteristic	0.80 f_{yk}

SLE Cracking control

Environ mental class group	SLE combinati on	Reinforcement			
		Sensible prestressed structures		Less sensible not prestressed structures	
Standard	Frequent	Crack opening	w_2	Crack opening	w_3
	Quasi perman.	Crack opening	w_1	Crack opening	w_2
Aggressive	Frequent	Crack opening	w_1	Crack opening	w_2
	Quasi perman.	Decompres sion	-	Crack opening	w_1
Very aggressive	Frequent	Crack formation	-	Crack opening	w_1
	Quasi perman.	Decompres sion	-	Crack opening	w_1

BRIDGE DESIGN

Actions on rail bridges

EN 1991-2 September 2003
Norme tecniche per le costruzioni luglio-07

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Traffic actions

Load models for railway traffic

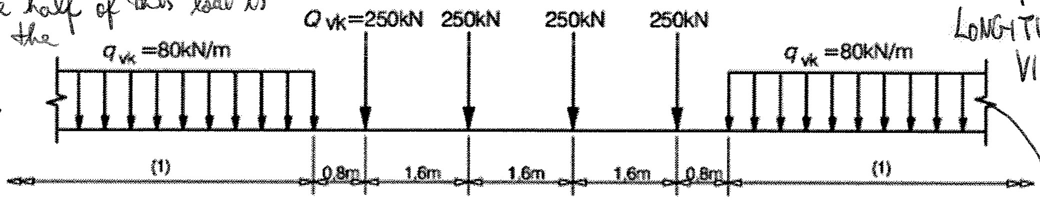
LM71 (it's the most famous)

Load Model 71 represents the static effect of vertical loading due to normal rail traffic.

The uniform distributed load can be applied in segments in order to achieve the most unfavourable effect (also, in order to load only the zone of the influence surface that you need to load to maximize the effects) → achieve the most unfavourable effect

This is the total load on a track. One half of this load is on 1 rail, the other half is on the other rail.

LONGITUDINAL VIEW



Key
(1) No limitation

q_{VK} is longer as much as I want and it's segmentable in order to achieve the most unfavourable effect.

Traffic actions

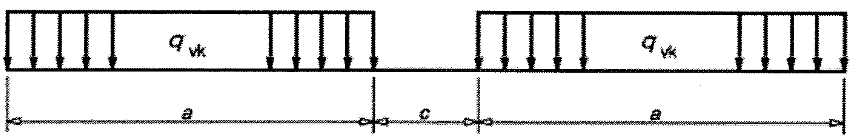
Load models for railway traffic

SW

(1) Load Model SW/0 (is light) represents the static effect of vertical loading due to normal rail traffic on continuous beams.

(2) Load Model SW/2 (is heavy) represents the static effect of vertical loading due to heavy rail traffic.

These loads are not segmentable. You can move it, but not divide it into pieces.

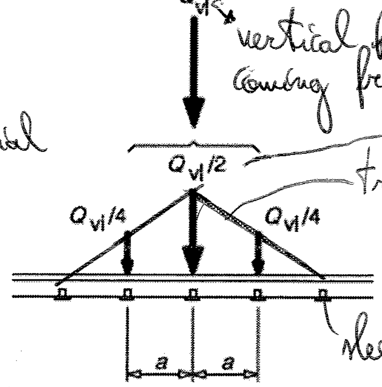


Load Model	q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15,0	5,3
SW/2	150	25,0	7,0

Longitudinal distribution of wheel load by the rail

The physical model of the wheel is very complicated, because the contact zone of a circle (wheel) is just a point. But, if so, the force will tend to ∞.

So we'll consider that: there is always a small contact area and plasticization of the material of both rail and wheel, that is moving in our element (because our element is in rotation)



vertical force coming from 1 wheel.

Convention: when the wheel is on 1 sleeper ⇒ the total load is carried by:
- 1/2 by the sleeper;
- 1/4 by the 2 sleepers near the first one.

triangular distribution of the load. This load is moving ⇒ this load ($Q_{vi}/2$) can be centered on a sleeper or between 2 sleepers (or in a generic position).

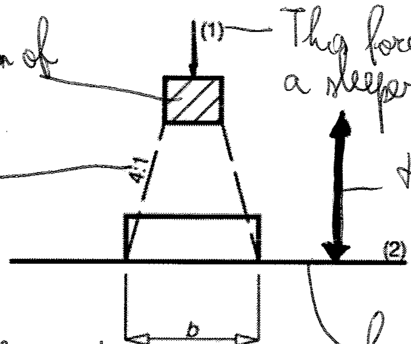
sleeper

Key

Q_{vi} is the point force on each rail due to Load Model 71 or a wheel load of a Real Train in accordance with 6.3.5, Fatigue Train or HSLM (except for HSLM-B)

a is the distance between rail support points

Longitudinal distribution of load by the ballast



The force is diffused under a sleeper

Cross section of the sleeper

Slope of the ballast

thickness of the ballast

Key
(1) Load on sleeper
(2) Reference plane

The force in the sleeper is transferred with an angle of 4:1 to the ballast.

plane on which the ballast is placed (extrados of the bridge: extrados of the concrete slab)

Traffic actions

Actions for non public footpaths → are used for maintenance

Uniform distributed load of 10 kN/m^2 without dynamic increment.

it's too big for the pedestrian load → it takes into account that into footpaths there are maintenance equipment

The main problem of a rail bridge is: in train all the damping and suspension systems are harder than in a car → we don't have a soft damping interface

The speed of the train is bigger than the speed of the car.

Dynamic effects : we don't have a soft damping interface, but a rigid one.

Limits of bridge natural frequency n_0 [Hz] as a function of L [m]

A fully dynamic investigation is not easy to do. We use a more simple method to speed up our design.

The upper limit of n_0 is governed by dynamic enhancements due to track irregularities and is given by:

$n_0 = 94,76L^{-0,748}$ (6.1)

The lower limit of n_0 is governed by dynamic impact criteria and is given by:

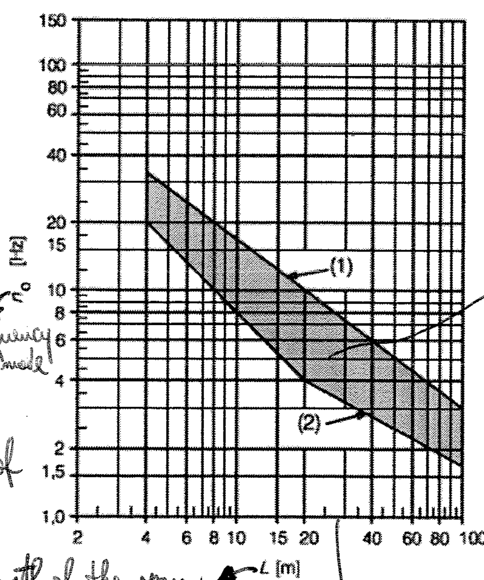
$n_0 = 80/L$ for $4\text{m} \leq L \leq 20\text{m}$

$n_0 = 23,58L^{-0,592}$ for $20\text{m} < L \leq 100\text{m}$ (6.2)

where:

frequency of 1st mode of vibration ← n_0 is the first natural frequency of the bridge taking account of mass due to permanent actions.

L is the span length for simply supported bridges or L_Φ for other bridge types.



first natural frequency of the deck (first mode of vibration)

frequency of

length of the span

this grey area means:
- if you are inside: you can use simplified analysis
- if you are outside: full dynamic analysis

horizontal axis: more or less a logarithmic scale.

The coefficient $\Phi(\Phi_2, \Phi_3)$ can NOT be used for:

- The Unloaded train
- Real trains
- Trains for fatigue analysis

So ϕ are used only with SW0/SW2.

(in Italy we like to use ballast!!!)

in Italy is very uncommon (in Northern countries it is more common)

For steel deck without ballast (track directly connected to the deck) should be considered a coefficient β additional to → $\Phi(\Phi_2, \Phi_3)$ ("beta")

$$\beta = 1.0 \quad for \quad L_\phi < 8m \quad and \quad L_\phi > 90m$$

$$\beta = 1.1 \quad for \quad 8m < L_\phi < 90m$$

Determinant length

to calculate ϕ (one table for steel / one table for concrete)

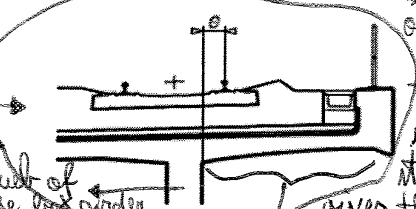
Steel deck plate: closed deck with ballast bed (orthotropic deck plate) (for local and transverse stresses)

	Deck with cross girders and continuous longitudinal ribs:	
1.1	Deck plate (for both directions)	3 times cross girder spacing
1.2	Continuous longitudinal ribs (including small cantilevers up to 0,50 m)[a]	3 times cross girder spacing
1.3	Cross girders	Twice the length of the cross girder
1.4	End cross girders	3,6m [b]

Determinant length

Concrete deck slab with ballast bed (for local and transverse stresses)

4.1	Deck slab as part of box girder or upper flange of main beam	
	- spanning transversely to the main girders	3 times span of deck plate
	- spanning in the longitudinal direction	3 times span of deck plate
	- cross girders	Twice the length of the cross girder
	- transverse cantilevers supporting railway loading	- $e \leq 0{,}5$ m: 3 times the distance between the webs - $e > 0{,}5$ m: *



web of the box girder

in his: our railway authority is afraid of dynamic effects: they don't want cantilever system. But in a box girder there are 2 cantilevers: so there are limitations in the dimensions of these cantilevers.

Our railway authority is really afraid of the dynamic effects. The cantilever structure is the structure that gives the biggest dynamic effects (like trampolino).

very big!

cantilever structure

also with $e \leq 0{,}5$ (that is very small), the dynamic effects are big (3 times the distance between the webs)!!!

Determinant length

Concrete deck slab with ballast bed (for local and transverse stresses)

4.2	Deck slab continuous (in main girder direction) over cross girders	Twice the cross girder spacing
4.3	Deck slab for half through and trough bridges:	
	- spanning perpendicular to the main girders	Twice span of deck slab + 3m
	- spanning in the longitudinal direction	Twice span of deck slab
4.4	Deck slabs spanning transversely between longitudinal steel beams in filler beam decks	Twice the determinant length in the longitudinal direction

Determinant length

Main girders		
5.3	Portal frames and closed frames or boxes:	
	- single-span	Consider as three-span continuous beam (use 5.2, with vertical and horizontal lengths of members of the frame or box)
	- multi-span	Consider as multi-span continuous beam (use 5.2, with lengths of end vertical members and horizontal members)
5.4	Single arch, archrib, stiffened girders of bowstrings	Half span
5.5	Series of arches with solid spandrels retaining fill	Twice the clear opening
5.6	Suspension bars (in conjunction with stiffening girders)	4 times the longitudinal spacing of the suspension bars

Determinant length

Structural supports		
6	Columns, trestles, bearings, uplift bearings, tension anchors and for the calculation of contact pressures under bearings.	Determinant length of the supported members

Horizontal actions

Centrifugal forces

where:

Q_{tk}, q_{tk} Characteristic values of the centrifugal forces [kN, kN/m]

Q_{vk}, q_{vk} Characteristic values of the vertical loads specified in 6.3 (excluding any enhancement for dynamic effects) for Load Models 71, SW/0, SW/2 and "unloaded train". For load model HSLM the characteristic value of centrifugal force should be determined using Load Model 71. → there is a incongruence

f Reduction factor (see below)

v Maximum speed in accordance with 6.5.1(5) [m/s]

V Maximum speed in accordance with 6.5.1(5) [km/h]

g Acceleration due to gravity [9,81 m/s²]

r Radius of curvature [m]

You use HSLM load model, but horizontal actions must be calculated with LM71.

For the load models SW/2 and "unloaded train" the value of the reduction factor f should be taken as 1,0. → we have no reduction

Horizontal actions

Centrifugal forces

function of the speed of the line and of L_f.

(8) For Load Model 71 (and where required Load Model SW/0) the reduction factor f is given by:

$$f = \left[1 - \frac{V-120}{1000}\left(\frac{814}{V} + 1{,}75\right)\left(1 - \sqrt{\frac{2{,}88}{L_f}}\right)\right] \quad (6.19)$$

e.g.: SW/0 (slide 56/119): a = 15 m so L_f is 15+15 m.

centrifugal force

it's like the speed limit of a road (even if your car can reach bigger velocity)

subject to a minimum value of 0,35 where:

L_f is the influence length of the loaded part of curved track on the bridge, which is most unfavourable for the design of the structural element under consideration [m].

V is the maximum speed in accordance with 6.5.1(5).

is the speed of the line, not the maximum speed that the train can general reach.

$f=1$ for either $V \leq 120$ km/h or $L_f \leq 2{,}88$ m

$f<1$ for 120 km/h $< V \leq 300$ km/h (see Table 6.7 or Figure 6.16 or equation 6.19) and $L_f > 2{,}88$m

$f_{(V)} = f_{(300)}$ for $V > 300$ km/h.

→ $f(V=300)$ ⇒ we have a plafond at V=300km/h (f don't depend from V anymore).

Horizontal actions

3. Traction and braking forces

it should be placed under the loaded area up to a maximum of 1000 kN!

Traction force: acceleration

Q_{lak} = 33 [kN/m] $L_{a,b}$ [m] ≤ 1000 [kN] for Load Models 71, SW/0, SW/2 and HSLM

Braking force: in reality is distributed

Q_{lbk} = 20 [kN/m] $L_{a,b}$ [m] ≤ 6000 [kN] for Load Models 71, SW/0 and Load Model HSLM → for passenger trains

Q_{lbk} = 35 [kN/m] $L_{a,b}$ [m] for Load Model SW/2 (for heavy trains)

Why $Q_{lbk} < Q_{lak}$, but the maximum value is bigger? If you want to accelerate a train you use only the wheels that have motor. If you want to brake a train → you use all the wheels.

The characteristic values of traction and braking forces shall not be multiplied by the factor Φ (see 6.4.5.2) or by the factor f in 6.5.1(6).

In time: it's easier to stop a train than to accelerate it → (only the wheels of the first coaches are engined, instead brakes are in all coaches) If you need to stop fast → max Q_{lbk} is higher than max Q_{lak}.

So traction is a concentrated force (because also in reality is like this) and braking in reality is distributed, but we consider it a concentrated force (at the top or at the end of the train) in order to simplify the design process.

Traction and braking forces for decks with more than 1 track

The forces that arrive in the rails are the opposite of what the actions are (ACTION → ← REACTION)

Bridges with 2 tracks: → which is the 1° or the 2° track? It's up to you! It depends on which is the worst position in the influence line (of bending moment for example).

- 1° track: braking
- 2° track: traction

→ If I have a train that is braking in 1 track and I have the other train that is accelerating in the other track → the 2 forces are in the same direction.

Bridges with more than 2 tracks: → tracks near to the station

- 1° track: braking
- 2° track: traction
- 3° track: 50% braking
- 4° track and subsequent : Unloaded for horizontal forces

ESTATE: +30°C ; steel deck + ballast = +20°C → differenza = 10°C
e.g. INVERNO: -40°C ; steel deck + ballast = -20°C → differenza = 20°C

c) Thermal variation on the rail

is a piece of steel that is directly exposed to the sun, more...

1) The thermal variation on the rail can be neglected if the rail has expansion joints.
2) Otherwise +30°C / - 40°C from the rail regulation temperature.

very big

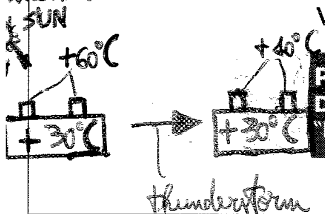
every single elements of the rail is welded to the other → no joints (saldato)

It can happen that the rail and the deck don't have the same direction of stresses

In case 2) applies:

- For STEEL decks the thermal variations of the rail should be applied with the thermal variation of the deck with the same sign.
- For CONCRETE or composite decks the thermal variations (positive and negative) of the rail should be coupled with nil thermal variation of deck and the thermal variation of the deck (positive or negative) should be coupled with nil thermal variation of the rail.

thermal because one goes with reason and the other with day. Sometimes we have shortening in the rail and elongation in the deck because of different thermal inertia:



For this reason we don't want big thermal actions because we have "deformazioni impedite" → thermal stress. To mitigate this problem when we weld 2 pieces of rails, we don't do it at temperature but at a temperature that is almost the medium value between +30°/-40°C ambient.

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Interaction effects among Track – ballast – deck – piers - foundation

The problem is that the rail is a very small element and has a very small stiffness compared to the stiffness of the bridge, so, if the bridge falls down → the rail follows it → the weak point of our study is the rail.

(1) Where the rails are continuous over discontinuities in the support to the track (*e.g.* between a bridge structure and an embankment) the structure of the bridge (bridge deck, bearings and substructure) and the track (rails, ballast etc.) jointly resist the longitudinal actions due to traction or braking. Longitudinal actions are transmitted partly by the rails to the embankment behind the abutment and partly by the bridge bearings and the substructure to the foundations.

(2) Where continuous rails restrain the free movement of the bridge deck, deformations of the bridge deck (*e.g.* due to thermal variations, vertical loading, creep and shrinkage) produce longitudinal forces in the rails and in the fixed bridge bearings.

(3)P The effects resulting from the combined response of the structure and the track to variable actions shall be taken into account for the design of the bridge superstructure, fixed bearings, the substructure and for checking load effects in the rails.

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Parameters affecting the combined response of the structure and track

d) Properties of the track:
   - axial stiffness of the rail,
   - resistance of the track or the rails against longitudinal displacement considering either:
     - resistance against displacement of the track (rails and sleepers) in the ballast relative to the underside of the ballast, or
     - resistance against displacement of the rails from rail fastenings and supports *e.g.* with frozen ballast or with directly fastened rails,

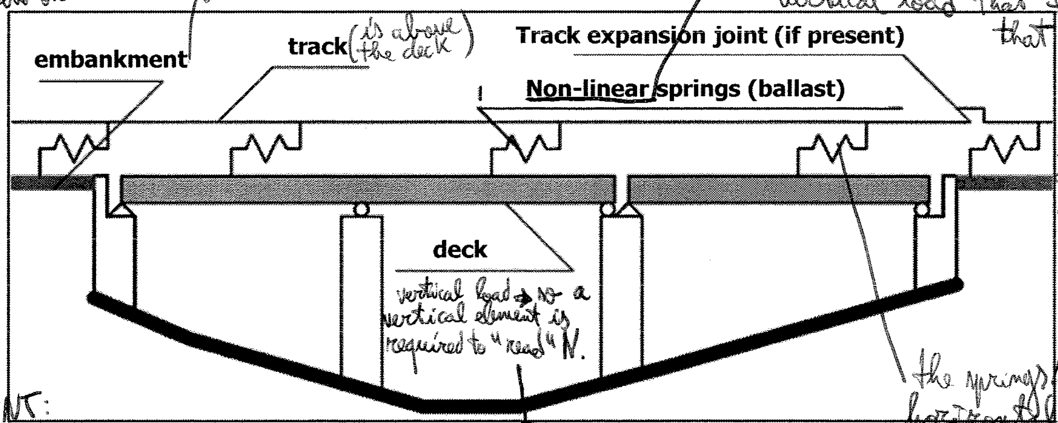
   where the resistance against displacement is the force per unit length of the track that acts against the displacement as a function of the relative displacement between rail and the supporting deck or embankment.

Vedi anche pagina seguente appunti!

Ballast or devices are modelled as springs. Their stiffness depend on the friction (→ non-linear springs) so depend on the vertical load that I have at that point.

Scheme of structural model to evaluate the interactions

because the springs ~~[illegible]~~ depend on the friction, so depend on the vertical load that I have at that point.

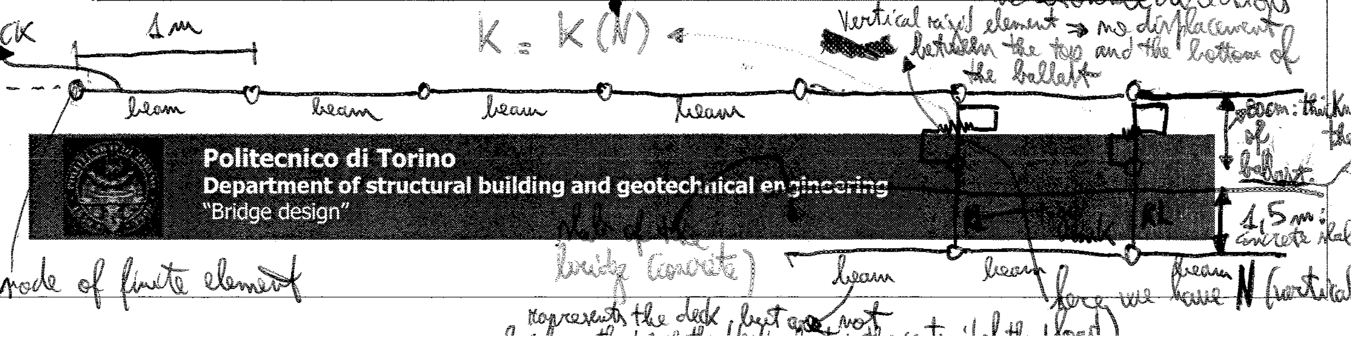


ballast: only horizontal displacement can occur.

(ballast or devices)

the springs are in horizontal direction

FINITE ELEMENT:



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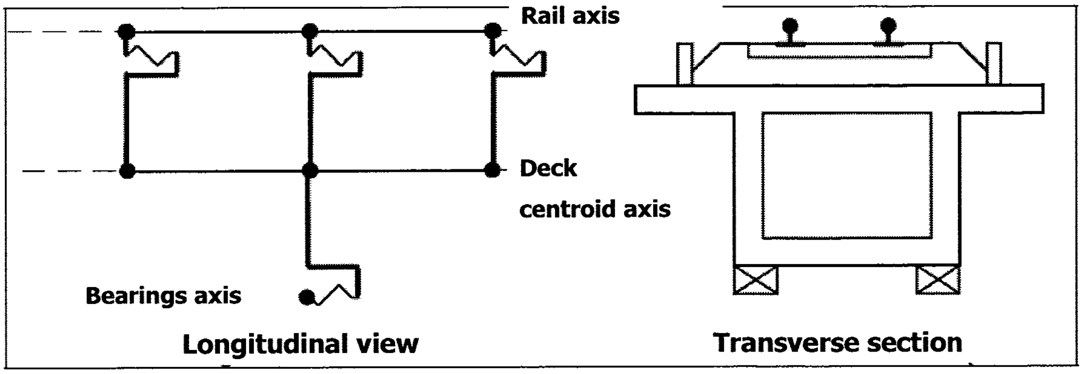
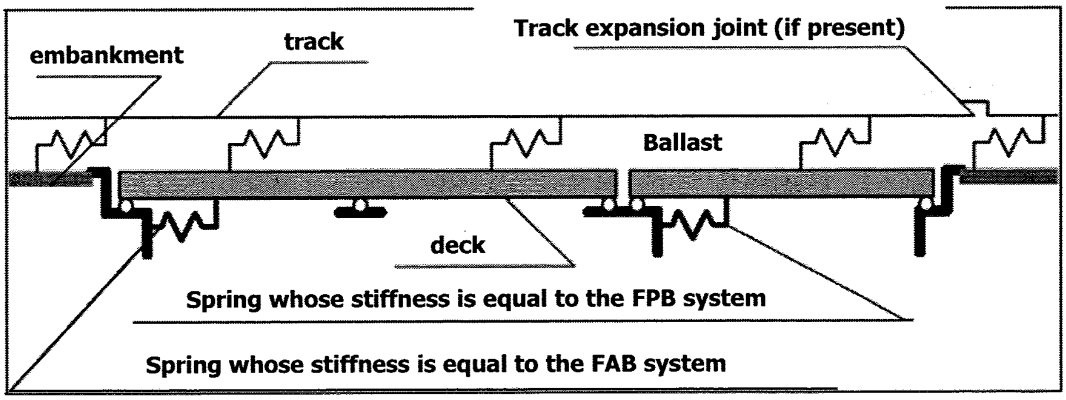
Scheme of track-deck-bearings connection



Simplified structural model



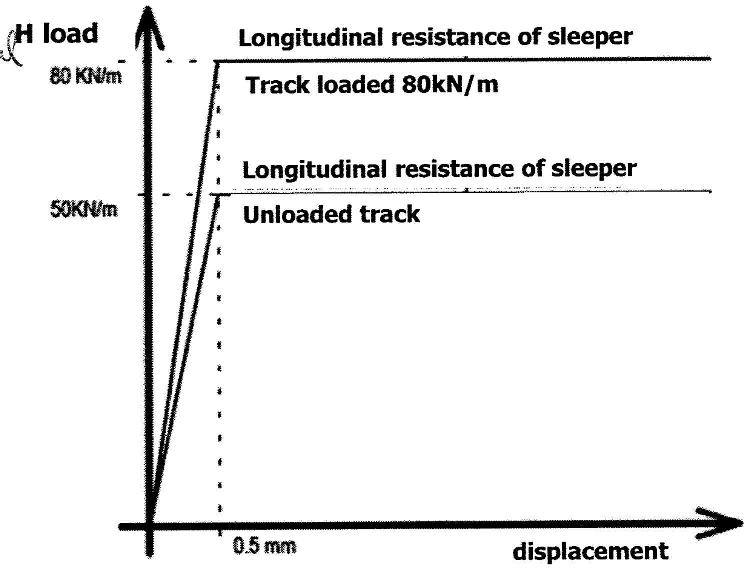
FPB = Foundation – Pier – Bearings

FAB = Foundation – Abutment - Bearings

not common in Italy (alternative to the ballast)

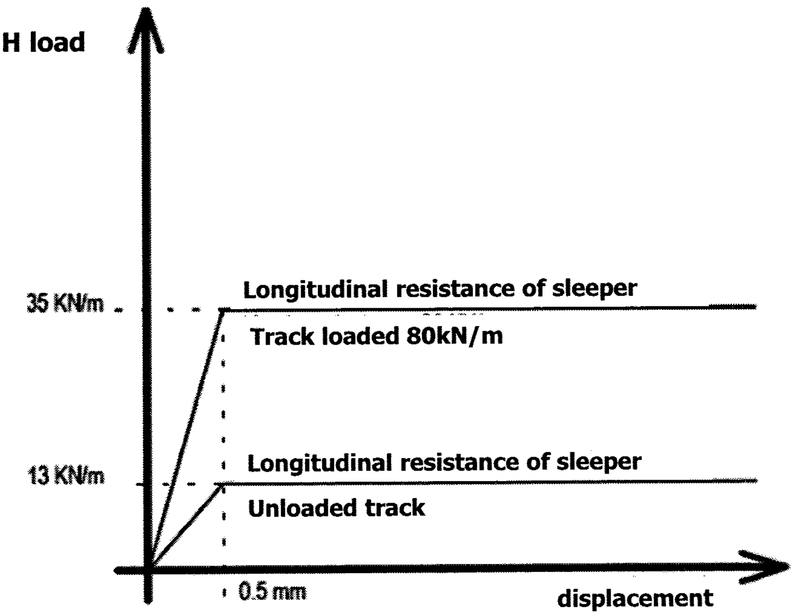
K joint: variation of longitudinal shear force with vertical load for 1 track

It's a mechanical device: it has a narrow range of behaviour.



Elastic joint:

variation of longitudinal shear force with vertical load for 1 track



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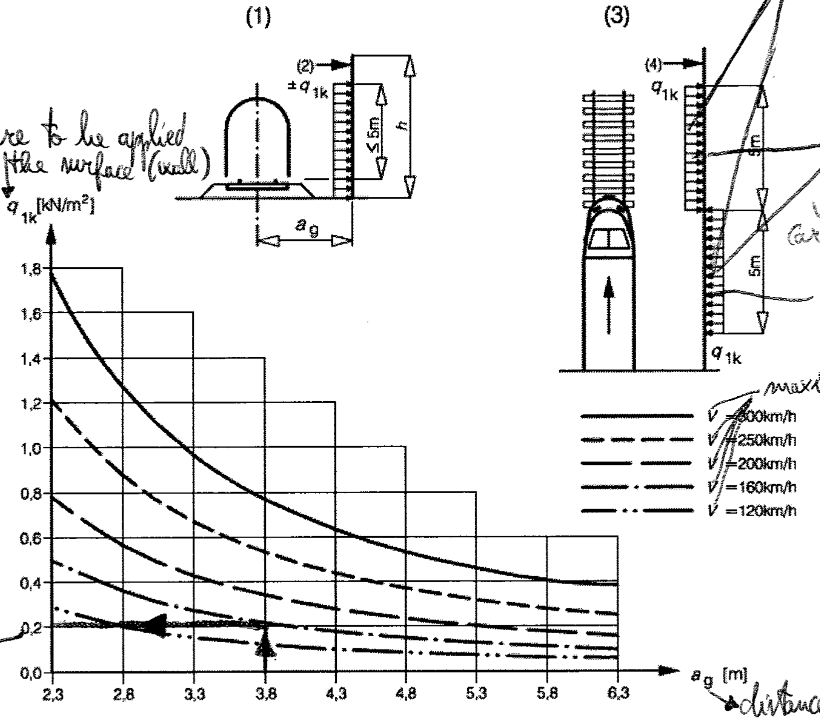
the force can have 2 different directions

Simple vertical surfaces parallel to the track (e.g. noise barriers)



pressure to be applied to the surface (wall)

it's the same that happens when with a car you pass a lorry this force pull you to the train

maximum speed that the train can reach in that point

20 Kg/m²

distance from the axis of the train

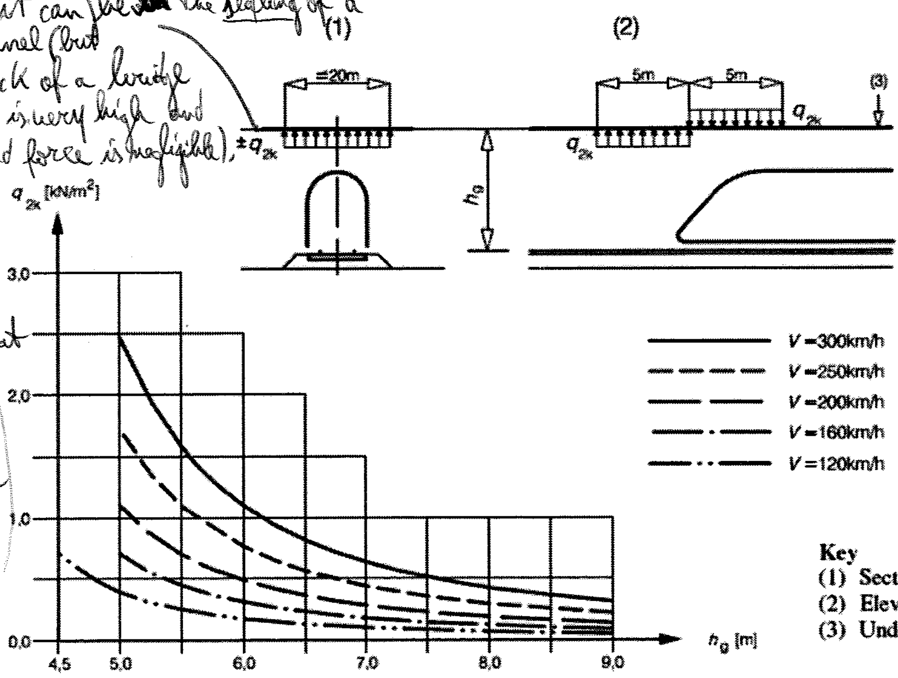
Key
(1) Section
(2) Surface of structure
(3) Plan view
(4) Surface of structure

it is very high and heavy → wind force is negligible)

"sigillatura"

Simple horizontal surfaces above the track (e.g. overhead protective structures)

it can be the sealing of a tunnel (but or a deck of a bridge also that deck is very high and heavy → wind force is negligible), But, during maintenance, it can be a temporary structure, that is not so heavy ⇒ ⇒ this wind force can create problems ⇒ take into account.



Key
(1) Section
(2) Elevation
(3) Underside of the structure

e.g.: steel structures built for pedestrian to cross the line

6.6.6 Surfaces enclosing the structure gauge of the tracks over a limited length (up to 20 m) (horizontal surface above the tracks and at least one vertical wall, *e.g.* scaffolding, temporary constructions)

is typical of temporary building structures (scaffolding, temporary structures built for maintenance, ...).

(1) All actions should be applied irrespective of the aerodynamic shape of the train:
– to the full height of the vertical surfaces:

don't use the correction factor k.

$$\pm k_4 q_{1k} \qquad (6.35)$$

where:

q_{1k} is determined according to **slide 101**
$k_4 = 2$
– to the horizontal surfaces:

$$\pm k_5 q_{2k} \qquad (6.36)$$

where:

q_{2k} is determined according to **slide 102**
$k_5 = 2{,}5$ if one track is enclosed,
$k_5 = 3{,}5$ if two tracks are enclosed.

The train leaves the correct position on the track → 2 cases

Derailment over the bridge ... what will happen ???

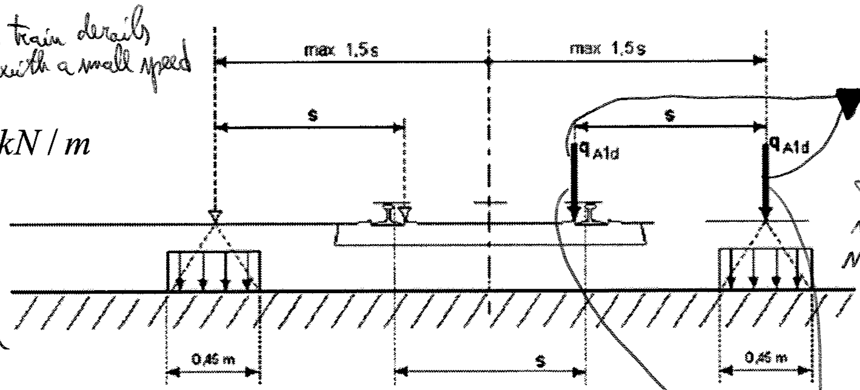
Case 1: the train derails but with a small speed

is representable

$$q_{A1d} = 60kN/m$$

uniformly distributed load

This load is representable: you can place where you like.



the 2 wheels of the train

the train has not enough momentum to jump out off the rails.

The load includes dynamic effect and may be placed transversally in every position within the field ± 1.5 s

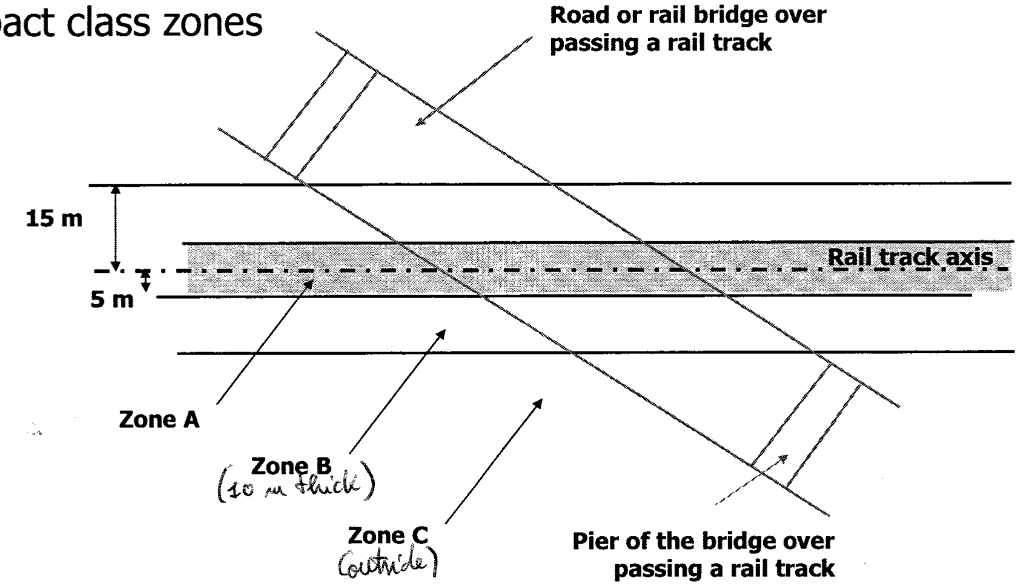
the designer has to choose the position of this 2 forces ... of course the worst situation is this image (closer to the edge) that is the outermost position.

Only small entity damage can be accepted in order to re-open the line after light maintenance: we don't want to have damage on the bridge and remain in serviceability conditions.

we want to

Derailment under a road or a rail bridge

Impact class zones



Derailment under a road or a rail bridge

Impact forces : are function of the impact zones

Zone A
- **4000 kN parallel to the track** → 400 tons (case: 400 cars crushing to the pier → big force, but the pier should remain in position: if the structure collapses → more bad consequences.)
- **1500 kN orthogonal to the track**

Zone B
- **2000 kN parallel to the track**
- **750 kN orthogonal to the track**

Zone C
- **Nothing**

These forces should be considered at 1.80 m from the rail level, and not acting simultaneously → (for example for zone A: you put 4000 kN and you design, then you put 1500 kN and you design).

Values of the multi component actions

Values inside brackets should be considered when the action is favourable.

Group 4 should be considered only for crack control. Value 0.6 should be use d for 2 tracks loaded; while value 0.4 should be considered for more than 2 tracks loaded.

SLU actions safety factors

for buildings we use 1,5.

		Coeff.	EQU(1)	A1 STR	A2 GEO	Accidental situation	Seismic situation
Permanent actions	Fav. Unfav.	γ_{G1}	0,90 1,10	1,00 1,35	1,00 1,00	1,00 1,00	1,00 1,00
Non struct. (2) permanent actions	Fav. Unfav.	γ_{G2}	0,00 1,50	0,00 1,50	0,00 1,30	1,00 1,00	1,00 1,00
Ballast (3)	Fav. Unfav.	γ_B	0,90 1,50	1,00 1,50	1,00 1,30	1,00 1,00	1,00 1,00
Variable traffic loads (4)	Fav. Unfav.	γ_Q	0,00 1,45	0,00 1,45	0,00 1,25	0,00 0,20(5)	0,00 0,20(5)
Other variable actions	Fav. Unfav.	γ_{Qi}	0,00 1,50	0,00 1,50	0,00 1,30	0,00 1,00	0,00 0,00
Prestressing	Fav. Unfav.	γ_P	0,90 1,00(6)	1,00 1,00(7)	1,00 1,00	1,00 1,00	1,00 1,00

SLE actions combination factors

(1) 0.8 for only one track loaded, 0.6 for two tracks loaded, 0.4 if three or more tracks are loaded

(2) If the wind is taken as the base action, Ψ_0 coefficients for traffic loads should be taken equal to 0.

if $0 < f < 20$ Hz ⇒ human life frequency (people have nausea)
if $f > 20$ Hz ⇒ it's a vibration: something like a sound.

Strong SLE done only for rail bridges.

SLE Verifications for rail-traffic safety

SLE: there are extras than the traditional 3 (crack, deformation, stresses limitation)

static action

vibration like stress (it's from 20 Hz to 200 Hz). this frequency makes you sick (nauseating).

- Vertical acceleration of the deck
  $a_v \leq 3.5 m/s^2$ if $0<f<20$ Hz

it's not a small acceleration: it's related to the comfort of the passengers and the stability of goods and train.

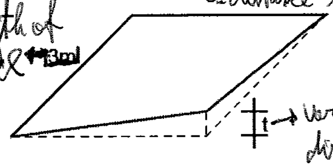
- Torsional deformation of the deck
  (to be calculated with LM71 and incremented with dynamic effect)

According to the speed of the line, we admit a torsional behaviour on 3 m length. It's a confort requirement.

V≤ 120 km/h;	t ≤ 4,5 mm/3m
120<V≤ 200 km/h;	t ≤ 3,0 mm/3m
V > 200 km/h;	t ≤ 1,5 mm/3m

length of the rail

s = distance between the rails.



t → variation, in the vertical direction, of 1 corner.

If V>200 km/h ⇒ t≤1.2 mm/3m for real trains with dynamic effect

BRIDGE DESIGN

COURBON METHOD

we use now because is good to predimension (to have an idea about the dimension of the bridge), but none we'll use to design (we use FEM).
The same consideration we can do for Engheser method.

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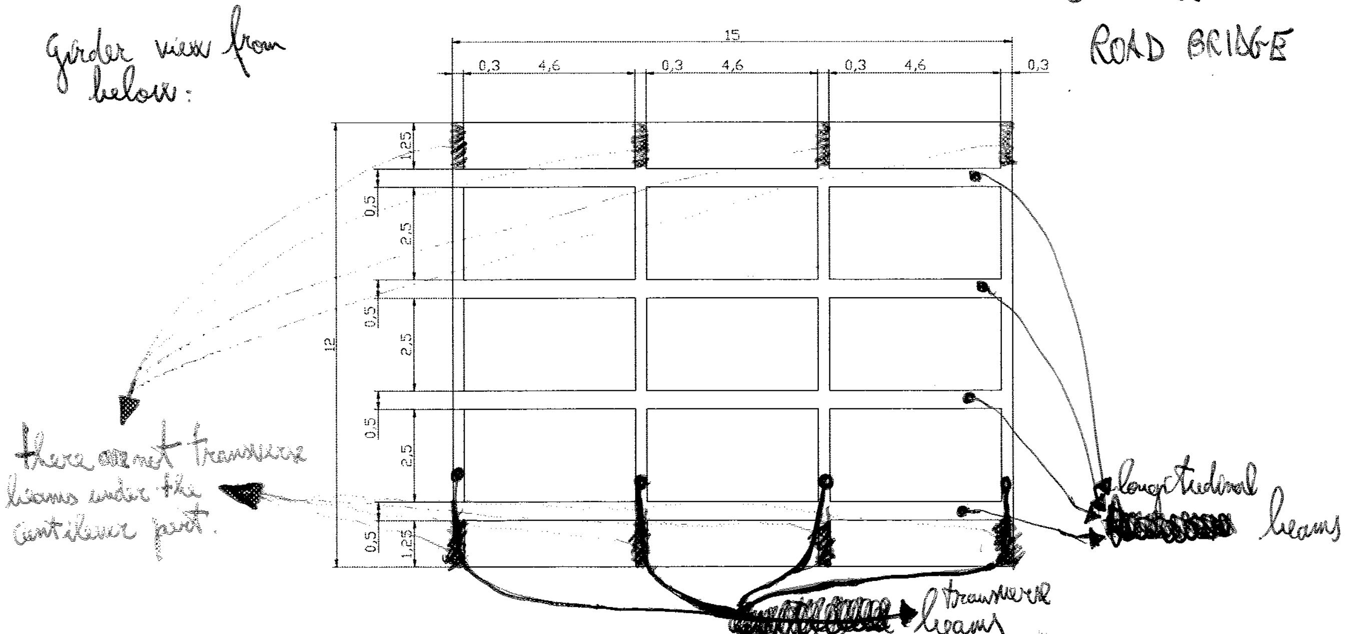
We'll study a girder for a didactic bridge (none will never build a girder bridge with the following shape.)

Girder deck dimensions - Bottom view

It's a ROAD BRIDGE

Girder view from below:



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(we saw it in the theory)

The beams subjected to the highest bending moment are the external ones, so the other beams are designed as they were subjected to the same actions. This reduces design time and is a safe approximation.

(so they'll be a little bit overdimensioned, but none cares)

We proceed calculating the internal actions (bending moment and shear) in the mid-span section of an external beam called beam 1. it's just a choice (general you should do for every section)

(we don't have time to calculate N, T, ... but in reality you should)

Values of the multi component actions

general you have to do for all groups.

In this exercise we will solve the structure only for the multi component action group n° 1. Needless to say that the other groups have to be taken into account too. "superfluo"

	Loads on carriageway					Loads on footways
	Vertical			Horizontal		Vertical
Group of actions	Main action LM1-2-3-4-6	Special vehicles	Crowd	Braking Accel.	Centrifugal	Uniform
1	Characteristic value					2.5 kN/m²

Load analysis

- **Permanent loads** g_2
  1. Kerb
  2. Pavement
  3. Vehicle restraint system
  4. Pedestrian parapet (balaustra pedonale)

we know that is empty, but we calculate as it's full (safe approximation)

1. Kerb $g_{2k} = b \cdot h \cdot l \cdot \gamma = 1.5m \cdot 0.23m \cdot 15m \cdot 25 \frac{kN}{m^3} = 129kN$
2. Pavement $g_{2p} = b \cdot l \cdot \gamma = 1.5m \cdot 15m \cdot 3 \frac{kN}{m^2} = 67.5kN$

The load value for the pavement takes into account that several layers of asphalt may be placed one over another during maintenance of the road. → general, when it's built, the weight is smaller than 3 kN/m2, but after maintenance it becomes 3 kN/m2, because we put more layers of asphalt

Load analysis

is an approximation for excess

3. V. R. S. (vehicle restraints system) $g_{2vrs} = l \cdot \gamma = 15m \cdot 2 \frac{kN}{m} = 30kN$

4. Pedestr. parapet $g_{2pp} = l \cdot \gamma = 15m \cdot 1.0 \frac{kN}{m} = 15kN$

Permanent load on the outermost beam

$$g_{2,1b} = (g_{2,k} + g_{2,p} + g_{2,vrs} + g_{2,pp})/l =$$
$$= (129 + 68 + 30 + 15) kN/15m = 242/15 = 16kN/m \cong \underline{44\%} g_{1,1b}$$

self weight of the bridge

very high value related to the self weight of the bridge. It's a upper limit: in reality we'll always have percentage lower than this. The longer the span, the smaller is that percentage (≃ 10%). In this example

One simplification:

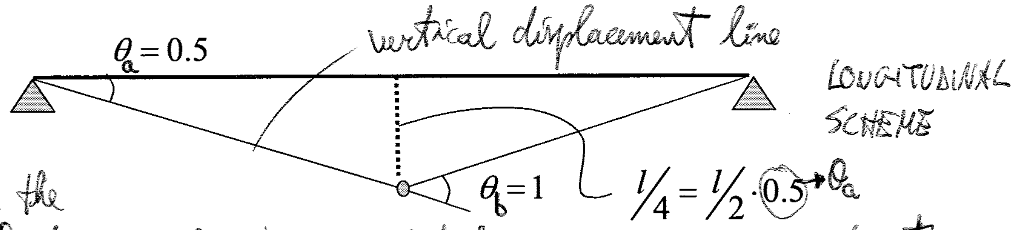
a. The permanent load for the outermost beam is grater then for the other beams. In this example the load of kerb and barriers is fully given to the outermost beam, in reality it would distribute itself according to Courbon theory on the others beams resulting in a lesser weight for beam one.

Bending moment in mid-span

- **Drawing influence surface**

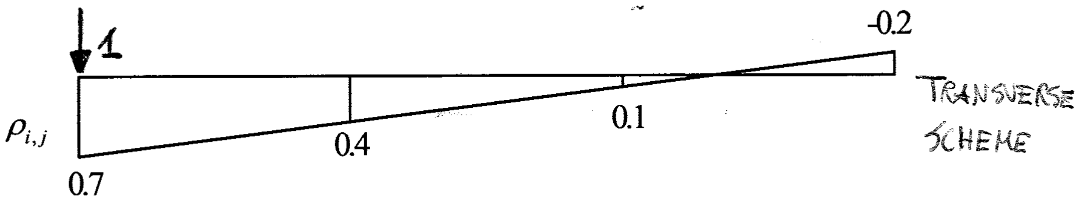
One dimensional influence line for longitudinal simply supported beam

For M influence line in the mid span → put hinge in the mid span and put $\theta = 1$



Now we have to see what happens in the transverse direction:

Transversal load repartition according to Courbon theory



Transverse distribution

$\rho_{1,j}$ has 2 different definitions (both true):

$\rho_{1,j}$ Is the amount of the load P=1 applied on the beam 1 that goes on the beam j (j=1÷4)

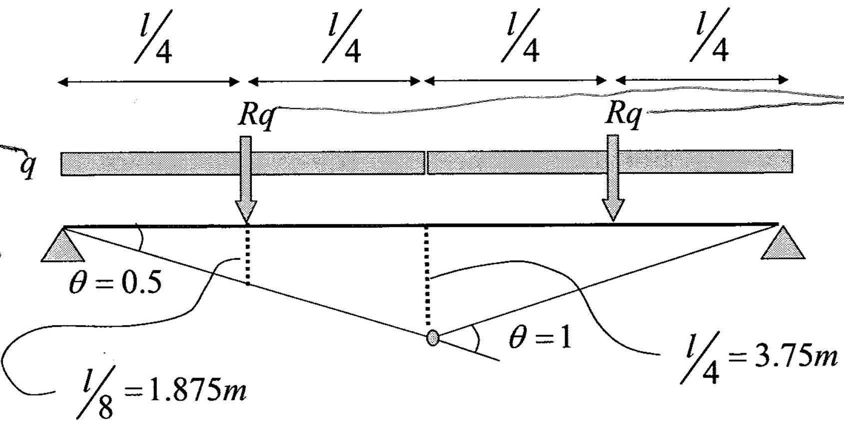
Or:

$\rho_{1,j}$ Is the amount of the load P=1 applied on the beam j (j=1÷4) that goes on the beam 1 (it's a direct application of Betti-Maxwell theory)

Now we do the same thing for longitudinal distributed loads.

Longitudinal distribution (uniformly distributed loads)

we place q all over the bridge, because the influence line is everywhere positive.



As the influence line is straight, we can substitute the uniform load with its resultant.

general value of q.

$$\Rightarrow M_{S,q} = 2\cdot(1.875\cdot Rq) = 3.7\cdot \frac{l}{2} q = 3.7\cdot 7.5q = 27.75q$$

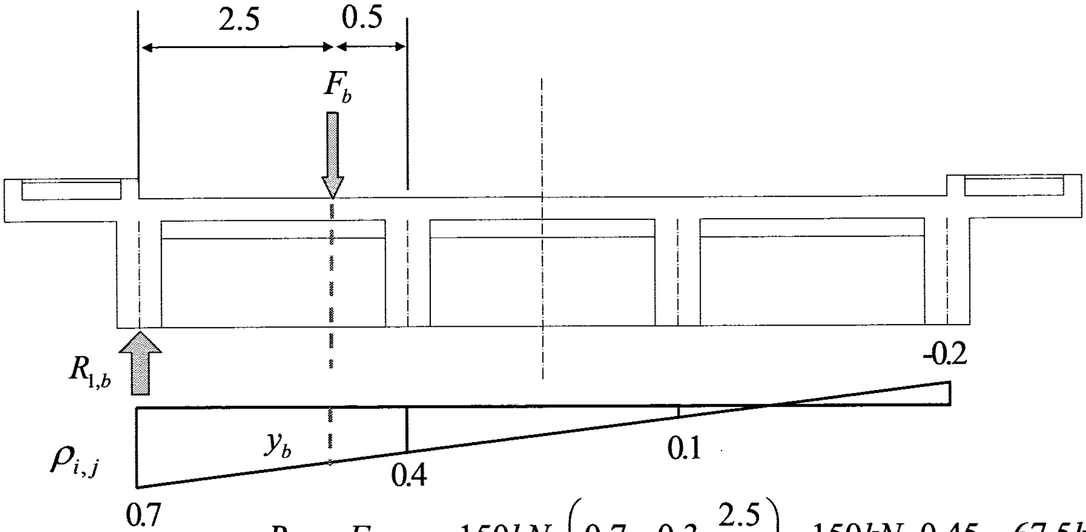
Now we see Transverse distribution of the load (where to place loads in the transverse direction)

Carriageway width = 9m
Width of each notional lane = 3m
Number of notional lanes = 3 → I don't have remaining area (it's another supposition of our exercise)

Location	Tandem system *TS*	*UDL* system
	Axle loads Q_{ik} (kN)	q_{ik} (or q_{rk}) (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

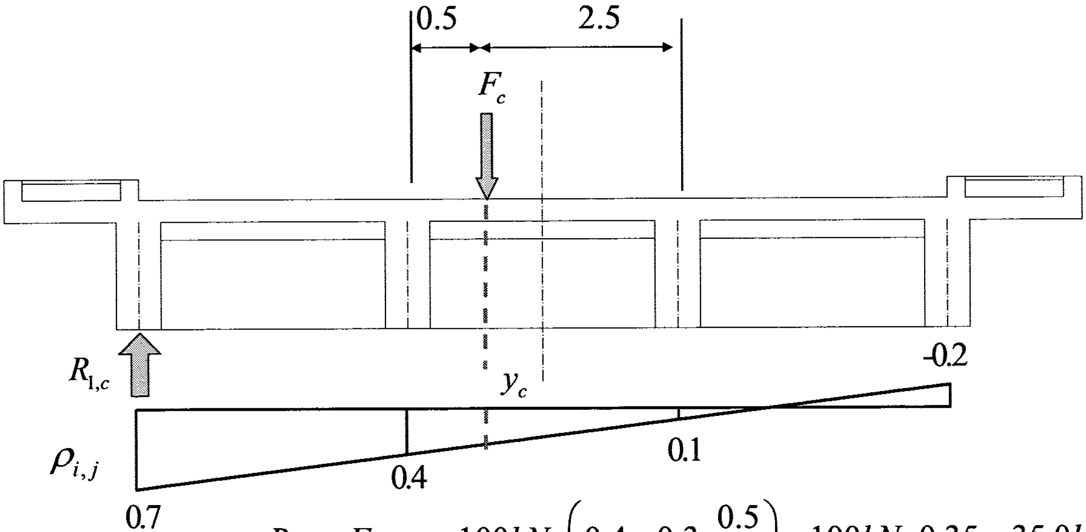
Now I do the same thing for the other force of the tandem:

Transverse distribution (concentrated loads)



$$R_{1,b} = F_b \cdot y_b = 150kN \cdot \left(0.7 - 0.3 \cdot \frac{2.5}{3}\right) = 150kN \cdot 0.45 = 67.5kN$$

Transverse distribution (concentrated loads)

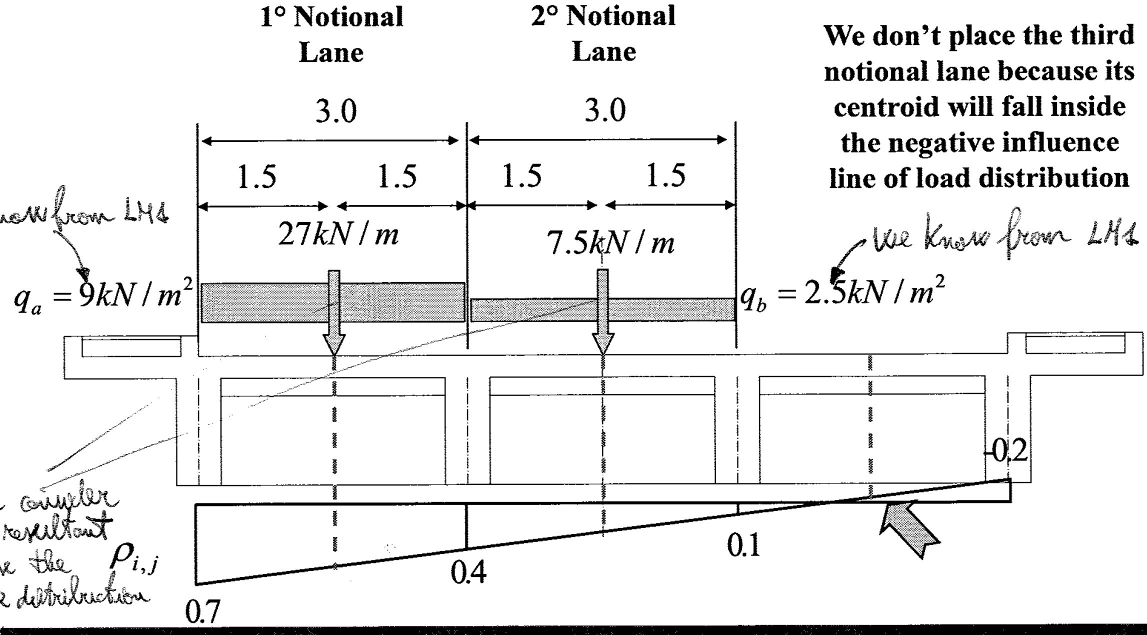


$$R_{1,c} = F_c \cdot y_c = 100kN \cdot \left(0.4 - 0.3 \cdot \frac{0.5}{3}\right) = 100kN \cdot 0.35 = 35.0kN$$

Now we do the same thing for

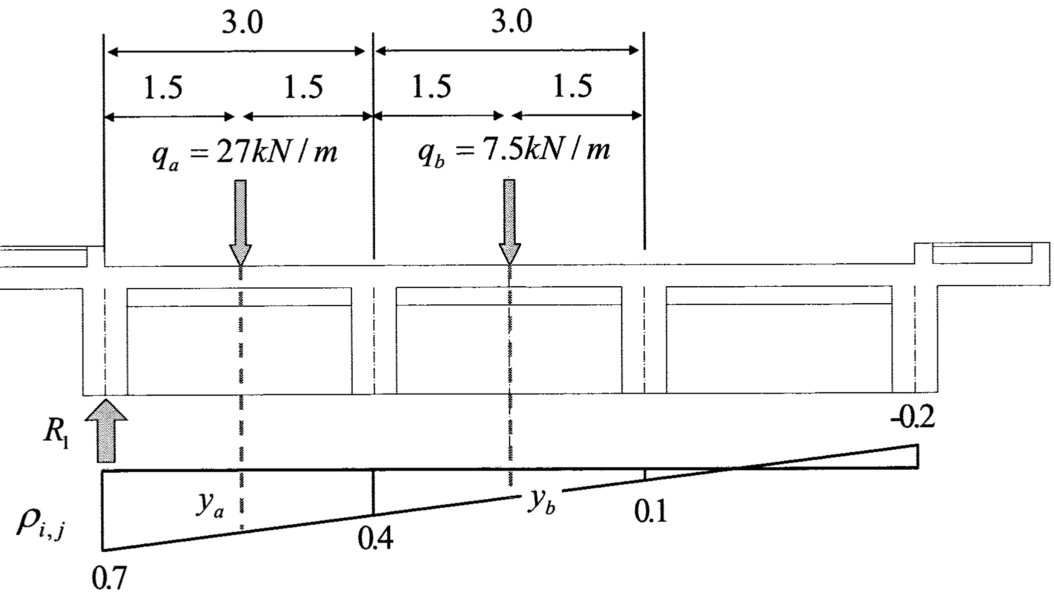
Transverse distribution (uniformly distributed loads)

We don't place the third notional lane because its centroid will fall inside the negative influence line of load distribution



We consider the resultant (because the transverse distribution function is linear; if it was curved → integral of q among the curve)

Transverse distribution (uniformly distributed loads)

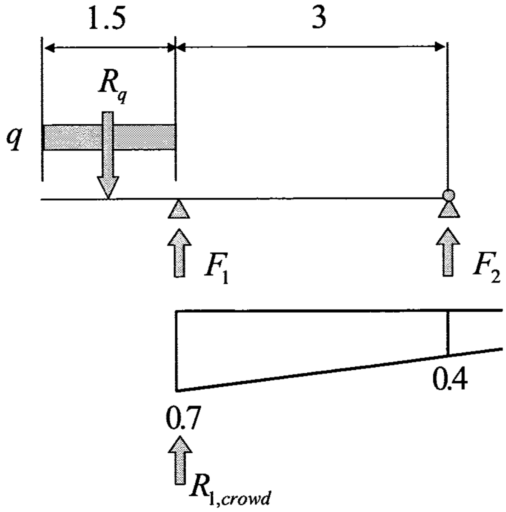


$$R_{1,u.distr.} = R_{1,a} + R_{1,b} = q_a \cdot y_a + q_b \cdot y_b = 27 \cdot 0.55 + 7.5 \cdot 0.25 = 14.85 + 1.87 = 16.7kN/m$$

Transverse distribution (crowd)

is 2,5 kN/m² if it's in combination with traffic (like now)

"per square meter"



$$q = 2.5kN/m^2$$

$$R_q = q \cdot 1.5m = 3.75kN/m$$

$$F_2 = -3.75 \cdot 0.75/3 = -0.94kN/m$$

$$F_1 = 3.75 + 0.94 = 4.69kN/m$$

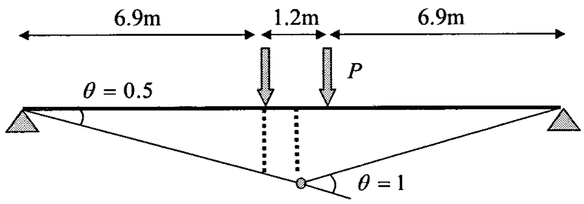
$$R_{1,crowd} = F_1 \cdot 0.7 + F_2 \cdot 0.4 = 2.9kN/m$$

Same result if we extrapolate the Curbon transverse line outside beam n1 and we multiply Rq for the value under Rq.

We have done the transverse distribution; we go back to longitudinal distribution

Bending moment in mid-span

Concentrated tandem system



slide 24/92

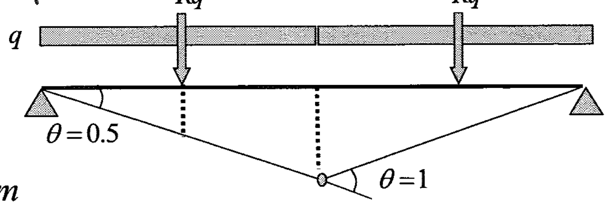
$$R_{1,concentrated} = P = 215kN$$

$$M_{S,concentrated} = 6.9R_{1,concentrated} = 1484kNm$$

slide 16/92

it comes from the uniformly distributed part of the traffic.

Uniformly distributed



slide 26/92

$$R_{1,u.distr.} = q = \underline{16.7kN/m}$$

$$M_{s,u.distr.} = 27.75R_{1,u.distr} = 463kNm$$

slide 17/92

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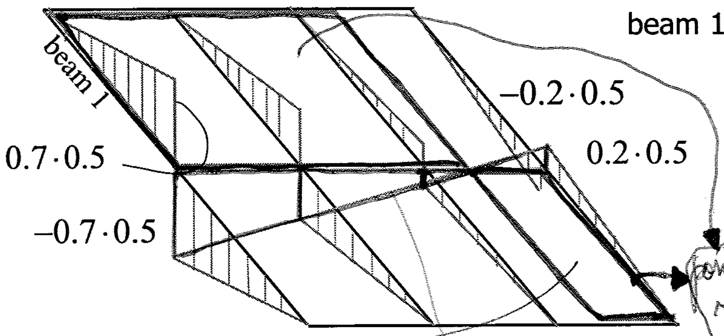
Shear in mid-span

- **Drawing influence surface**

If we modulate the two graph seen before we obtain

The blue area has to be loaded to maximize the mid-span shear in beam 1



Bending was continuos but not derivable (⇒ not C_1). Shear is not continuos (⇒ not C_0)

positive regions

(they are not continuos: they touch only in 1 point)

this is positive region because we multiply negative · negative

transverse distribution of Courbon

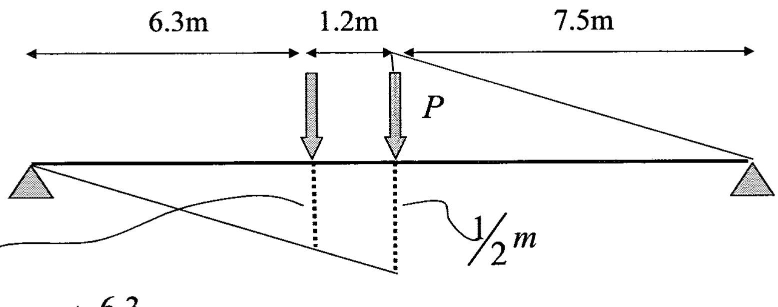
negative part of the influence line in the longitudinal direction

negative part of the transverse function (Courbon)

It was unpredictable, before drawing the influence surface, imagine this portion of blue area

Shear in mid-span

- **Variable concentrated traffic load**



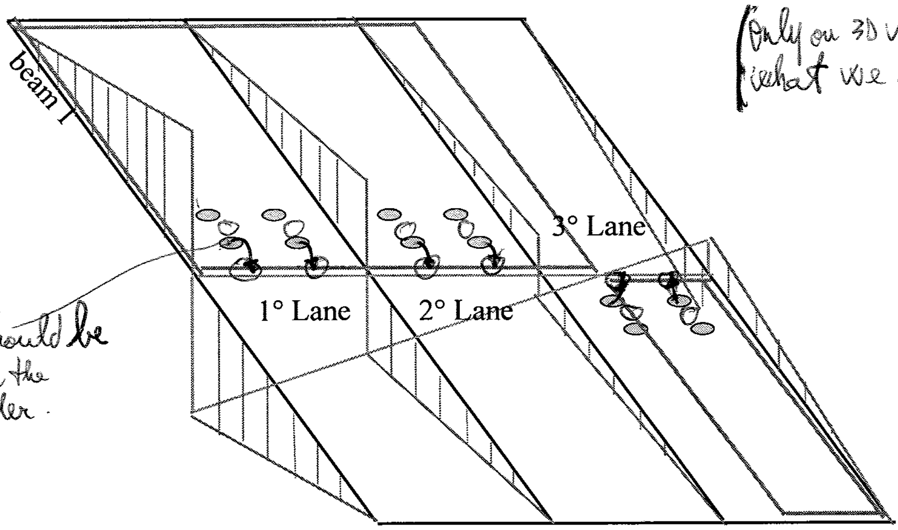
$$\frac{1}{2}\cdot\frac{6.3}{7.5}=0.42m$$

$$\Rightarrow V_{S,P}=(0.5+0.42)\cdot P=0.92P$$

Transverse distribution (concentrated loads)

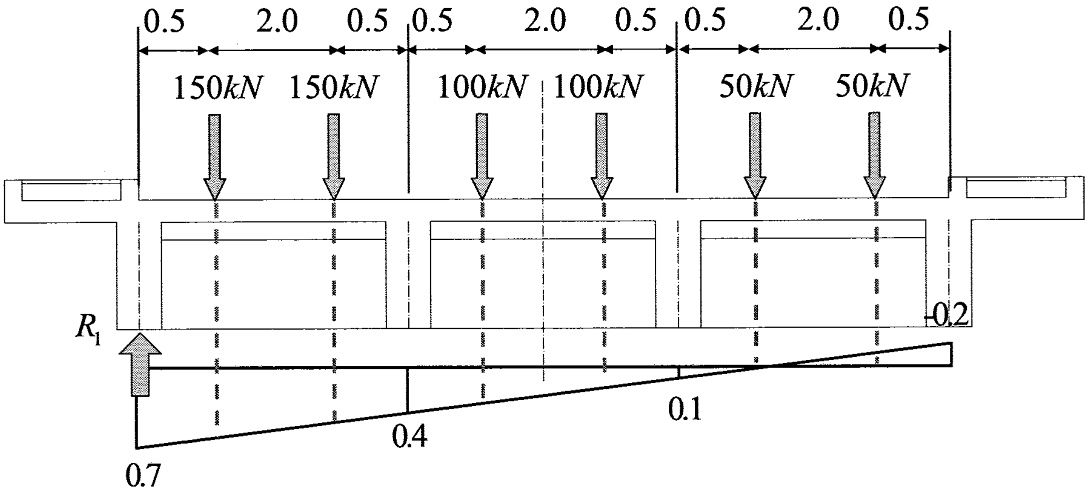
Longitudinal location of previously seen concentrated loads

(Only on 3D view we understand what we are doing)



they should be applied on the border.

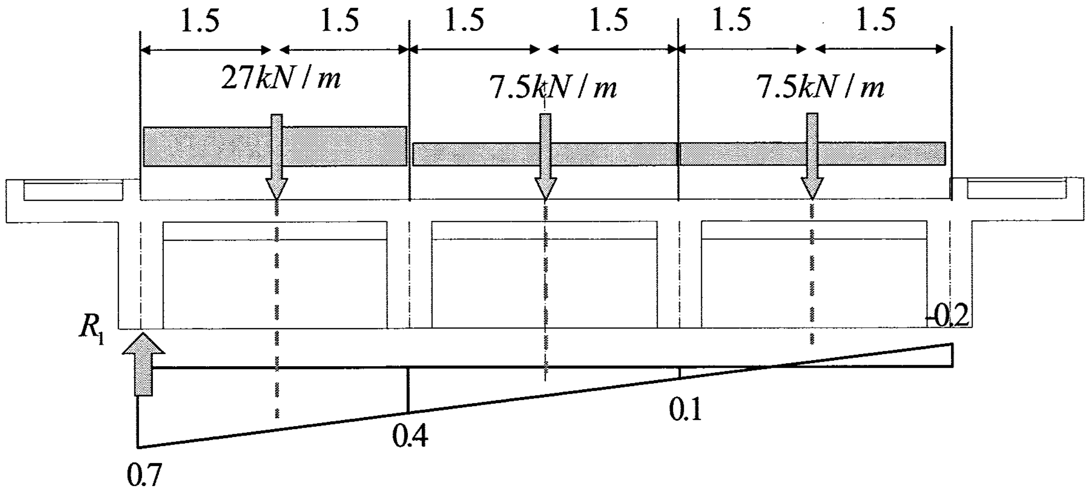
Transverse distribution (concentrated loads)



$$R_{1,concentrated} = 150\cdot(0.65+0.45)+100\cdot(0.35+0.15)+50\cdot(-0.05+0.15)=$$
$$=165+50+5=220kN$$

in this image they seem the opposite but remember that we are in the other region (look at slide 37)

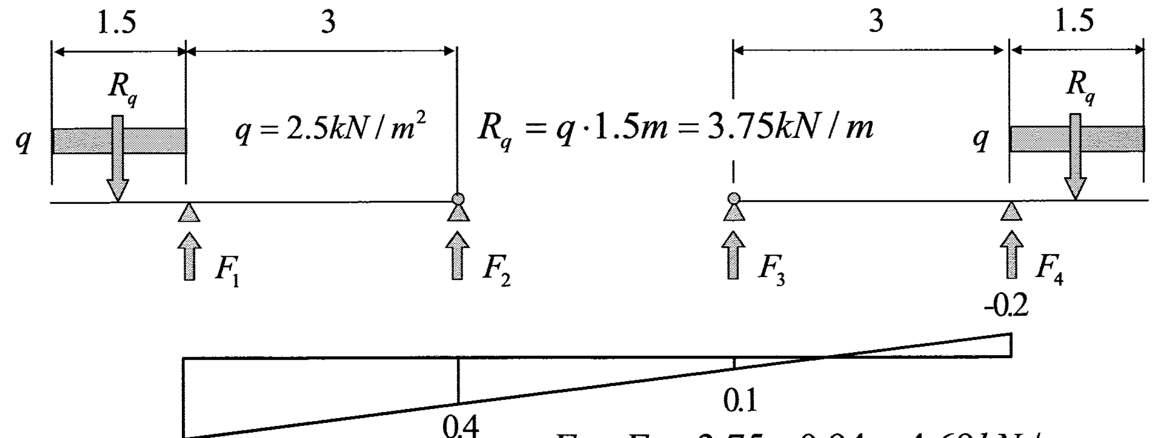
Transverse distribution (uniformly distributed loads)



$$R_1, u.distr. = 27 \cdot 0.55 + 7.5 \cdot 0.25 + 7.5 \cdot 0.05 = 17.1kN/m$$

Transverse distribution (crowd)

(look at next slide to see where crowd is placed, according to the influence surfaces)



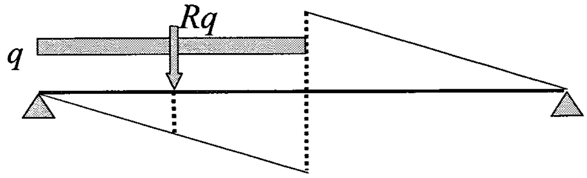
$$F_1 = F_4 = 3.75 + 0.94 = 4.69kN/m$$
$$F_2 = F_3 = -3.75 \cdot 0.75/3 = -0.94kN/m$$

$$R_{1,crowd} = F_1 \cdot 0.7 + F_2 \cdot 0.4 + F_3 \cdot -0.1 + F_4 \cdot 0.2 =$$
$$= 4.69 \cdot 0.9 - 0.94 \cdot 0.3 = 3.94kN/m$$

they become opposite because we are in the other section

Shear in mid-span

Crowd



$$R_{1,crowd} = q = 3{,}94\, kN/m$$

$$V_{s,crowd} = 1.875 R_{1,crowd} = 7{,}38\, kN$$

Total shear from vertical traffic actions

$$V_{s,Vtraffic} = V_{s,concentrated} + V_{s,u.distr.} + V_{s,crowd} = 202 + 32 + 7 = 241\, kN$$

Non traffic actions: WIND

Location: Piemonte 250m o.s.l.

Wind referring speed $v_b = v_{b,0} = 25 \dfrac{m}{s}$

Kinetic referring pressure $q_b = \dfrac{1}{2}\rho v_b^2 = \dfrac{1}{2} 1.25 \cdot 25^2 = 391 \dfrac{N}{m^2}$

Geografic zone 1
Terrain roughness class D (open land without obstacles)
Site exposition category II $k_r = 0.19$
$z_0 = 0.05m$
$z_{min} = 4$ m

We imagine the building to be at the land level.
Maximum height of the structure z=3m

Vertical position of the centroid of the deck

Total mass of the bridge

1. Longitudinal beams $Mg_{1lb} = 225kN \cdot 4 = 900kN$
2. Transverse beams $Mg_{1tb} = 56kN \cdot 4 = 224kN$
3. Slab $Mg_{1s} = 1125kN$

Total $Mg = 900 + 224 + 1125 = 2249kN$

Static moment of bridge masses with respect to the intrados blue line in the next slide

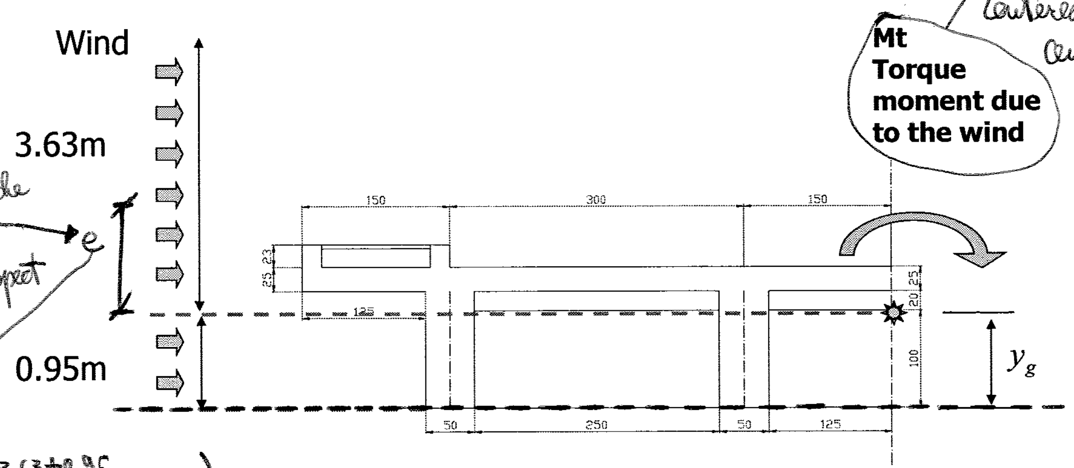
1. Longitudinal beams $Sg_{1lb} = 225kN \cdot 4 \cdot 0.6m = 540kNm$
2. Transverse beams $Sg_{1tb} = 56kN \cdot 4 \cdot 0.5m = 112kNm$
3. Slab $Sg_{1s} = 1125kN \cdot 1.325m = 1491kNm$

Total $Sg = 540 + 112 + 1491 = 2143kNm$

Vertical position of the centroid of the deck

because the wind is not vertically centered on the centroid.

eccentricity of the resultant of wind action respect the centroid of the deck e



$$\left(\frac{3{,}63+0{,}95}{2} - 0{,}95\right)$$

Vertical position of the centroid $y_g = \frac{S_g}{M_g} = \frac{2143}{2249} = 0.95m$

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ULS combination

Bending moment in mid-span of beam 1

$$M_{S,tot} = 1.35 M_{S,perm} + 1.35 M_{S,traffic} + 1.50 M_{S,wind} =$$
$$= 1.35 \cdot 1518 + 1.35 \cdot 2028 + 1.50 \cdot 14 = 4808 kNm$$

Shear in mid-span of beam 1

$$V_{S,tot} = 1.35 V_{S,perm} + 1.35 V_{S,traffic} + 1.50 V_{S,wind} =$$
$$= 1.35 \cdot 0 + 1.35 \cdot 239 + 1.50 \cdot 0 = 323 kN$$

We calculated M and V for longitudinal beams, using Courbon.

Pay attention:

It's not possible to evaluate the internal actions in the transverse beams using **Courbon**, because Courbon hypothesis doesn't locate transverse beams in a specific position but smears them in the whole length of the deck.

"spalma"

the deck is so rigid, so all the deck distributes the load and no load goes to transverse beams.

If we want to know the internal actions in the transverse beams we have to use the **Engesser** model.

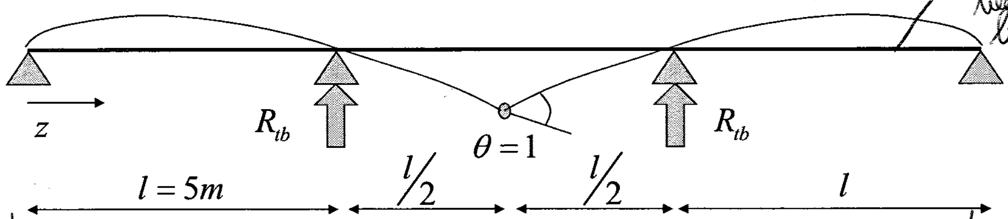
We don't know where to put the load, so we have to draw the influence line of M for the mid-span section.

Bending moment in mid-span

- **Drawing influence surface**

One dimensional influence line for longitudinal beam (continuous on transverse beams) Influence line of a beam 2 times iperstatic

1st hypothesis of Engesser: consider this longitudinal beam as rigidly supported by transverse beams.



we have to know how to calculate these formulas

$$y_{a1}(z) = \begin{cases} \dfrac{z}{10}\left(\dfrac{z^2}{l^2} - 1\right) & for\ 0 \le z \le l \\ \dfrac{1}{5}\left(\dfrac{3z^2}{2l} - 2z + \dfrac{l}{2}\right) & for\ l \le z \le \dfrac{3}{2}l \end{cases}$$

reaction that arise if you put that concentrated rotation

$$R_{tb} = \frac{3EI_b}{5l^2}$$

does not depend on any kind of load (of course: it's an influence line)

The difference between Courbon and Engesser is: Engesser takes into account the discontinuity into transverse repartition due to the fact that transverse beams are present only in some location.

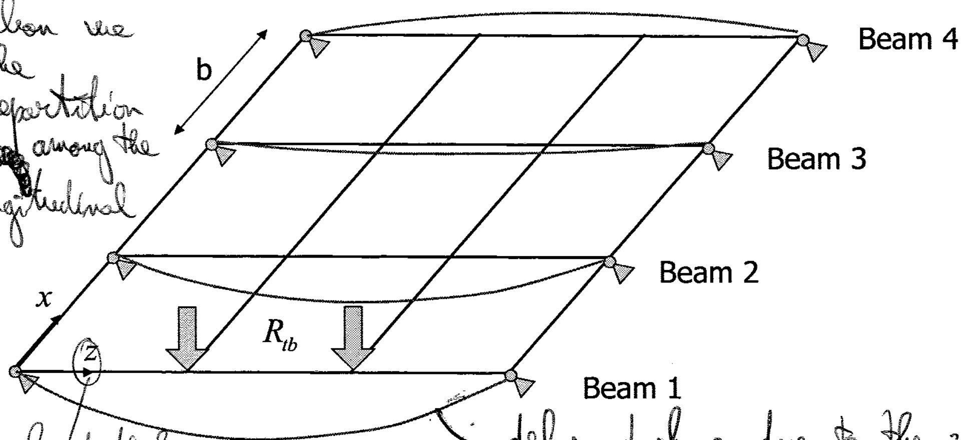
Bending moment in mid-span

- **Drawing influence surface**

changed in sign on the whole structure and we Courbon theory to do the transverse repartition on the other beams

We apply the virtual reactions R_{tb} on the girder and we calculate with Courbon theory the global deformation of the deck.

With Courbon we calculate the transverse repartition of R_b among the other longitudinal beams.



longitudinal direction

deformed shape due to the 2 reactions

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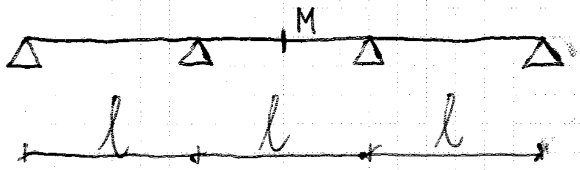
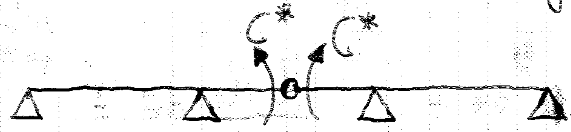
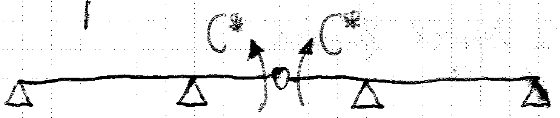
CALCOLO EQUAZIONE DELLA LINEA DI INFLUENZA DELLA SLIDE 57/92 - (ENGESSER METHOD)



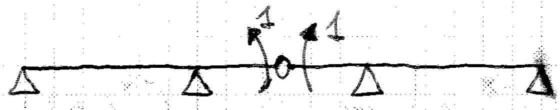
Inserisco una cerniera nella sezione M e applico due coppie C^* tali da procurare una rotazione relativa unitaria negativa nella sezione M: $(\theta_{rel} = -1)$



Utilizzo il P.L.V. per determinare C^*:



SISTEMA 0

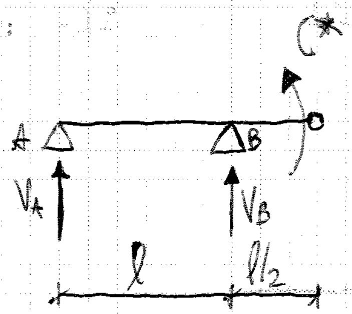


SISTEMA 1

sistema fittizio che mi serve per compiere lavoro con la rotazione unitaria relativa e negativa.

P.L.V.: $\displaystyle\int_0^{3l} \frac{M_0 M_1}{EI}\, dz = -1$

Calcolo M_0:

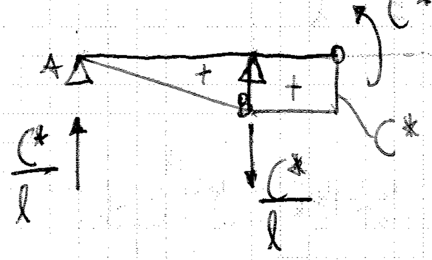


$\uparrow^{+}\ V_A + V_B = 0$

$\circlearrowleft^{+}(A):\ +V_B \cdot l + C^* = 0 \Rightarrow$

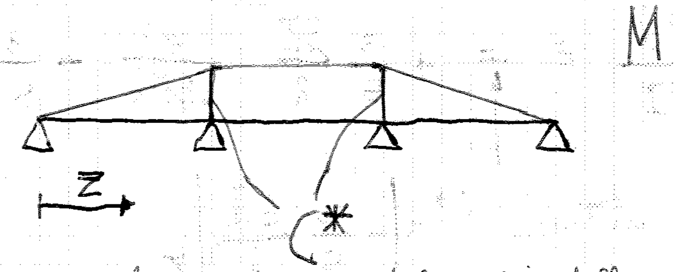
$\Rightarrow V_B = -\dfrac{C^*}{l}$

$V_A = \dfrac{C^*}{l}$



M_0

Il diagramma di momento flettente sarà:



Ora determino analiticamente la deformata della struttura caricata dalle due coppie C^*, andando ad integrare 2 volte la curvatura:

per $0 \leq z \leq l \Rightarrow M_x(z) = -\frac{C^*}{l} \cdot z$

$$\frac{d^2 y_1}{dz^2} = -\frac{M_x}{EI} = \frac{C^*}{lEI} z$$

$$\frac{dy_1}{dz} = \frac{C^*}{lEI}\frac{z^2}{2} + C_1$$

$$y_1 = \frac{C^*}{lEI}\frac{z^3}{6} + C_1 z + C_2$$

per $l \leq z \leq \frac{3}{2} l \Rightarrow M_x(z) = -C^*$

$$\frac{d^2 y_2}{dz^2} = -\frac{M_x}{EI} = \frac{C^*}{EI} z$$

$$\frac{dy_2}{dz} = \frac{C^*}{EI}\frac{z^2}{2} + C_3$$

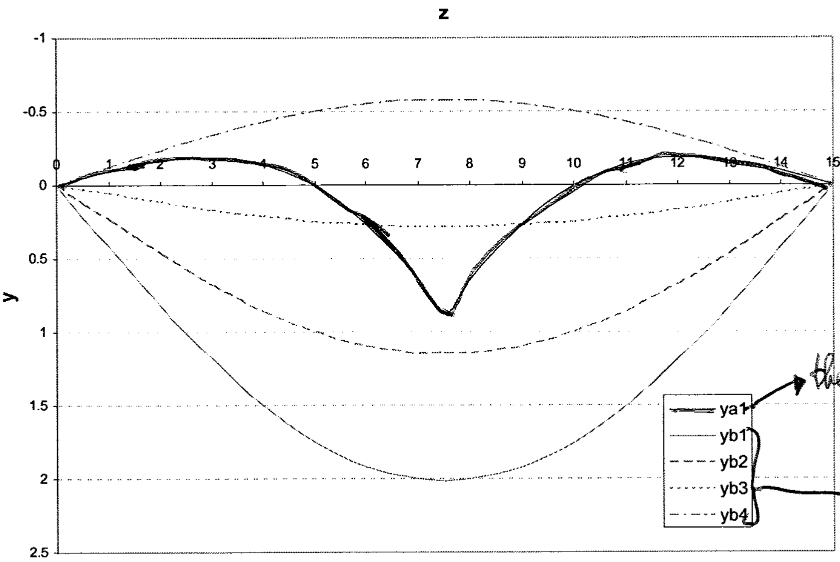
$$y_2 = \frac{C^*}{EI}\frac{z^3}{6} + C_3 z + C_4$$

Applico le c.c. per trovare le costanti di integrazione:

$$y_1(z=0) = 0 \Rightarrow 0 = 0 + 0 + C_2 \Rightarrow \boxed{C_2 = 0}$$

$$y_1(z=l) = 0 \Rightarrow 0 = \frac{C^*}{lEI}\frac{l^3}{6} + C_1 l = 0 \Rightarrow \boxed{C_1 = -\frac{C^* l}{6EI}}$$

- **Drawing influence surface**

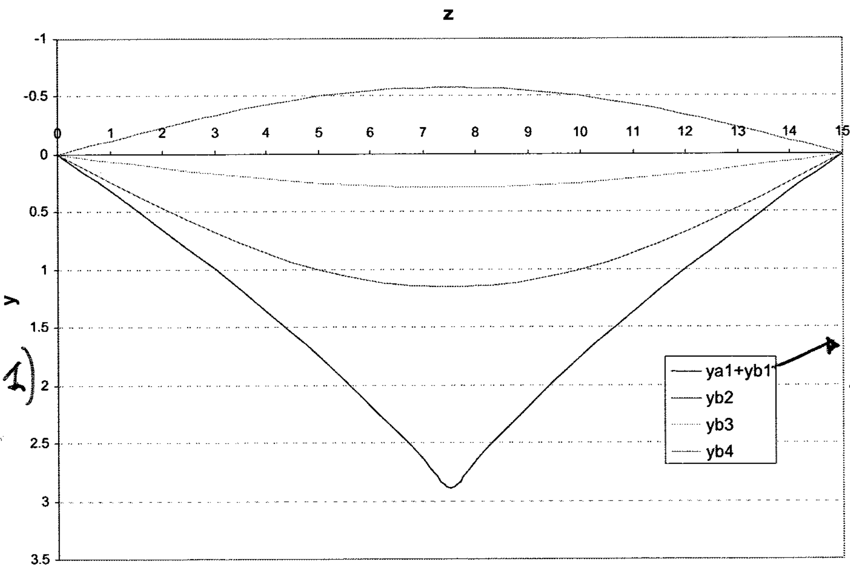


the same of slide 57/92

slide 58/92

- **Drawing influence surface**

The final surface has discontinuity (due to beam 1)



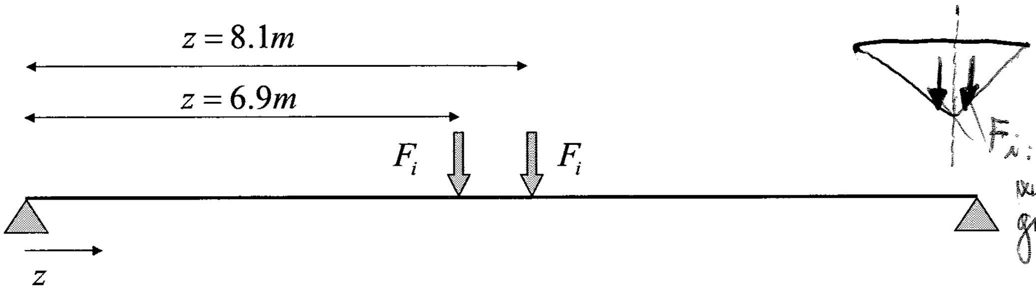
SUPERPOSITION of EFFECTS

1. We have to distribute on the longitudinal beams the vertical loads acting on the slab using the simply supported schemes seen before

We transfer loads from the real position to the longitudinal beams

$$F_1 = 150kN$$
$$F_2 = 150 + 100 = 250kN$$
$$F_3 = 100kN$$

2. Once the loads are on the beams we can use the influence lines shown in slide 61 to calculate the bending moment in mid-span.



F_i: we place F_i where they give the maximum effect

for simmetry

$$y_{a,1}(6.9) + y_{b,1}(6.9) = y_{a,1}(8.1) + y_{b,1}(8.1) = 0.6 + 2.0 = 2.60$$
$$y_{b,2}(6.9) = y_{b,2}(8.1) = 1.14$$
$$y_{b,3}(6.9) = y_{b,3}(8.1) = 0.285$$

$$M_{s,F1} = 2 \cdot \left[y_{a,1}(6.9) + y_{b,1}(6.9) \right] \cdot F_1 = 2 \cdot 2.60 \cdot 150 = 780kNm$$
$$M_{s,F2} = 2 \cdot y_{b,2}(6.9) \cdot F_2 = 2 \cdot 1.14 \cdot 250 = 570kNm$$

bending moment in beam 1 due to the forces applied on beam 3

$$M_{s,F3} = 2 \cdot y_{b,3}(6.9) \cdot F_3 = 2 \cdot 0.285 \cdot 100 = 57kNm$$

total bending moment on beam 1 due to concentrated forces

$$M_{S,concentrated} = M_{S,F1} + M_{S,F2} + M_{S,F3} = 780 + 570 + 57 = 1407kNm$$

With Courbon model it was

$$M_{S,concentrated} = 1484kNm$$

5% difference

it's smaller than Courbon because the bridge is short and we have a lot of transverse beams (general you do not put more than 4 transverse beam for each span)
For very rigid bridges Courbon and Engesser produce more or less the same value (like now). For longer bridges

"a" system

→ expressions of the slide 54/92

$$y_{a1}(z)=\begin{cases}\dfrac{z}{10}\left(\dfrac{z^2}{l^2}-1\right) & \text{for } 0\le z\le l\\[2ex] \dfrac{1}{5}\left(\dfrac{3z^2}{2l}-2z+\dfrac{l}{2}\right) & \text{for } l\le z\le \dfrac{3}{2}l\end{cases}$$

integral over the (blue) line of slide 61/92.

$$\int_0^{3l} q\cdot y_{a1}(z)dz=2\left(\int_0^{l} q\cdot\frac{z}{10}\left(\frac{z^2}{l^2}-1\right)dz+\int_l^{3l/2} q\cdot\frac{1}{5}\left(\frac{3z^2}{2l}-2z+\frac{l}{2}\right)dz\right)$$

$$=2q\left(\int_0^{l}\frac{z}{10}\left(\frac{z^2}{l^2}-1\right)dz+\int_l^{3l/2}\frac{1}{5}\left(\frac{3z^2}{2l}-2z+\frac{l}{2}\right)dz+\right)$$

$$=2q\left(-\frac{l^2}{40}+\frac{3l^2}{80}\right)=\frac{1}{40}q\cdot l^2=0.625q$$

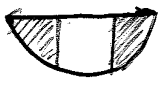
we calculate for an half → multiply by 2 for simmetry.

"b" system

$$y_{b,i}(z)=\rho_{1,i}\cdot\begin{cases}\dfrac{1}{10l^2}\left(-z^3+6l^2z\right)\ (1) & \text{for } 0\le z\le l\\[2ex] \dfrac{1}{10l^2}\left(-z^3+(z-l)^3+6l^2z\right) & \text{for } l\le z\le 2l\\[2ex] \dfrac{1}{10l^2}\left(-z^3+(z-l)^3+(z-2l)^3+6l^2z\right)\ (2) & \text{for } 2l\le z\le 3l\end{cases}$$

because the integral of the (1) is equal to the integral (area) of (2) (look at slide 58/92)

$$\int_0^{3l} q\cdot y_{b,i}(z)dz=\rho_{1,i}\left(2\int_0^{l} q\cdot\frac{1}{10l^2}\left(-z^3+6l^2z\right)dz+\int_l^{2l} q\cdot\frac{1}{10l^2}\left(-z^3+(z-l)^3+6l^2z\right)dz+\right)$$



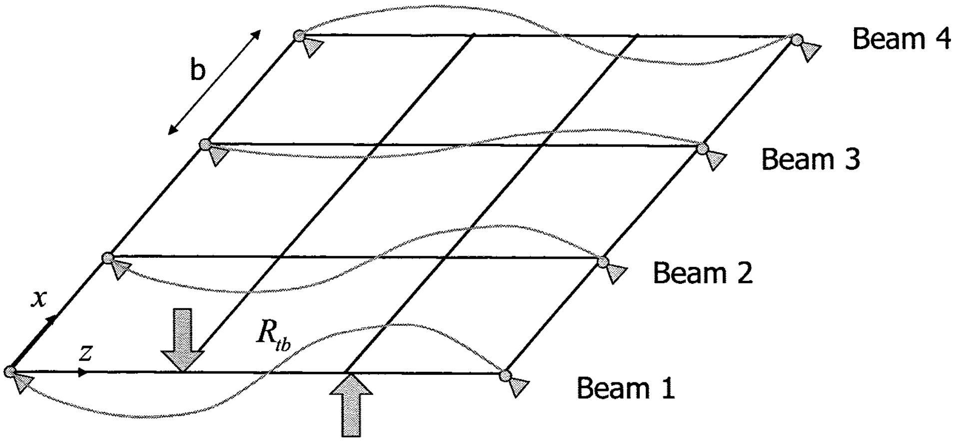
$$=\rho_{1,i}\frac{q}{10l^2}\left(2\int_0^{l}\left(-z^3+6l^2z\right)dz+\int_l^{2l}\left(-z^3+(z-l)^3+6l^2z\right)dz+\right)$$

$$=\rho_{1,i}\frac{q}{10l^2}\left(\frac{11}{2}l^4+\frac{11}{2}l^4\right)=\rho_{1,i}q\frac{11}{10}l^2$$

Shear in mid-span

- **Drawing influence surface**

  We apply the virtual reactions R_{tb} on the girder and we calculate with Courbon theory the global deformation of the deck.



- **Drawing influence surface**

  The equation of the surface drawn in the previous page is

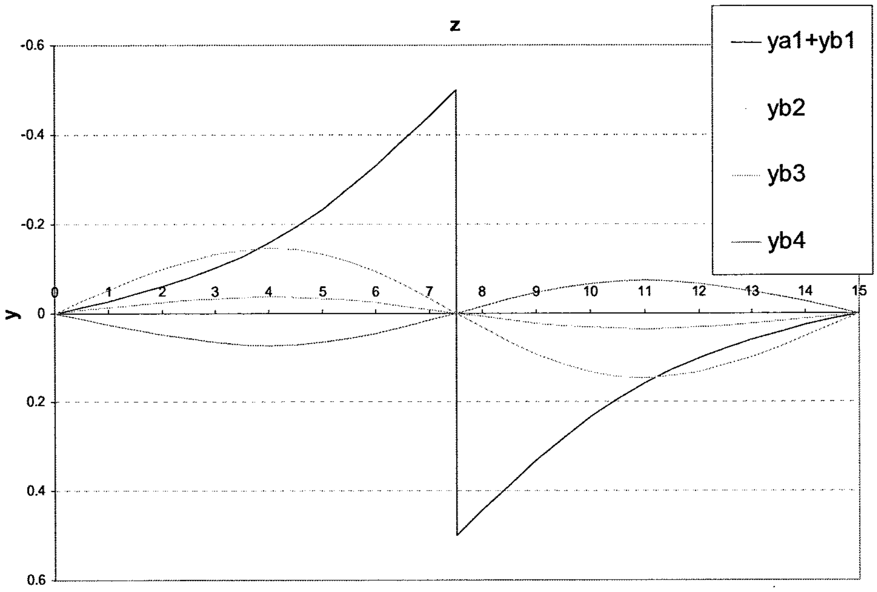
$$y_b(z,x)=\left(0.7-0.9\frac{x}{3b}\right)\cdot\begin{cases}\frac{2}{l^3}\left(\frac{z^3}{6}-\frac{l^2 z}{3}\right) & \text{from } z=0 \text{ to the first force} \quad \textit{for } 0\le z\le l\\ \frac{2}{l^3}\left(\frac{z^3}{6}-\frac{(z-l)^3}{2}-\frac{l^2 z}{3}\right) & \text{from 1st to the 2nd force} \quad \textit{for } l\le z\le 2l\\ \frac{2}{l^3}\left(\frac{z^3}{6}-\frac{(z-l)^3}{2}-\frac{(z-2l)^3}{2}-\frac{l^2 z}{3}\right) & \text{from the 2nd force to the end} \quad \textit{for } 2l\le z\le 3l\end{cases}$$

Transverse direction

Longitudinal direction

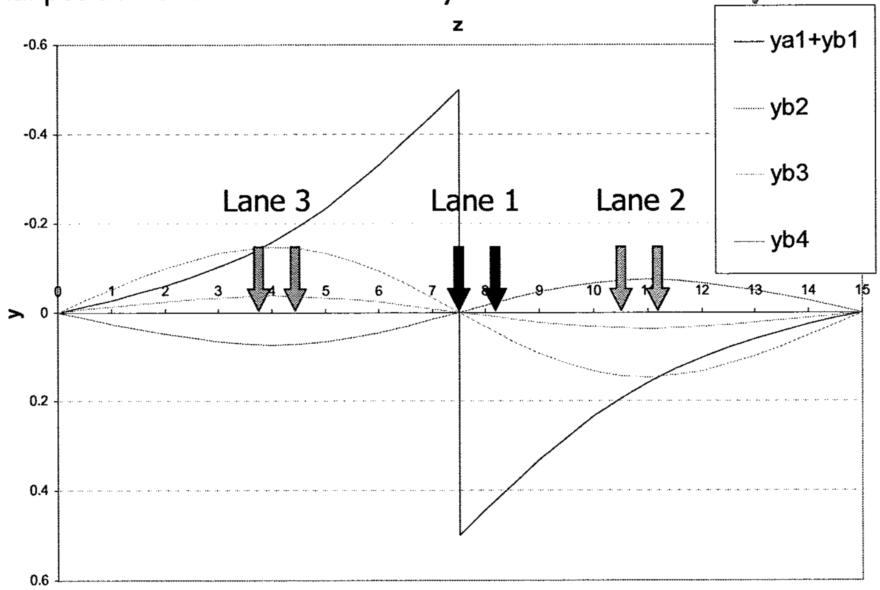
Courbon transverse repartition function

- **Drawing influence surface**



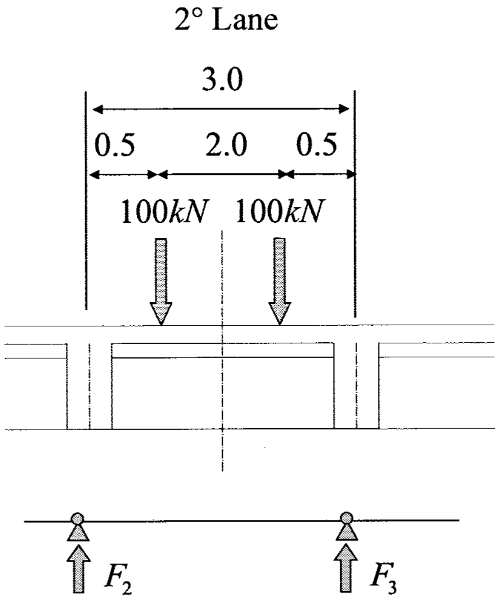
Concentrated loads

Longitudinal position of the three tandem systems : / is not so easy as it was for bending moment. (look at the next slide)



we have to calculate the position of the maximum of the green and blue curves (look at 78/92)

Concentrated loads : 2° lane



The z corresponding to the maximum value of $y_{b,2}(z)$ has to be calculated. For sake of simplicity it is done for 0<z<7.5 and then used for the symmetric points with z>7.5m.

$$y_{b,2}(z) = 0.4\frac{2}{l^3}\left(\frac{z^3}{6} - \frac{l^2 z}{3}\right)$$

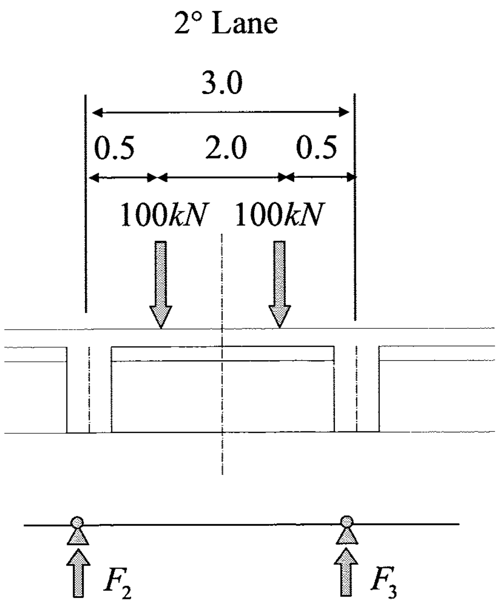
$$\frac{\partial y_{b,2}(z)}{\partial z} = 0 \Rightarrow \frac{\partial}{\partial z}\left[0.4\frac{2}{l^3}\left(\frac{z^3}{6} - \frac{l^2 z}{3}\right)\right] = 0$$

$$z = \sqrt{\frac{2}{3}}l = 4.08m$$

$$z_{t1} = 4.08 - 0.6 = 3.48m$$
$$z_{t1} = 4.08 + 0.6 = 4.68m$$

to check by eyes, look at slide 78/92.

Concentrated loads : 2° lane



$$y_{b,2}(11.52) = -y_{b,2}(3.48) = 0.140$$
$$y_{b,2}(10.32) = -y_{b,2}(4.68) = 0.140$$

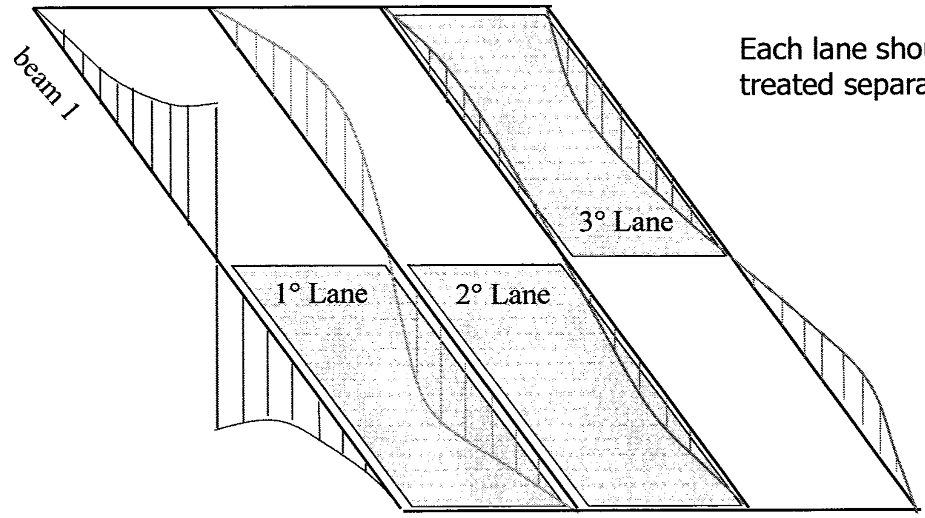
$$y_{b,3}(11.52) = -y_{b,3}(3.48) = 0.035$$
$$y_{b,3}(10.32) = -y_{b,3}(4.68) = 0.035$$

$$F_2 = F_3 = 100kN$$

$$V_{c,2} = 2 \cdot F_2 (0.140 + 0.035)$$
$$= 0.350 \cdot 100 = 35kN$$

Uniformly distributed loads

Location of uniformly distributed loads



Each lane should be treated separately

Again: we have uniformly distributed load → we have to integrate the influence line!

"a" system

$$y_{a1}(z)=\begin{cases}\dfrac{z}{3l}\left(1-\dfrac{z^2}{l^2}\right) & for\ 0\le z\le l\\[2ex] \dfrac{z}{3l}\left(1-\dfrac{z^2}{l^2}\right)+\left(\dfrac{z}{l}-1\right)^3 & for\ l\le z\le \dfrac{3}{2}l\end{cases}$$

N.B. For sake of simplicity the following calculations are done for 0<z<7.5 and then used for the symmetric values with z>7.5m.

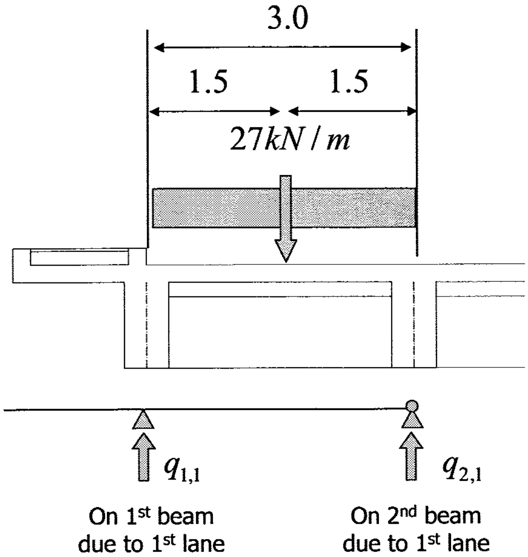
$$\int_0^{3l/2} q\cdot y_{a1}(z)dz=\int_0^l q\cdot\frac{z}{3l}\left(1-\frac{z^2}{l^2}\right)dz+\int_l^{3l/2} q\cdot\left[\frac{z}{3l}\left(1-\frac{z^2}{l^2}\right)+\left(\frac{z}{l}-1\right)^3\right]dz$$

$$=q\left(\int_0^l \frac{1}{3l}\left(z-\frac{z^3}{l^2}\right)dz+\int_l^{3l/2}\frac{1}{3l}\left(z-\frac{z^3}{l^2}\right)+\left(\frac{z}{l}-1\right)^3 dz\right)$$

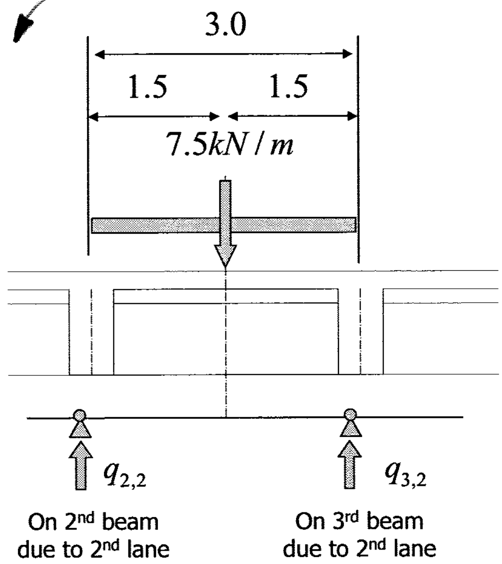
$$=ql\left(\frac{1}{12}-\frac{25}{192}+\frac{1}{64}\right)=-\frac{1}{32}ql=-\frac{1}{32}q5=-0.156q$$

Distributed loads

1° Lane



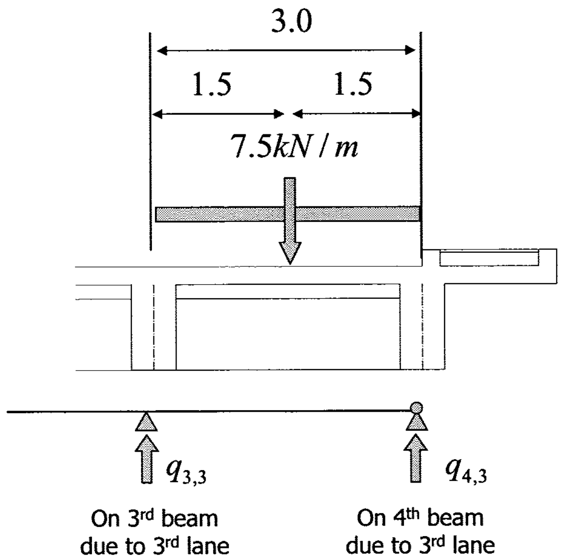
2° Lane



$$q_{1,1} = q_{2,1} = 27/2 = 13.5kN/m \qquad q_{2,2} = q_{3,2} = 7.5/2 = 3.75kN/m$$

Distributed loads

3° Lane



$$q_{3,3} = q_{4,3} = 7.5/2 = 3.75kN/m$$

Distributed loads : total shear in mid-span

We add the contribution of the three lanes

$$V_{S,distributed} = V_{d,1,1} + V_{d,2,1} + V_{d,2,2} + V_{d,3,2} + V_{d,3,3} + V_{d,4,3} =$$
$$= 18.35 + 9.29 + 2.58 - 0.65 + 0.65 + 1.29 = 31.5kN$$

With Courbon model it was

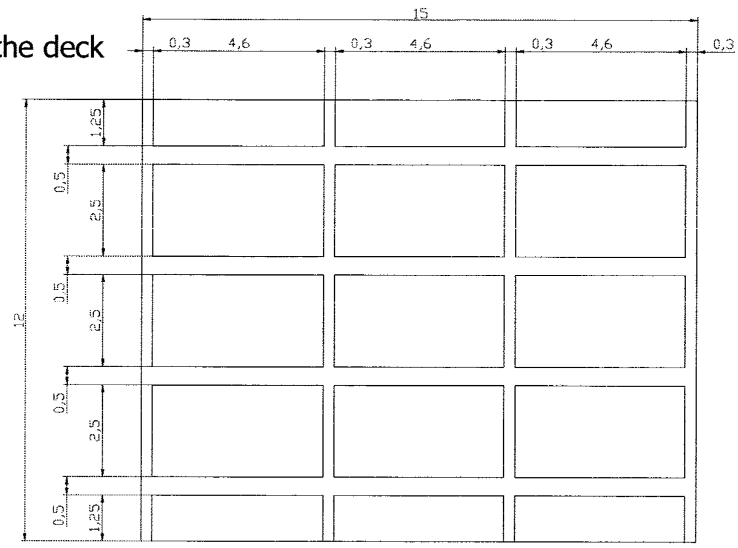
$V_{S,concentrated} = 32kN$ 0% difference

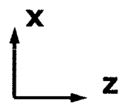
The missing thing of this lesson is that we should calculate with Engesser, the internal actions on the transverse beams (with Courbon we can't do it, only with Engesser we can).

1.Referring system

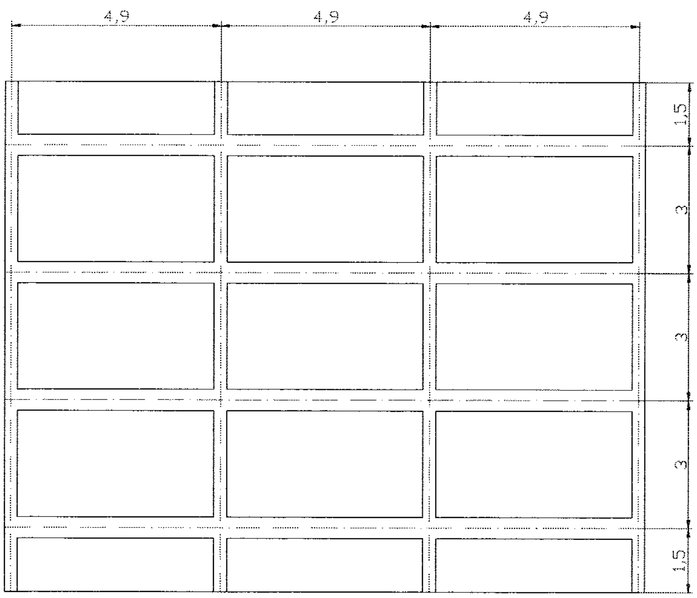
X Transverse direction of the deck
Y Vertical axis
Z Longitudinal direction of the deck

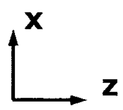
Girder deck dimensions - Bottom view





Girder deck dimensions – Interaxis between main members





3. Effective flange width of longitudinal beams

5.3.2 Geometric data

5.3.2.1 Effective width of flanges (all limit states)

(3) The effective flange width b_{eff} for a T beam or L beam may be derived as:

$$b_{eff} = \sum b_{eff,i} + b_w \leq b$$

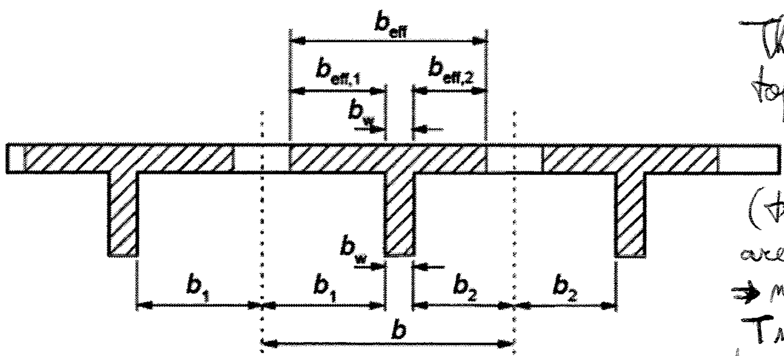
where

$$b_{eff,i} = 0{,}2b_i + 0{,}1l_0 \leq 0{,}2l_0$$

and

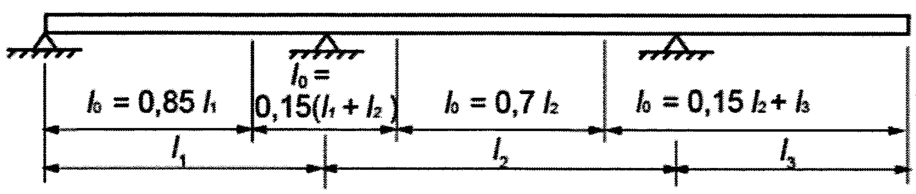
$$b_{eff,i} \leq b_i$$

(4) For structural analysis, where a great accuracy is not required, a constant width may be assumed over the whole span. The value applicable to the span section should be adopted.



The slab becomes the top flange of the longitudinal beams.
(the longitudinal beams are connected to the slab → not rectangular, but T shape. So we have to calculate how much of the slab can be considered as the top flange of the beams (and give a contribute to the resistance))

l_0 = distance between points of zero bending moment



A T-section has small torque inertia.

Torque inertia moment $I_t = I_{t,web} + \frac{1}{2} I_{t,flange}$ *

For a rectangular section $I_t = \frac{b \cdot a^3}{\beta}$

$$n = \frac{b}{a} \qquad \beta \cong \left(1+\frac{1}{n^2}\right)\left[3.56 - 0.56\left(\frac{n-1}{n+1}\right)^2\right]$$

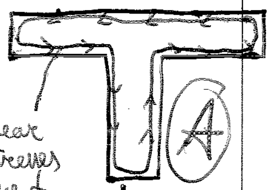
a

b

$$n_{web} = \frac{1.2}{0.5} = 2.4 \Rightarrow \beta_{web} = 4.07 \Rightarrow I_{t,web} = \frac{1.2 \cdot 0.5^3}{4.07} = 3.69E-02m^4$$

$$n_{flange} = \frac{3}{0.25} = 12 \Rightarrow \beta_{flange} = 3.18 \Rightarrow I_{t,flange} = \frac{3 \cdot 0.25^3}{3.18} = 1.47E-02m^4$$

$$I_t = (3.69 + 1.47/2)E-02 = 4.43E-02m^4$$



(A)

shear stresses due to torque (Bredt theory)

* There are not these vertical forces anymore ⇒ we don't have this contribution to the torque moment, so we assume that $I_{t,flange}$ is $\frac{1}{2} I_{t,flange}$ because of the continuity of the structure

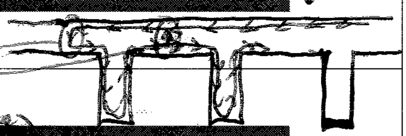


image (A)

5. Cross section properties of transverse beams

are more simple because they are rectangular (they are not connected to the slab)

Area

$$A_c = b_w \cdot h_w = 0.3 \cdot 1.0 = 0.3m^2$$

bending inertia moments

$$I_{yy} = \frac{0.3 \cdot 1.0^3}{12} = 0.025m^4$$

$$I_{xx} = \frac{0.3^3 \cdot 1}{12} = 0.0075m^4$$

torque inertia moments

$$n_{transv} = \frac{1}{0.3} = 3.33 \Rightarrow \beta_{transv} = 3.70 \Rightarrow I_{t,transv} = \frac{1 \cdot 0.3^3}{3.70} = 7.29E-03m^4$$

Bending moment in mid-span of beam 1

For concentrated loads

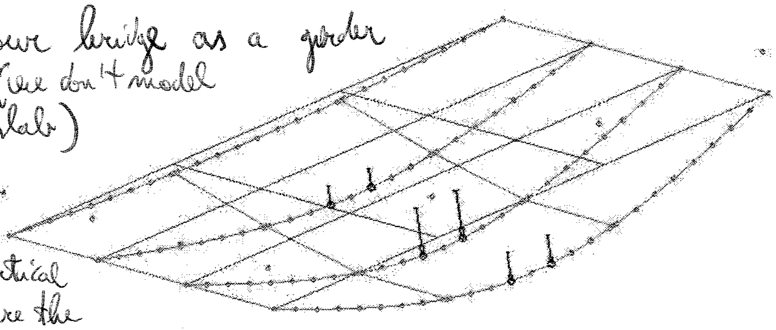
$$M_{S,concentrated} = 1202kNm$$ (it's a good result!)

With Courbon model it was $M_{S,concentrated} = 1484kNm$ 23% difference

With Engesser model it was $M_{S,concentrated} = 1407kNm$ 17% difference

We model our bridge as a girder of beams (we don't model as a slab)

this point is used only to define a vertical plane where there are the beams



and this point → this point is auxiliary node (used to put axis is vertical)

Bending moment in mid-span of beam 1

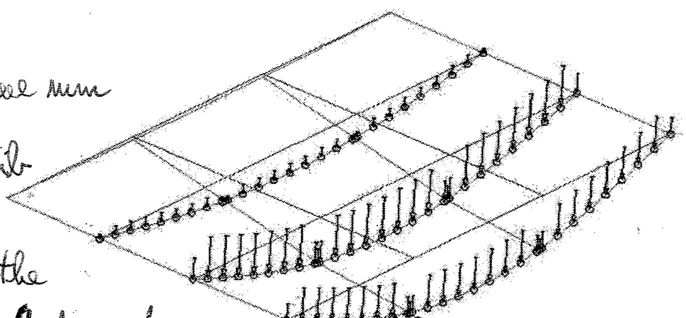
For distributed loads

$$M_{S,distributed} = 406kNm$$

With Courbon model it was $M_{S,distributed} = 463kNm$ 14% difference

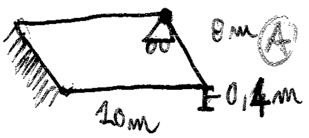
With Engesser model it was $M_{S,distributed} = 338kNm$ 17% difference

→ In this case, if we sum $M_{S,concentr} + M_{S,distrib}$ ⇓ $M_{S,total}$ of FEM is the smallest value (but is not a general rule).



Courbon, Engesser, FEM are only models; reality is different (⇒ Courbon, Engesser and FEM are all wrong).

N.B: When I talk about 2 dimensional elements, I mean structural element (A), not FEM element (B)



Structural analysis of 2 dimension elements

1 dimension is small in regard to the other 2

$\frac{10}{0{,}4} = \frac{10 \cdot 10}{4} = 25$

the thickness is 25 times smaller than the other dimension ⇒ is it a 2dimensional element or not?

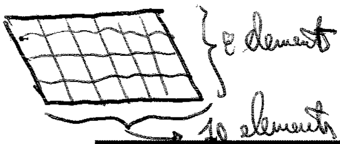
Structural analysis of 2 dimension elements can be done in several way:

- closed form solutions (simple structures under simple loads conditions)
  "crazy people" solved the integrals in Taylor series.
- tabular methods (simple structures under simple loads conditions)
- approximate methods (stripe methods)
- numerical methods (finite differences, finite elements)

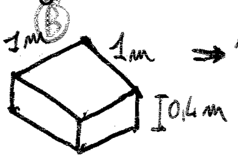
"2 dimension element" doesn't depend on the number of FEM elements that I use for the mesh.

the 2nd aspect is: we use shell elements for mesh:

Finite element methods is nowaday the most popular as it allows to analyse generic structures under generic load and restraint conditions.



⇒ a single element is

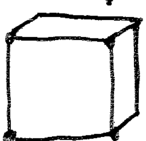


⇒ if I refer to this element the thickness is 2,5 times smaller than the other dimensions

N.B. at the top of the page

If I mesh a Eulero-Bernoulli beam with 1 or 2 or 5 or 100'000 beam elements ⇒ I reach always ~~exactly the same result (because I have finite integrals)~~. If I use Timoshenko beam ⇒ results are different (because I have numerical integrals) to solve

SHELLS:



Which are the internal action of a shell???

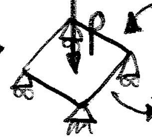
The complete solution of the most generic 2D structure, a shell, gives as output:

- **3 membrane components** n_x, n_y, $n_{xy} = n_{yx}$ (σ_x, σ_y, τ_{xy}) for Cauchy
- **5 plate components**
  - **2 bending moments and a torque moment** m_x, m_y, $m_{xy} = m_{yx}$
  - **2 out of the plane shears** t_x, t_y

5 INTERNAL ACTIONS: 2 bending moment, 2 shears, 1 torque

We saw (about the shells):
- PLANE STRESS → MEMBRANE
- PLANE STRAIN
- AXIAL SYMMETRY (stato di deformazione assialsimmetrica)

the deformations are anyway in the plane the structure (like for plane strain)



the structure deformates outside its plane. → PIASTRA (plate)

are grouped in a family called "2D SOLIDS", but is a bit ambiguous, because in our mind is not a solid, but a thin element. In this "2D SOLIDS" family, the next structure doesn't belong in it:

→ CONTINUE IN THE NEXT PAGE

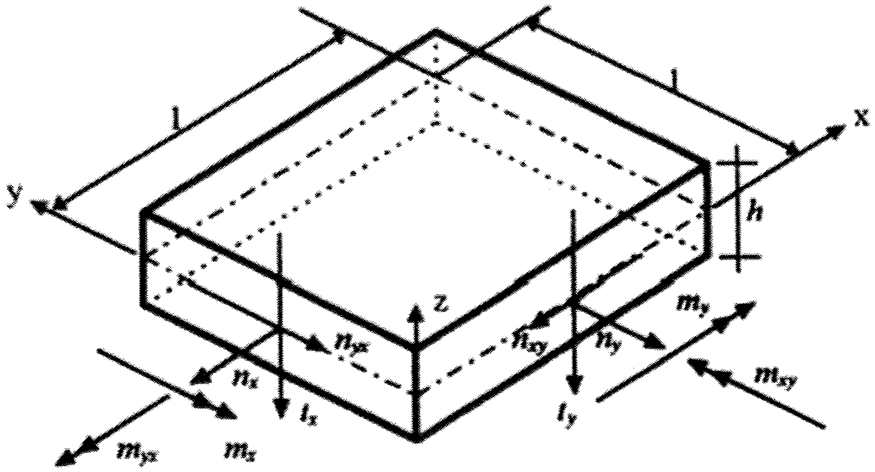
Pay attention

All these internal actions are for unit lenght, for instance:

$n_x, n_y, n_{xy}, t_x, t_y \rightarrow$ [kN/m]

$m_x, m_y, m_{xy} \rightarrow$ [kNm/m]

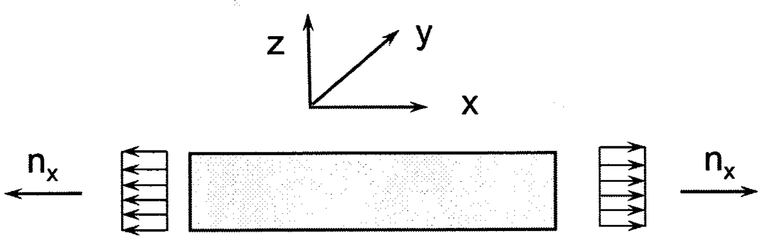
plane stress actions



Sign convention for membrane internal actions:

- n_x: gives rise to σ_x stresses, positive if the stresses are tensile

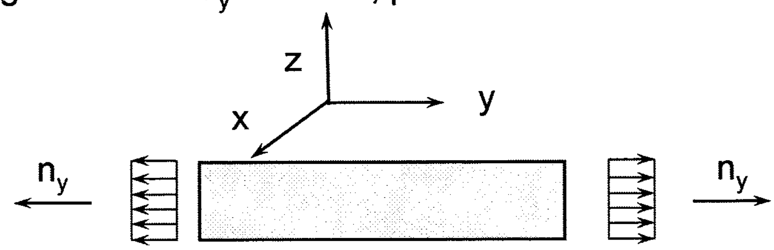
$$\sigma_x = \frac{n_x}{h}$$



$$\sigma_x = \frac{n_x}{h}$$

- n_y: gives rise to σ_y stresses, positive if the stresses are tensile

$$\sigma_y = \frac{n_y}{h}$$

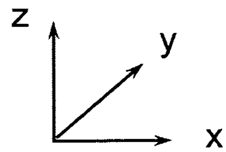


$$\sigma_y = \frac{n_y}{h}$$

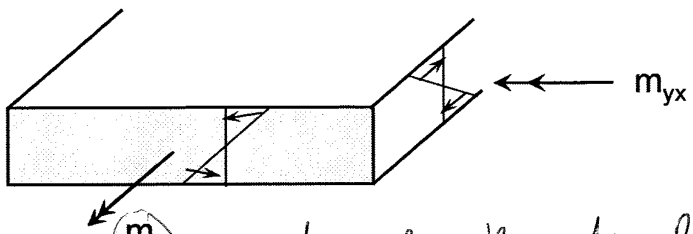
Riavolture

- m_{xy}: gives rise to τ_{xy} stresses, it is positive as shown in the figure

torque bending moment



$$\tau_{xy} = \frac{12 m_{xy}}{h^3} z$$

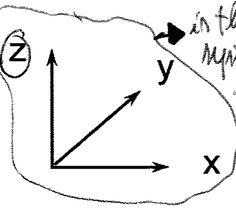


m_{xy} is positive if on the positive face give rise to positive τ in the positive direction of z axis.
directed as the axis

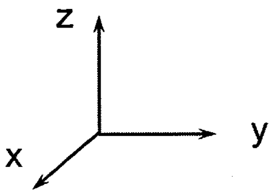
Sign convention for out of the plane shears:

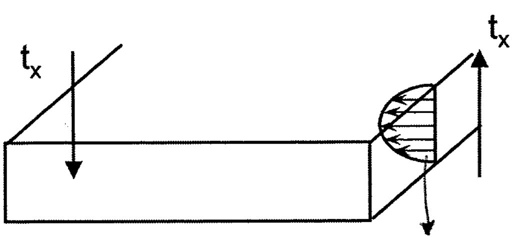
They are positive if directed along positive z on positive faces and along negative z on negative faces.

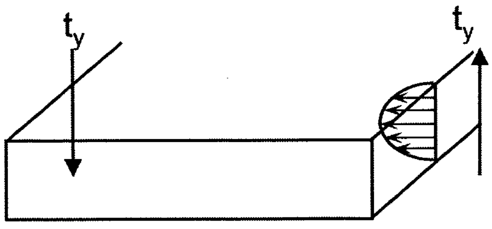
is the out of the plane axis of the element



is the local referring system of the element, no the global referring system of the structure







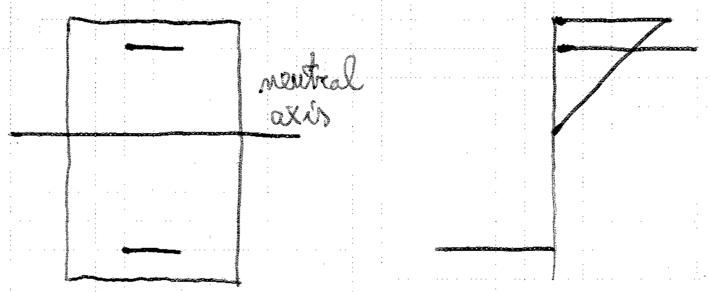
$$(\tau_{xz})_{max} = \frac{3}{2}\frac{t_x}{h}$$

Jourawsky

$$(\tau_{yz})_{max} = \frac{3}{2}\frac{t_y}{h}$$

(✳) → CONTINUE FROM THE BOTTOM OF SLIDE 12/72

In SLE LIMIT STATE, for a beam, we have:

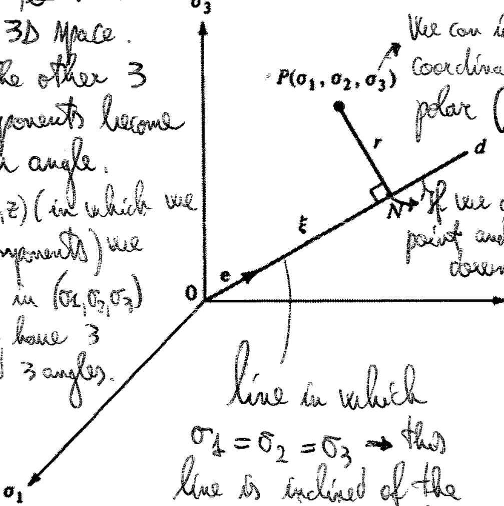


LEZ. 05-11-2015

3D space with
Principal stresses referring system.

We know that stresses are 6 (3 σ and 3 τ), but we don't have a physical imagine of a 6D space, so we use a 3D space. The other 3 components become an angle.

From (x,y,z) (in which we have 6 components) we will arrive in ($\sigma_1, \sigma_2, \sigma_3$) in which we have 3 directions and 3 angles.



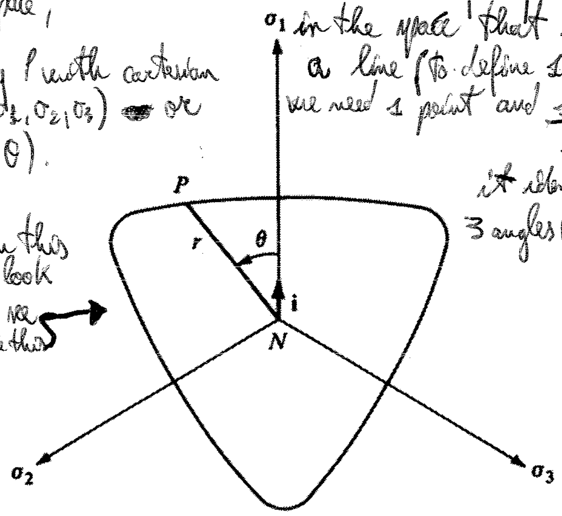
We can identify P with cartesian coordinates ($\sigma_1, \sigma_2, \sigma_3$) or polar ($\xi, r, \theta$).

If we are in this point and we look down we see like this

line in which $\sigma_1 = \sigma_2 = \sigma_3$ → this line is inclined of the same angle respect to $\sigma_1, \sigma_2, \sigma_3$. This line is called hydrostatic line, because $\sigma_1 = \sigma_2 = \sigma_3$ and all $\tau = 0$ (like in the sea underwater)

ma se, da qui in poi
Deviatoric or
Octahedral plane and lode angle

There is only 1 plane in the space that is ⊥ to a line (to define 1 plane we need 1 point and 1 line.) → it identifies 3 angles (directions)



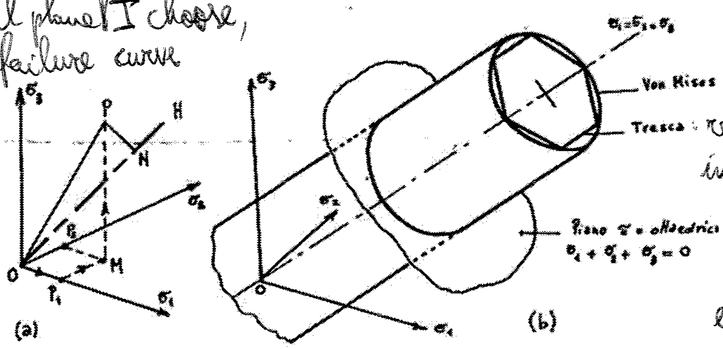
According to Von Mises theory, if we move on hydrostatic line → we don't break our material

This is hydrostatic tension, the opposite is hydrostatic compression line

Let's see the resisting criteria (for concrete ALONE, not for reinforced concrete)

Tresca and Von Mises failure criteria

Whichever ortogonal plane to the hydrostatic line I choose, the shape of the failure curve doesn't change. Is it true or not? For steel yes, for brittle material...



Tresca: regular exagon inscribed in the Von Mises circle.

With Tresca the direction θ is important, because an exagon is not a circle → failure is not regardless on the θ angle.

$$\sigma_3 - \sigma_2 = \pm\sigma_{cu} \quad \sigma_2 - \sigma_1 = \pm\sigma_{cu} \quad \sigma_1 - \sigma_3 = \pm\sigma_{cu}$$

Tresca

$$F(J_2', J_3') = 4J_2'^2 - 27J_3'^2 - 9\sigma_y^2 J_2'^2 + 6\sigma_y^4 J_2'^2 - \sigma_y^6 = 0 \quad (A)$$

the failure is reached when the difference between 2 principal stresses is equal to a certain stress. Write this concept with an equation is very difficult expression (A)

And: They depend on material properties:

$$\alpha = \frac{1}{9k^{1.4}} \qquad \beta = \frac{1}{3.7k^{1.1}}$$

$$c_1 = \frac{1}{0.7k^{0.9}} \qquad c_2 = 1 - 6.8(k - 0.07)^2$$

Where k is the ratio between tensile and compressive strength ≅ 0.1

$$k = f_{ctm}/f_{cm}$$

Von Mises and Tresca are also called "1 parameter criteria", because you only give the σ_{cu} to identify the failure surface. This is not true for Mohr and Coulomb: to identify the Coulomb criteria you give the friction angle (φ) and the cohesion (c) ⇒ "2 parameter criteria" (like Mohr)

The previous criteria are not used for concrete.

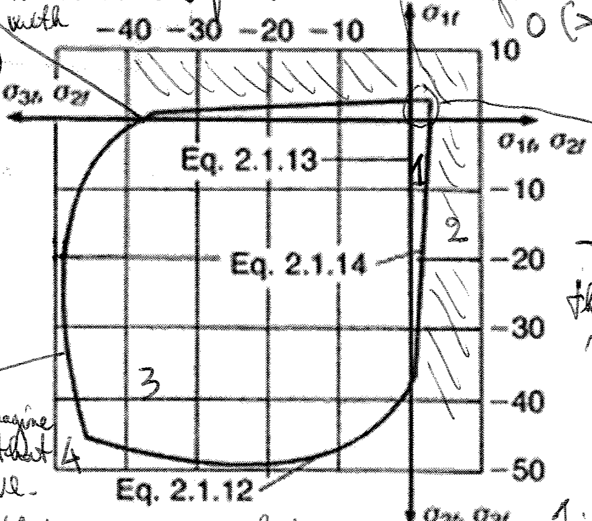
If concrete is not cracked (Φ≤0) the principal stresses are inside the resistance dominion, which can be evaluated, according to Model Code 90, with the following relations.

where it's \\\ it means that Φ (of slide 13/72) is greater than 0 (>0).

starting point and opening of the cone.

f_{cm}: cylindric strength ← here we have the uniaxial compression failure (like with a specimen)

Failure criteria for concrete in a 2D space.



when we are here we have the tension strength of the material.

The bigger is the compression, the bigger is the τ that the material can bear.

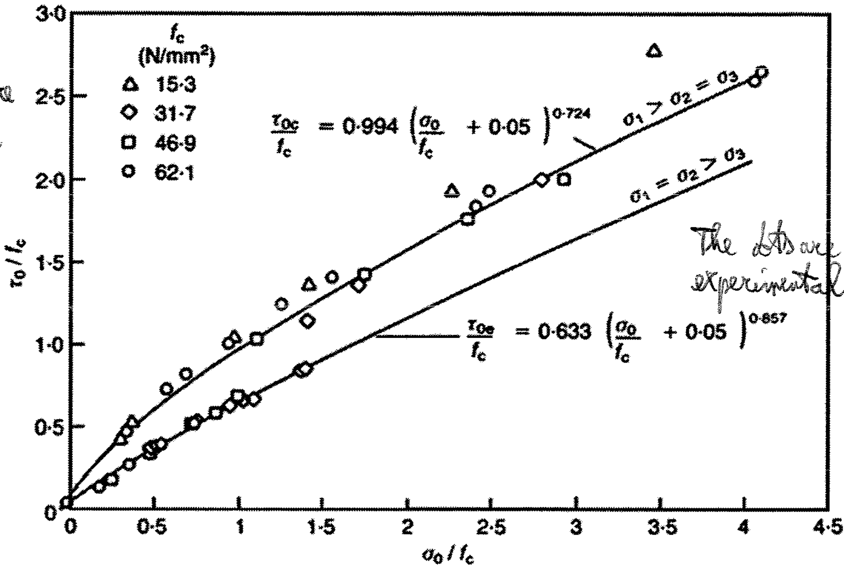
Kupfer and Gerstle (1973) failure curve

from this curve we can't imagine the 3D failure surface that give rise to this curve. → it's that one in the slide 22/72

1: not cracked, 2: cracked
3: not crushed; 4: crushed

$$r=\sqrt{3}\tau_{0u}=\sqrt{3}\,\frac{2\tau_{0u}\left(\tau_{0c}^2-\tau_{0e}^2\right)cos\theta+\tau_{0c}\left(2\tau_{0e}-\tau_{0c}\right)\sqrt{4\left(\tau_{0c}^2-\tau_{0e}^2\right)cos^2\theta+5\tau_{0e}^2-4\tau_{0c}\tau_{0e}}}{4\left(\tau_{0c}^2-\tau_{0e}^2\right)cos^2\theta+\left(\tau_{0c}-2\tau_{0e}\right)^2}$$

In this equation you don't see the hydrostatic stress, because τ_{0c} and τ_{0e} are function of the hydrostatic stresses.



The dots are the experimental tests.

this model was developed for thicknesses that have reinforcement inside (like the imagine at the top of next slide), not outside

If Φ>0 we have to work in cracked state using a sandwich model.

The basic idea derives from the analogy with the beam element.

Three different layers are individuated inside the shell element.

The two external ones bear membrane actions coming both from membrane and the plate external actions.

The inner one, working as a beam web bears the out of the plane shears.

Each layer has a constant thickness and the following quantities are introduced:

N.B.

Usually these second quantities are set equal to the respective ones presented in the previous slide.

They can be set to different values in order to place different layers of reinforcement for the different actions in the same concrete layer.

The membrane actions that interest the external layer can then be calculated by writing equilibrium equations as follows

so there is a proportion based on the distance

membrane action in the upper layer

these signs are correct with the convention seen at the start of these slides

$$n_{xs} = n_x \frac{z_x - y_{xs}}{z_x} + \frac{m_x}{z_x} \qquad n_{yxs} = n_{yx} \frac{z_{yx} - y_{yxs}}{z_{yx}} - \frac{m_{yx}}{z_{yx}}$$

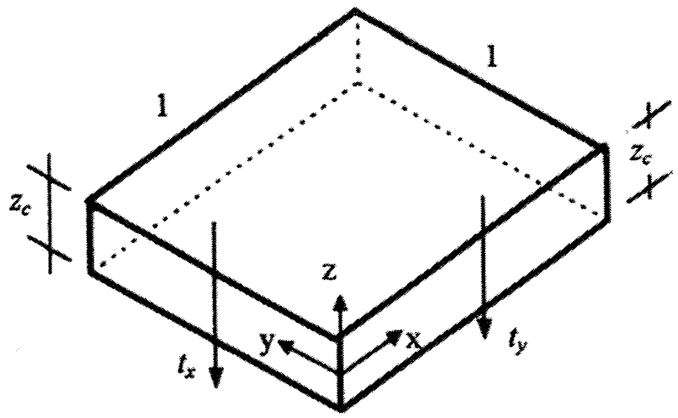
$$n_{xi} = n_x \frac{z_x - y_{xi}}{z_x} - \frac{m_x}{z_x} \qquad n_{yxi} = n_{yx} \frac{z_{yx} - y_{yxi}}{z_{yx}} + \frac{m_{yx}}{z_{yx}}$$

$$n_{ys} = n_y \frac{z_y - y_{ys}}{z_y} + \frac{m_y}{z_y} \qquad n_{xys} = n_{xy} \frac{z_{xy} - y_{xys}}{z_{xy}} - \frac{m_{xy}}{z_{xy}}$$

$$n_{yi} = n_y \frac{z_y - y_{yi}}{z_y} - \frac{m_y}{z_y} \qquad n_{xyi} = n_{xy} \frac{z_{xy} - y_{xyi}}{z_{xy}} + \frac{m_{xy}}{z_{xy}}$$

Out of the plane shears are carried by the internal layer of thickness t_c , that works in collaboration with the external ones.

Therefore we will consider an effective thickness z_c equal to the weighted average of z_x, z_y, z_{xy} and z_{yx}



Sandwich model:
Internal layer design

Why do we start the design with the internal layer?
It's the simpler one, and also this internal layer affects the design of the other 2 layers (like in beams: we translate the M diagram because of the shear)
so we'll have additional anchorage length of the reinforcements (not more quantity of reinforcement (peak is not influenced) but more anchorage length.)

It can be observed that the value of the principal shear

$$t_0^2 = t_x^2 + t_y^2 = t_n^2 + t_m^2$$

Is constant with respect to the angle φ. (whichever angle φ I choose)

Moreover there's a angle φ_0 that makes $t_n = t_0$ and $t_m = 0$;

This is the principal direction of shear and can be calculated as: (direction in which the shear is maximum)

$$\tan\varphi_0 = t_y / t_x$$

In this direction the shell element behaves like a beam as it is subjected to shear in only one direction.

Analogia con la superficie di sci (cioè dove il taglio è massimo) (superficie di M): se stiamo sciando lungo la direzione di max. pendenza e ci mettiamo ⊥ a tale direzione ⇒ stiamo fermi (siamo nella direzione di 0 pendenza). In una superficie c'è sempre la direzion di 0 pendenz e il taglio è 0.

Maybe in other courses we called it V_{Rdc} shear resistance of a concrete beam without stirrups. If $t_0 > V_{Rd1}$ ⇒ we use the Morsch truss system.

if $t_0 > \boxed{V_{Rd1}}$ shear reinforcement should be provided. The resisting mechanism is equal to a beam, a unit length wide, and oriented along the principal shear direction. According to Model Code 90 we have:

- verification of compressed struts. ⇒ we apply the beam "theory", prescription along the direction of maximum shear.

$$F_{scw} = \frac{t_0}{\sin\vartheta} \le F_{Rcw} = f_{cd2} z_c \cos\vartheta$$

compressed resistance of compressed struts in shear (is smaller than the uniaxial compressed resistance (not shear))

- verification of tensed reinforcement (vertical stirrups)

$$F_{Stw} = t_0 \le F_{Rtw} = \frac{A_{sw} f_{ywd}}{s} z_c \cot\vartheta$$

- force variation in longitudinal chords (tensed and compressed)

additional tension in the tensed zone $\Delta F_{st} = \Delta F_{sc} = \frac{t_0}{2}\cot\vartheta$ (safe approximation)

additional tension in the compressed zone (but we don't consider it, like in beams)

The angle θ is subjected to the same limitations seen for beams (26°<θ<45°). Whereas the forces $\Delta F_{st} = \Delta F_{sc}$ are directed in the direction φ_0 and should be decomposed along x and y.

I can write 2 equilibrium equations: 1 of translation equilibrium along x direction and 1 of translation equilibrium along y direction and then I ask the prism to stay still ("fermo")

about the last image of last slide

y direction: $n_{yc} = t_o \cot\vartheta \sin^2\varphi_0 = t_o \cot\vartheta \dfrac{\mathrm{tg}^2\varphi_0}{1+\mathrm{tg}^2\varphi_0} =$ (from trigonometry)

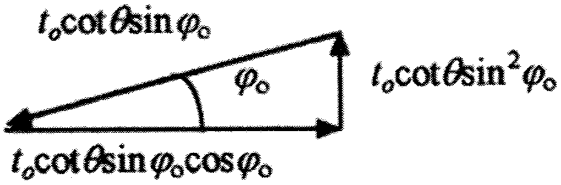
$$= t_o \cot\vartheta \frac{t_y^{\,2}/t_x^{\,2}}{1+t_y^{\,2}/t_x^{\,2}} = t_o \cot\vartheta \frac{t_y^{\,2}}{t_x^{\,2}+t_y^{\,2}}$$

x direction: $n_{xyc} = t_o \cot\vartheta \sin\varphi_0 \cos\varphi_0 = t_o \cot\vartheta \dfrac{\mathrm{tg}\varphi_0}{1+\mathrm{tg}^2\varphi_0} =$

$$= t_o \cot\vartheta \frac{t_y/t_x}{1+t_y^{\,2}/t_x^{\,2}} = t_o \cot\vartheta \frac{t_x t_y}{t_x^{\,2}+t_y^{\,2}}$$

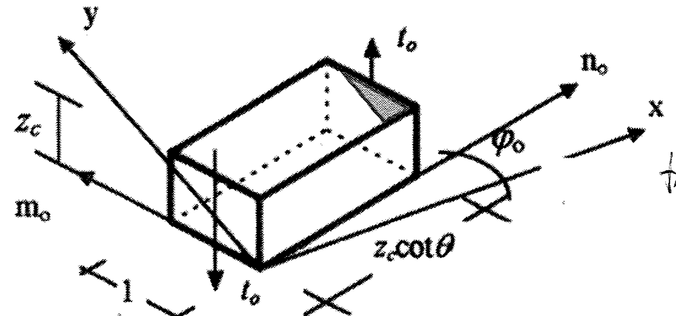
$$\boxed{n_{yc} = \frac{t_y^{\,2}}{t_o}\cot\vartheta \qquad n_{xyc} = \frac{t_x t_y}{t_o}\cot\vartheta}$$

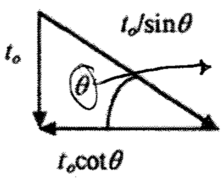
t_x and t_y are outputs of the FEM.



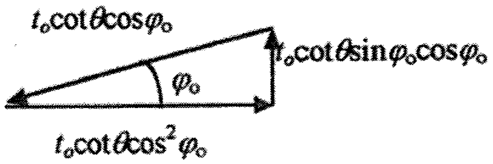
The same for the prism cutted with a vertical plane parallel to the y axis:

$$\boxed{n_{xc} = \frac{t_x^{\,2}}{t_o}\cot\vartheta \qquad n_{yxc} = n_{xyc} = \frac{t_x t_y}{t_o}\cot\vartheta}$$

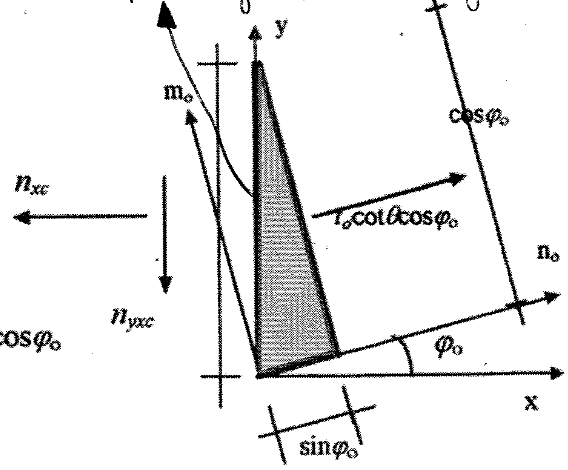




it's our choice (like in beam)



this cutting surface is 1 length here



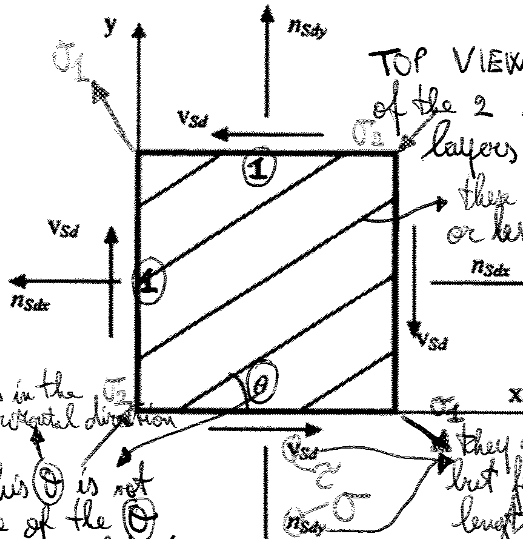
For seek of simplicity we refer to the mean level of reinforcement in each of the two layers:

$y_{xs}=y_{ys}=y_{xys}=y_{yxs}=y_s$ $\quad$ $y_{xi}=y_{yi}=y_{xyi}=y_{yxi}=y_i$ $\quad$ $z=y_s+y_i$

Each layer is subjected to membrane actions (see figure).

The design can be done according to plasticity theory with a lower bound solution.

The values of the three internal actions shown beside for each layer are given by the equations presented in the next slide.



TOP VIEW of one of the 2 external layers.

these lines go more or less in the direction of cracks

it is in the horizontal direction

they are not stresses but forces per unit length : τ = V/thickness

N.B.: this θ is not the same of the θ in the 38/72 slide (we call both θ because is an angle related to the shear)

In total we'll have 3θ:
- θ_c for the internal layer
- 2θ for the external layers (θ_s and θ_i)

θ_s, θ_i : related to membrane stresses

θ_c : out of the plane stresses

it is in the vertical direction

The designer, at the end, choose 5 parameters
- 2 thicknesses (of the external layers)
- 3 angles (1θ for the out of the plane and 2θ for the in of the plane)

↓ optimization of a 5 variable space

↓ output:
- reinforcements in the upper and lower layer,
- shear reinforcement

No shear reinforcement needed

the actions in the layer are not function of the out of the plane shear (t_x, t_y) and of θ_c

In the beam we choose only 1 parameter (θ angle of the shear)

$$n_{Sdx,s}=n_x\frac{z-y_s}{z}+\frac{m_x}{z}$$

$$n_{Sdx,i}=n_x\frac{z-y_i}{z}-\frac{m_x}{z}$$

$$n_{Sdy,s}=n_y\frac{z-y_s}{z}+\frac{m_y}{z}$$

$$n_{Sdy,i}=n_y\frac{z-y_i}{z}-\frac{m_y}{z}$$

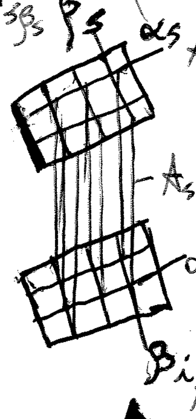
$$v_{Sd,s}=n_{xy}\frac{z-y_s}{z}-\frac{m_{xy}}{z}$$

$$v_{Sd,i}=n_{xy}\frac{z-y_i}{z}+\frac{m_{xy}}{z}$$

superiore

action on the layer

action on the element



Shear reinforcement needed

$$n_{Sdx,s}=n_x\frac{z-y_s}{z}+\frac{m_x}{z}+\frac{1}{2}\frac{t_x^2}{t_o}\cot\vartheta_c$$

$$n_{Sdx,i}=n_x\frac{z-y_i}{z}-\frac{m_x}{z}+\frac{1}{2}\frac{t_x^2}{t_o}\cot\vartheta_c$$

$$n_{Sdy,s}=n_y\frac{z-y_s}{z}+\frac{m_y}{z}+\frac{1}{2}\frac{t_y^2}{t_o}\cot\vartheta_c$$

$$n_{Sdy,i}=n_y\frac{z-y_i}{z}-\frac{m_y}{z}+\frac{1}{2}\frac{t_y^2}{t_o}\cot\vartheta_c$$

$$v_{Sd,s}=n_{xy}\frac{z-y_s}{z}-\frac{m_{xy}}{z}+\frac{1}{2}\frac{t_xt_y}{t_o}\cot\vartheta_c$$

$$v_{Sd,i}=n_{xy}\frac{z-y_i}{z}+\frac{m_{xy}}{z}+\frac{1}{2}\frac{t_xt_y}{t_o}\cot\vartheta_c$$

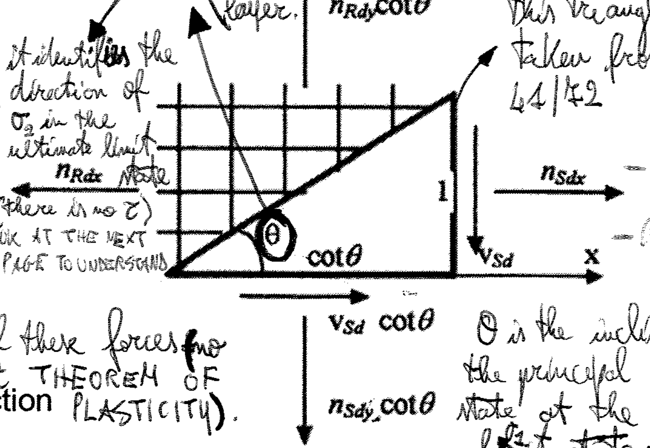
θ_c: vertical θ in the central layer (it's not the θ in the

LOOK ALSO AT THE NEXT NOTE'S PAGE

We cut the layer with a plane parallel to the direction of compressions at failure. this θ is θ_{pl} (and we have a θ_{pl} for the upper layer and a different θ_{pl} for the lower layer.)

This direction can be chosen by the designer starting from the value of the inclination of principal stress σ_2 at cracking and changing it of $-20°<\Delta\theta< 20°$ (there is no τ) LOOK AT THE NEXT PAGE TO UNDERSTAND

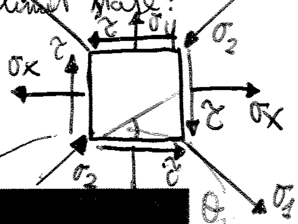
Remember that a big $\Delta\theta$ leads to wider cracks in service.



PEDICE: -S: acting force -R: resisting force

We only calculate forces and the equilibrium of these forces (no displacements) ⇒ we are applying the STATIC THEOREM OF PLASTICITY).

θ is the inclination of the principal compression state at the ultimate limit state:



θ_e elastic is the inclination of σ_2 in the elastic field.

Equilibrium to traslation in x direction

$$n_{Rdx} = n_{Sdx} + v_{sd}\cot\theta = A_{sx}\sigma_{sx} \rightarrow f_{yd}$$

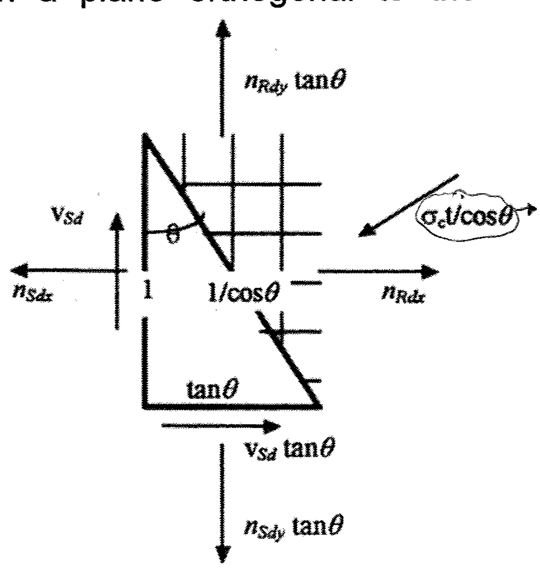
dimensionally it's $[mm^2/m]$ (general in slabs we put 1000 mm/m for example)

Equilibrium to traslation in y direction

$$n_{Rdy}\cot\theta = n_{Sdy}\cot\theta + v_{sd} \Rightarrow n_{Rdy} = n_{Sdy} + \frac{v_{sd}}{\cot\theta} = A_{sy}\sigma_{sy}$$

That situation is before that cracks open. Then the cracks open with θ_e inclination

Then we cut the same layer with a plane orthogonal to the direction of compressions



Equilibrium to traslation in x direction

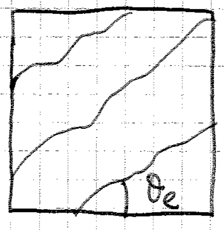
$$-\frac{\sigma_c t}{\cos\theta}\cos\theta + n_{Rdx} - n_{Sdx} + v_{sd}tg\theta = 0$$

And substituting the value of n_{Rdx} calculated in the previous slide we get

$$\sigma_c t = +\cancel{n_{Sdx}} + v_{sd}\cot\theta - \cancel{n_{Sdx}} + v_{sd}tg\theta$$

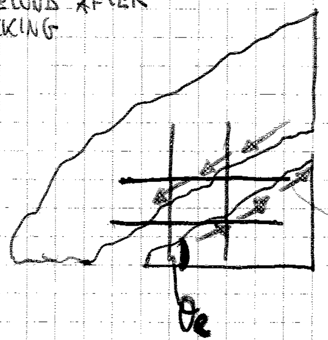
We have 4 equations in 3 variables (A_{sx}, A_{sy}, θ_{pl}) there are only 3 equations linearly independent.

CONTINUATION OF THE SLIDE 45/72



After crack we have:

A SECOND AFTER CRACKING

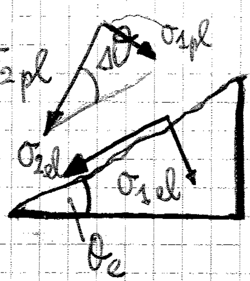


The cracks are skew to the reinforcements (not like in beam, where the bending cracks are $\perp$ to the bending reinforcements).

τ_{crack} (due to friction, interlock)

Generally concrete always works in the cracked stage. After cracking I have:

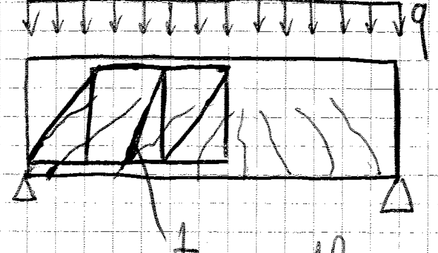
A SECOND AFTER THE SECOND ABOVE



$\theta_{pl} = \theta_e + \Delta\theta$

A second after crack $\Rightarrow$ the inclination of σ_{2el} is θ_e.
Then, a second after, the inclination becomes $\theta_e + \Delta\theta$, because of the presence of τ_{crack}.

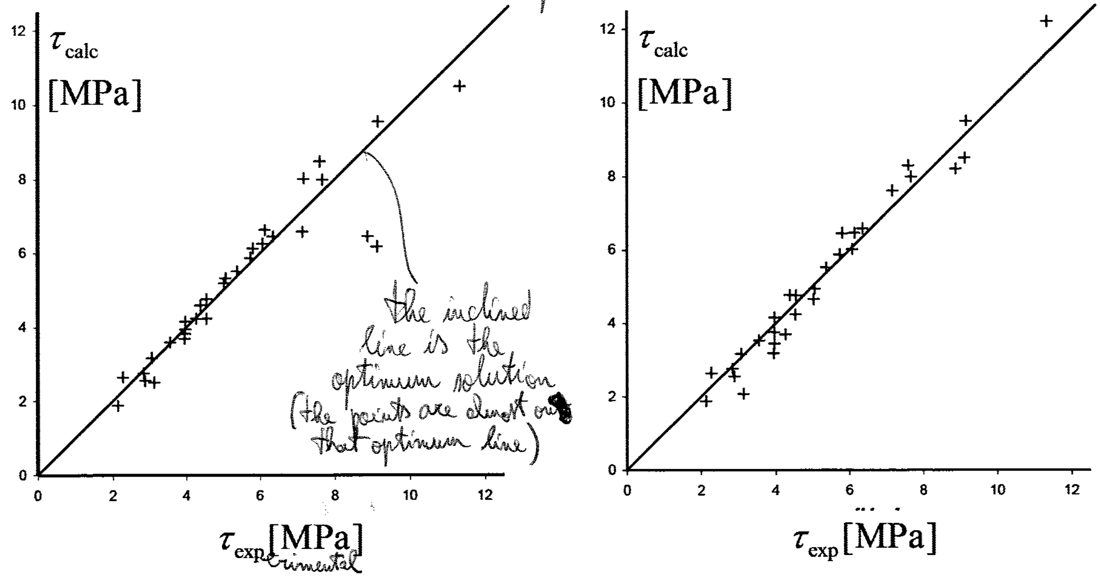
In a concrete beam we have:



truss model: you choose the θ_{pl} (the inclination of principal compression tension $(22° < \theta_{pl} < 45°)$)

In beams we don't calculate the θ_e, but only the θ_{pl} (we force the concrete to work with a certain θ_{pl} ...

Difference between our model and experimental research:



(a) (b)

Experimental versus calculated panel strength by Marti and Kaufmann (a) and by Carbone, Giordano and Mancini (b)

Something about layers thickness

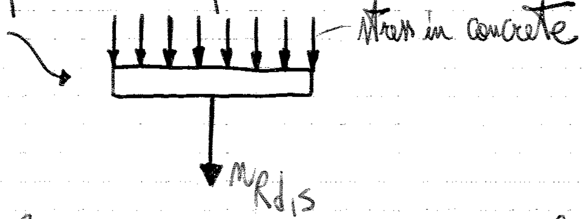
The choice of the thickness of the layers is up to the designers as far as:

- $t_s \geq 2 \times b'_s$ where b'_s is the distance between the upper reinforcement centroid and the extrados of the slab
- $t_i \geq 2 \times b'_i$ where b'_i is the distance between the lower reinforcement centroid and the intrados of the slab
- $t_s + t_i < h$

The reinforcement A_{sx} and A_{sy} calculated in the previous slides are supposed in the middle of the thickness of their layer.

SLIDE 51/72

Now we see what happen if reinforcement is not centered in the thickness of the layer.
Look at the drawing, but imagine it turned of 90°.
The thickness of the layer (t_s or t_i) is bigger than 2 times the cover (b'_s and b'_i), so the reinforcements are not centered in the center of the layer, but in this case is in the outerside of the layer.
$n_{Rd,s}$ and $n_{Rd,i}$ we imagined centered in the layer (no one was taking into account the eccentricity of the reinforcement $\Rightarrow$ we have to write 2 equilibrium equations between n_i, n_s and $n_{Rd,i}$, $n_{Rd,s}$ (2 unknowns: n_i and n_s)
You may ask: the stresses in concrete were supposed to be constant in the thickness of the layer and so they were in equilibrium with the stress in reinforcement ($n_{Rd,s}$ and $n_{Rd,i}$):



If, instead, I consider the single layer with the stress n_s in the reinforcement and the uniform distribution of stresses in concrete $\Rightarrow$ the single layer is not in equilibrium anymore $\Rightarrow$ in reality the stresses in concrete are not constant in the layer, but inclined in order to have equilibrium.
So, the σ_c that we calculated is the average stress in the layer.

We get

stresses in the reinforcement

$$\rho_\alpha \sigma_{s\alpha} = \frac{\sigma_x \sin\theta\cos\beta - \sigma_y \cos\theta\sin\beta + \tau_{xy}\cos(\theta+\beta)}{\sin(\theta-\alpha)\cos(\alpha-\beta)}$$

calculation of reinforcement in α direction

geometrical ratio of reinforcement

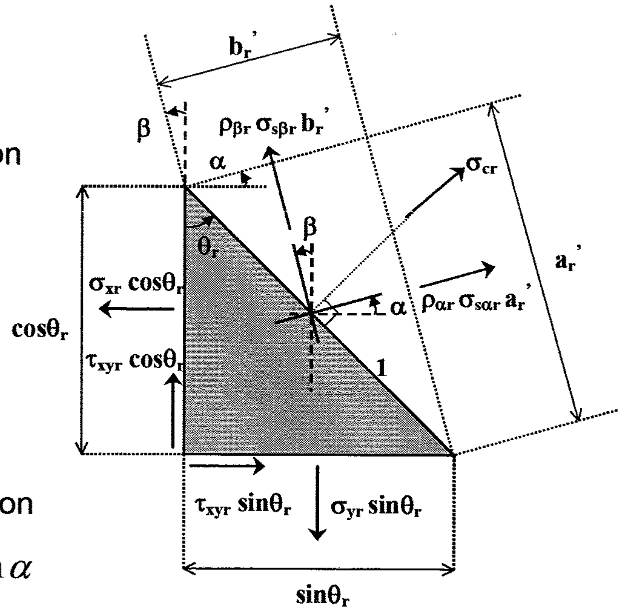
$$\rho_\beta \sigma_{s\beta} = \frac{\sigma_x \sin\theta\sin\alpha + \sigma_y \cos\theta\cos\alpha + \tau_{xy}\sin(\theta+\alpha)}{\cos(\theta-\beta)\cos(\alpha-\beta)}$$

Calculation of reinforcement in β direction

From which we can calculate the reinforcement ratios by choosing the reinforcement stress, remembering:

$$\sigma_{s\,\alpha,\beta} \le f_{yd}$$

Then we cut the same r layer with a plane orthogonal to the direction of compressions



Equilibrium to traslation in x direction

$$-\sigma_x\cos\theta + \tau_{xy}\sin\theta + \rho_\alpha\sigma_{s\alpha}a'\cos\alpha$$
$$-\rho_\beta\sigma_{s\beta}b'\sin\beta + \sigma_c\cos\theta = 0$$

Equilibrium to traslation in y direction

$$-\sigma_y\sin\theta + \tau_{xy}\cos\theta + \rho_\alpha\sigma_{s\alpha}a'\sin\alpha$$
$$-\rho_\beta\sigma_{s\beta}b'\cos\beta + \sigma_c\sin\theta = 0$$

4 equations, 3 unknowns ($A_{s\alpha}, A_{s\beta}, \theta_r$)

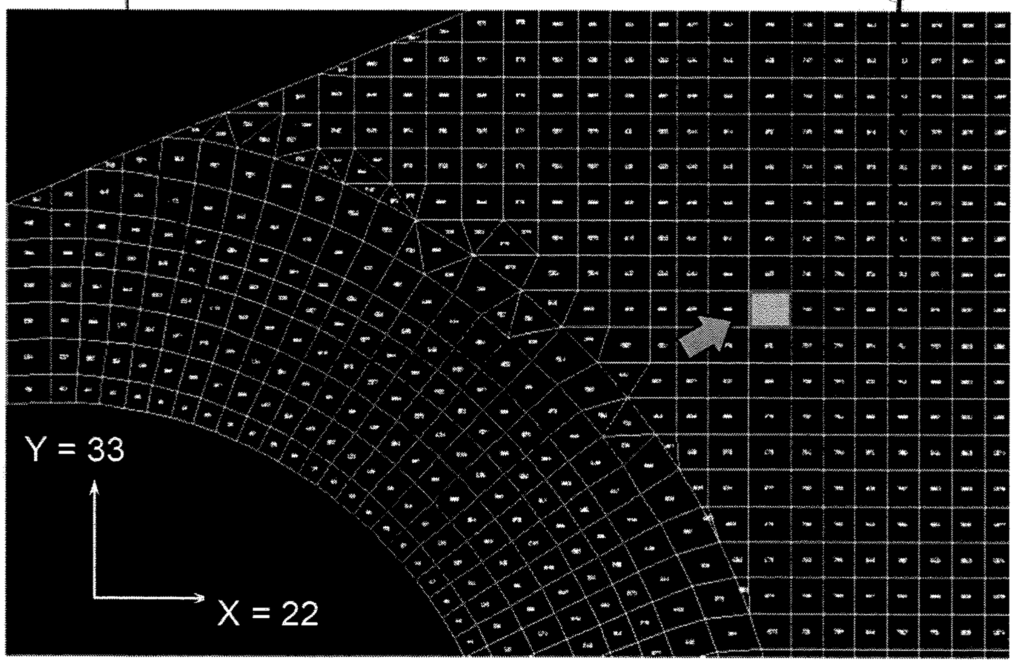
Sandwich model: Numerical example

---

turn of the road

Element chosen: n°682

direction of the road



TOP VIEW of FEM MODEL of a bridge.

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	Layers thicknesses	
H sez.	**tsup**	**tinf**
(m)	(m)	(m)
1.0000	0.23	0.18

Increment of internal actions due to shear (for the single layer)		
nsd22	**nsd33**	**nsd23**
(KN/m)	(KN/m)	(KN/m)
0	0	0

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	σ_c(f)	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
-633	-4070	481	no.	65.0	17.6	17.6	0.0	0.0	15.7	15.7

Lower layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	σ_c(f)	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
909	-1064	-711	yes	23.1	11.1	11.1	1212.0	607.2	31.3	15.7

Upper layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	σ_c(f)	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
-695	-3904	474	no	45.0	17.6	17.6	0.0	0.0	15.7	15.7

Lower layer verification

Internal actions on the layer			Cracked ?	Concrete parameters			Actions at $t_{sup}/2$		Reinforcement calculated at $c+\phi/2$	
nsd22	**nsd33**	**nsd23**	case	θ	ν fcd	σ_c(f)	$n_{R1(x)}$	$n_{R2(y)}$	**A_s(x)nec**	**A_s(y)nec**
(KN/m)	(KN/m)	(KN/m)	(-)	(°)	(N/mm²)		(kN/m)	(kN/m)	(cm²/m)	(cm²/m)
956	-1229	-693	yes	20.6	11.1	11.1	1216.7	611.5	31.3	15.7

Dimensioning of α reinforcement (transverse) in the superior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	**Nsd22** (KN/m)	**Nsd33** (KN/m)	**Nsd23** (KN/m)	**Msd22** (KNm/m)	**Msd33** (KNm/m)	**Msd23** (KNm/m)	**Vsd12** (KN/m)	**Vsd13** (KN/m)
Max M22	261	-5134	-219	657	1014	-464	79	-197

Load combination that maximizes this reinforcement

Dimensioning of β reinforcement (longitudinal) in the superior layer

Distance of reinforcement from the outer surface = 6 cm

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Max M22	261	-5134	-219	657	1014	-464	79	-197

Load combination that maximizes this reinforcement

	Layers thicknesses	
H sez.	**tsup**	**tinf**
(m)	(m)	(m)
1.0000	0.23	0.19

Increment of internal actions due to shear (for the single layer)		
nsd22	**nsd33**	**nsd23**
(KN/m)	(KN/m)	(KN/m)
0	0	0

Verification of maximum compressive stresses in concrete

Superior layer

Combination type	Nsd22 (KN/m)	Nsd33 (KN/m)	Nsd23 (KN/m)	Msd22 (KNm/m)	Msd33 (KNm/m)	Msd23 (KNm/m)	Vsd12 (KN/m)	Vsd13 (KN/m)
Min V12	188	-5215	-148	-88	143	-245	-43	-48

Load combination that maximizes the stress

	Superior layer		Inferior layer		Centoid	
Thicnness	$\sigma_{1,sup}$	$\sigma_{3,sup}$	$\sigma_{1,inf}$	$\sigma_{3,inf}$	$\sigma_{1,g}$	$\sigma_{3,g}$
[m]	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)	(N/mm²)
1.0000	0.97	-6.32	0.23	-4.93	0.21	-5.22

References

- *CEB-FIP Model Code 1990*, Thomas Telford – 1990
- *Eurocode 2 Design of concrete structures, Part 1-1: general rules and rules for buildings* - 2003
- *Eurocode 2 Design of concrete structures – Part 2: concrete bridges* - 2004

BRIDGE DESIGN

LEZ. 19-11-2013

LOCAL EFFECTS ON SLABS

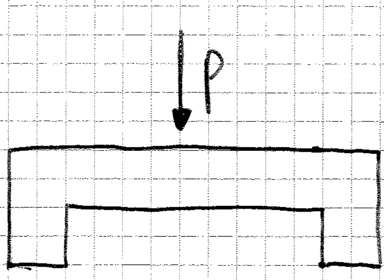
We'll talk about the slab of a girder (→ we'll not talk about slab bridges)
The approach that we'll see works if we design a slab by hand.

We'll see only 1 of the possible local effects on slab: the bending effect of a concentrated load applied.

Summary:

1. General considerations
2. Actions on the slab
3. Equivalent beam dimensioning

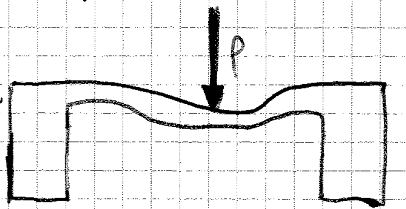
CONTINUATION OF THE SLIDE 4/25 : which restraint do we consider?

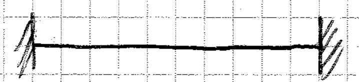


Deformed shape: I have, at least, 2 limit alternatives:

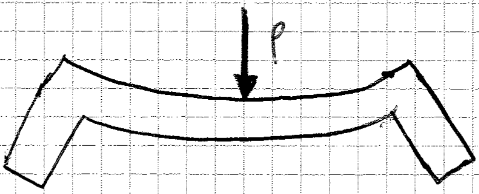
static scheme for the slab:

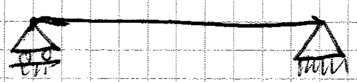
(1)





(2)





The real behaviour is in the middle, but we'll never know exactly the percentage of the 1st and the 2nd behaviour.

(1) " " ∞ " " .

(2) longitudinal beams have nil torsional stiffness.

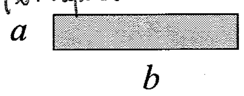
The rotation of the beams is function of the torsional stiffness of the beams themselves

The torsional stiffness is function of G (that, remember, is smaller than E).

G decreases about 4-5 times from uncracked to cracked stage (and you never know about the correct value to use) in the design

→ you don't know exactly which situation you'll have in reality.

1.2 Aspect ratios → we can have slab/beam in the point of view of the calculation (it depends on the aspect ratio)



Called b the longest side of a slab
Called a the shorter one

long size → b, short size → a

If $\frac{b}{a} > 2.5$ The slab can be dimensioned as equivalent beam, because, even if it's physically a slab → it behaves like a beam:

If $\frac{b}{a} < 2.5$ The slab has to be dimensioned as a real slab

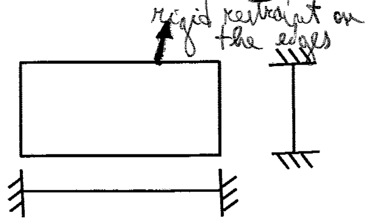
it's not "global this limit": it can be 2 or 3.

1.3 Level of restraint

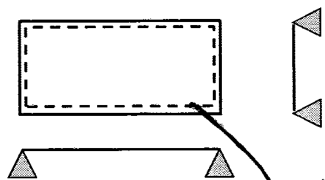
In both cases (real slab or equivalent beam) the real level of restraint provided by longitudinal beams and transverse beams is difficult to be correctly evaluated.

For a simplified analysis the slab can be dimensioned for the envelope of the two limit situations of:

rigid restraint on the edges



simple support on the edges



it means that here there is not ~~rotation~~ restraint

2.1 Permanent loads

- **Dead load** $g_1 = h_s \cdot \gamma_s = 0.25m \cdot 25\frac{kN}{m^3} = 6.25kN/m^2$

- **Carried permanent loads**

  pavement $g_{2,p} = 3.00kN/m^2$

  kerb $g_{2,k} = h_k \cdot \gamma_k = 0.23m \cdot 25\frac{kN}{m^3} = 5.75kN/m^2$

  pedestrian barrier $g_{2,pb} = 1.00kN/m$

2.2 Imposed deformations (shrinkage, thermal deformations...)

No imposed deformations will be taken into account for sake of simplicity

3. Equivalent beam dimensioning

3.1 Inner field of slab

In our case $\frac{b}{a} = \frac{14.7}{3} = 4.9 > 2.5$ We can use the equivalent beam simplification on the transverse span

3.1.1 Effective span calculation

(1) The effective span, l_{eff}, of a member should be calculated as follows:

$$l_{eff} = l_n + a_1 + a_2 \quad (5.8)$$

where:

l_n is the clear distance between the faces of the supports;
values for a_1 and a_2, at each end of the span, may be determined from the appropriate a_i values in Figure 5.4 where t is the width of the supporting element as shown.

3.1.2 Bending moment due to permanent loads

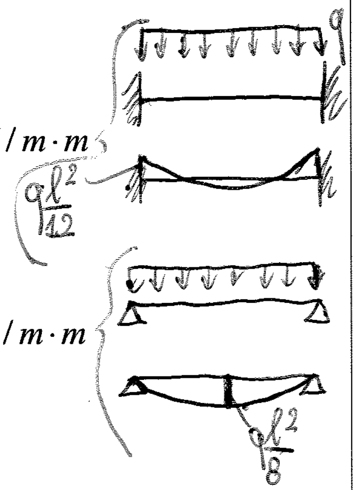
self-weight permanent load

Cantilever $M_{cant,g} = (6.25 + 5.75) \cdot \frac{1.38^2}{2} + 1.0 \cdot 1.38 =$

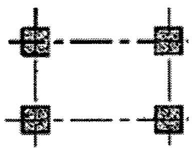
$11.4 + 1.4 = 12.8 kN/m \cdot m$

Inner span
Fully restrained (static scheme) $M_{in\_edge,g} = (6.25 + 3.00) \cdot \frac{2.75^2}{12} = 5.83 kN/m \cdot m$
At the edge

Inner span
Simply supported (static scheme) $M_{in\_mid,g} = (6.25 + 3.00) \cdot \frac{2.75^2}{8} = 8.74 kN/m \cdot m$
In mid-span

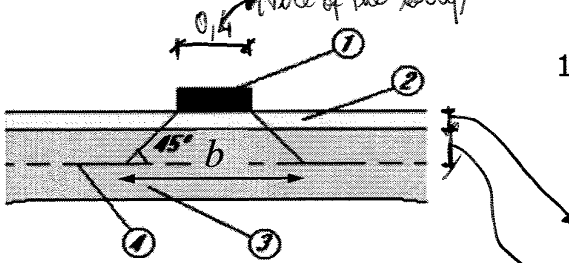


3.1.3 Bending moment due to variable loads

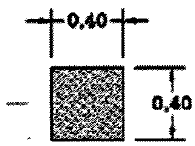


3.1.3.1 Load Model 1 (LM1)

Dispersal of concentrated loads



1 wheel dimension



2 pavement thickness = 0.13m

3 half slab thickness = 0.125m

Width of the dispersed load area

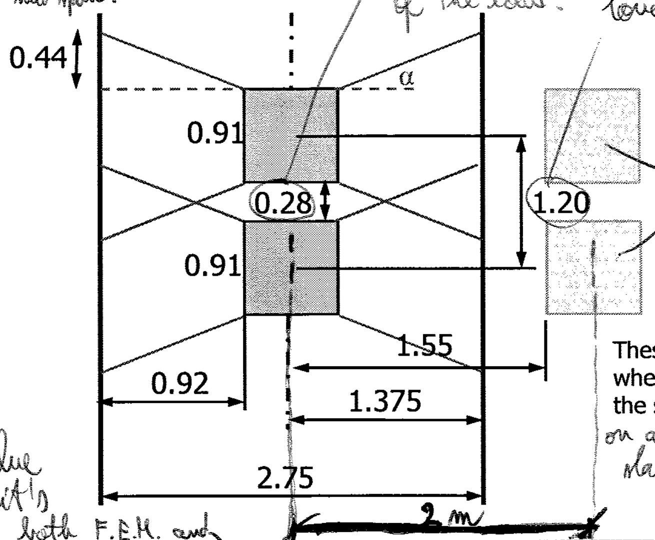
$b = 0.40 + 2 \cdot (0.13 + 0.125) = 0.91m$ → the value of the resultant of the uniformly distributed load decreases of about 3-4 times (because the loaded area increases from 0,4m to 0,91m)

What is the effective resisting cross section that bears the bending moment just calculated ?

We put the wheels to maximize the bending moment in the mid span:

TOP VIEW:

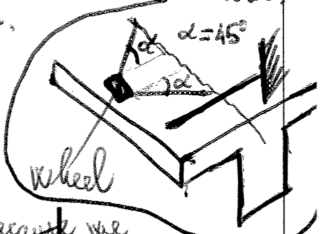
In longitudinal direction we place 2 single wheels of each tandem along the mid-span longitudinal axis of the slab as the other two of each tandem fall on the other field of the slab.



These other 2 wheels are outside the studied field (they go on another portion of slab)

The dispersion angle α is 25.6°: this value is given by the code and it's derived from both F.E.M. and experimental tests on slab → static schemes

we consider this value for both: full restraint – full restraint and simply supp. – simply supp. Only for the cantilever we have:



We want to calculate the amount of slab that has to resist to M = 85.8 KN·m/m.

The dispersion length is: $\underset{\text{resisting cross section}}{l_{disp}} = 0.91 + \underbrace{2 \cdot 0.92 \cdot tg(25.6°)}_{0{,}44} = 1.79m$

The overlapping length (painted in orange) spreads for

$l_{ovrl} = 0.44 \cdot 2 - 0.28 = 0.60m$

The acting bending moment is then

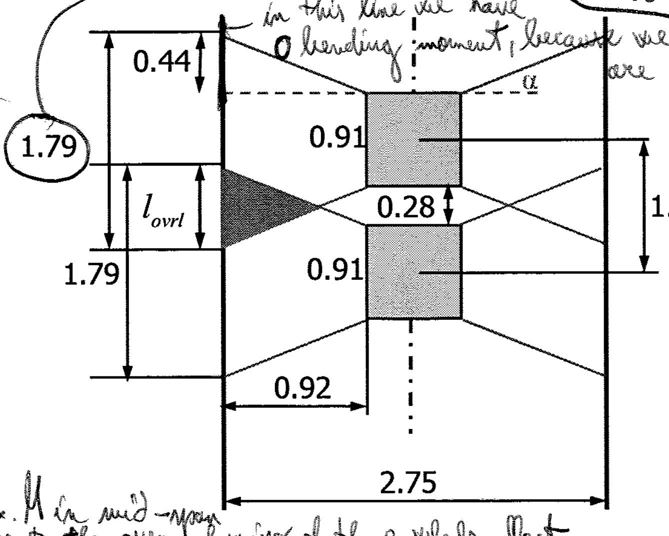
$M = M_{conc,trasv} / l_{disp} =$

$85.8 / 1.79 = 47.9 kNm/m$

This value has to be doubled as the overlapping length is >0 so

I have 2 wheels

$M_{conc,LM1} = 95.8kNm/m$ → max. M in mid-span due to the overlapping of the 2 wheels effect



What is the effective resisting cross section that bears the bending moment just calculated ?

In longitudinal direction we place 2 single wheels of each tandem in the position given us by the influence line just calculated.

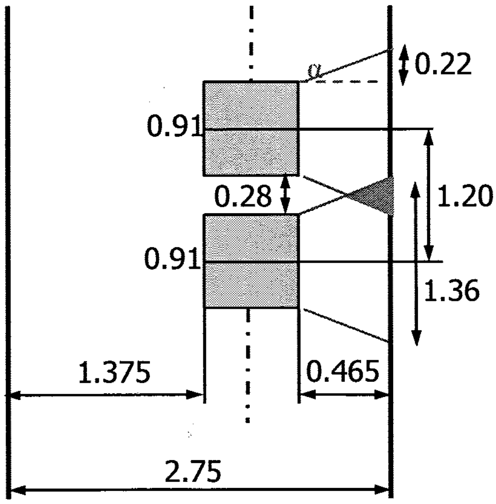
The dispersion angle α is 25.6°

The dispersion length is:

$$l_{disp} = 0.91 + 2 \cdot 0.465 \cdot tg(25.6°) = 1.36m$$

The overlapping length (painted in orange) is

$$l_{ovrl} = 0.22 \cdot 2 - 0.28 = 0.16m$$ (smaller than last time)



The acting bending moment is then

$$M = M_{conc,trasv} / l_{disp} = 57.2/1.36 = 42.1 kNm/m$$

This value has to be doubled as the overlapping length is >0 so

$$M_{conc,LM1} = 84.2 kNm/m$$

2 wheels

$$\frac{57,2 \cdot 2}{1,36 - 0,16 + 1,36} = \frac{114,4}{2,56} = 45 \, kN\frac{m}{m}$$

it's more realistic (check it with F.E.M. but it should be more realistic (see the next note page).

For the distributed live load we get

Maximum positive in mid span ⇒ Scheme of simply supported beam:

$$M_{distr,LM1} = \frac{ql^2}{8} = \frac{9 \cdot 2.75^2}{8} = 8.51 kNm/m$$

Maximum negative at the edges ⇒ Scheme of fully restrained beam:

$$M_{distr,LM1} = -\frac{ql^2}{12} = -\frac{9 \cdot 2.75^2}{12} = -5.67 kNm/m$$

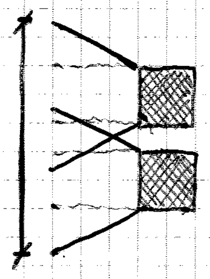
CONTINUATION OF THE SLIDE 24/25

Looking at the slide 18/25 $\Rightarrow l_{disp} - l_{ovel} + l_{disp} =$
$= 1{,}79 - 0{,}6 + 1{,}79 = 3\,m$

$M_{tot} = 85{,}8 \cdot \underset{\substack{\downarrow \\ \text{2 wheels}}}{2} = 172\ kN \cdot m/m$

$\Rightarrow \frac{172}{3} \cong 57\ kN \cdot m/m$: it's good approximation compared to the F.E.M. solution ($\Rightarrow$ better result than the result in the slide 18/25).

$\Rightarrow$ considering overlapping like on slide 18/25 is too on the safe side, while considering on the total dispersion length is better.



N.B. BEFORE DO F.E.M. DO ALWAYS THE HAND CALCULATION TO HAVE THE ORDER OF MAGNITUDE

LEZ. 10-12-2013 francesco.tondolo@polito.it (consulenza giovedì mattina alle 10:00)

BRIDGE DESIGN

CREEP EFFECTS ON PHASED CONSTRUCTION

Politecnico di Torino
Department of structural and geotechnical engineering
"Bridge design"

The following exercise will show an application of the 5th principle of linear viscoelasticity to an homogeneous structure with rigid bearings built in several phases.

The 4th principle takes into account only 1 variation of the static scheme → we'll apply the 4th principle for the last restraint and the 5th principle for the other previous restraints.

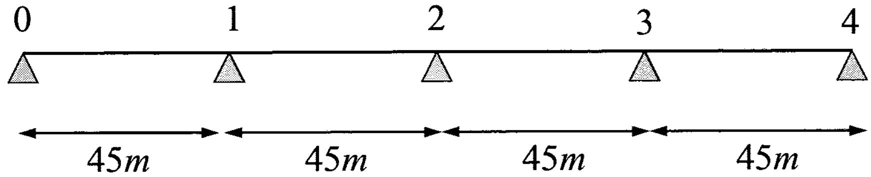
we'll use the simplified analysis (with ξ function)

$\sigma_c \le 0{,}45 \cdot f_{ck}$ (otherwise: NON LINEAR creep)

Final static scheme of the structure to build

This is the final scheme of the bridge, not the initial one! → construction phases

(a very important key point is the creep)

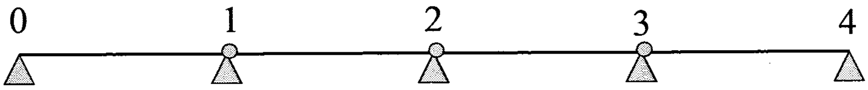


Creep means additional deformation that comes from the application of stresses.
We have to consider only dead load (permanent loads), because creep is very slow phenomenon. Variable load changes very fast.

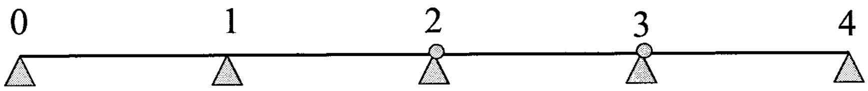
Politecnico di Torino
Department of structural and geotechnical engineering
"Bridge design"

Static schemes corresponding to the different phases

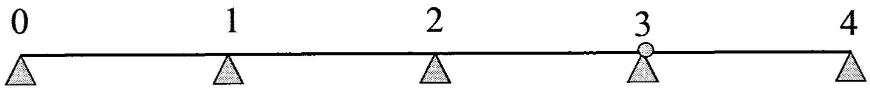
Scheme 0 28d<t<30d



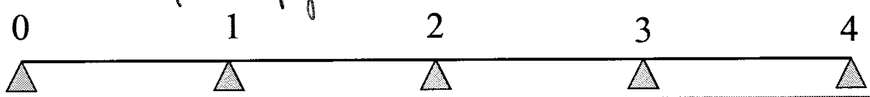
Scheme 1 30d<t<45d



Scheme 2 45d<t<60d



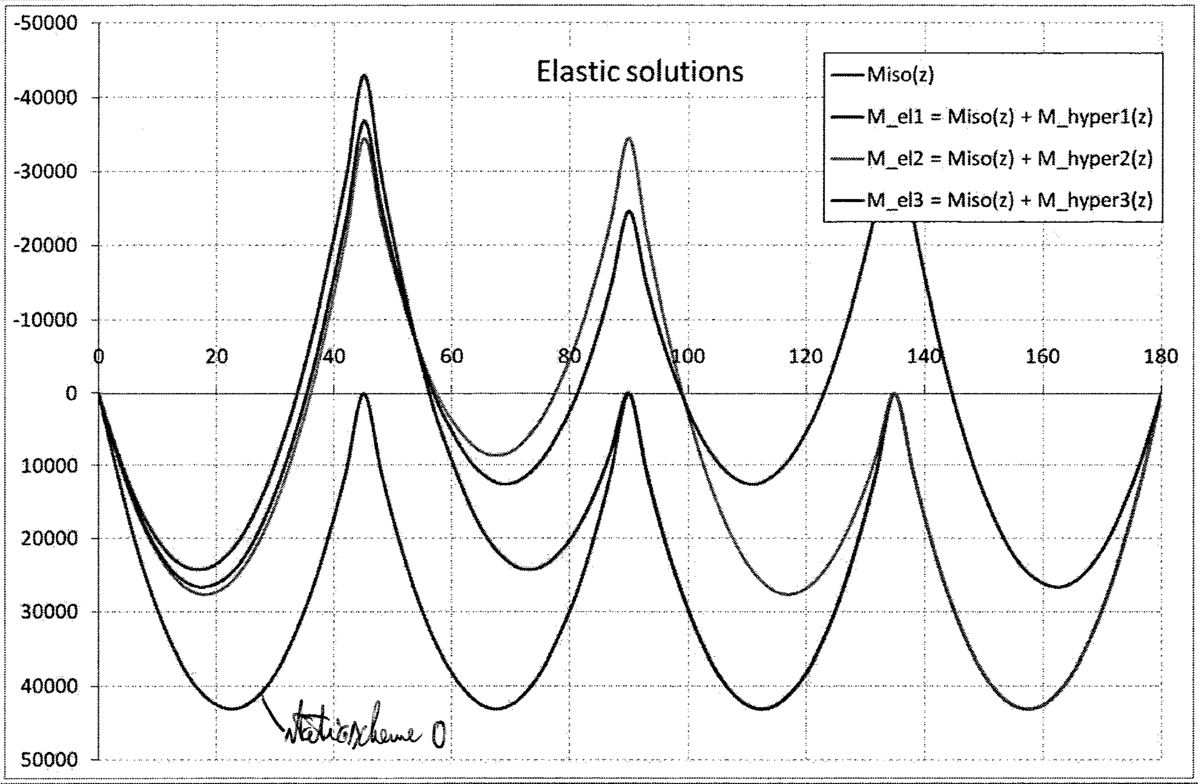
Scheme 3 60d<t : final configuration



Calculation procedure

1. Solve each of the static schemes seen above elastically and evaluate continuity bending moments.
2. Evaluate the influence functions $\alpha_j(z)$ that show the influence of the restraint j on the previous i:1,j-1. "i from 1 to j-1" → time of concreting
3. Evaluate the functions $\xi(t, t_1, t_0)$ ~~with~~ respect to section dimensions, relative humidity and time (e.g. they are given in graphical form in CEB fib bulletins) → time of changing the static scheme
4. Apply 5th principle of linear viscoelasticity

→ time of evaluation

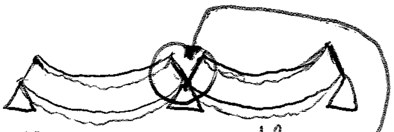


2. Evaluation of the influence functions $\alpha_j(z)$

The functions $\alpha_j(z)$ are the internal actions (for us only bending moment for sake of simplicity) calculated on the static scheme j-1 (the one previous to the introduction of the delayed restraint j) by the effect of the reaction given by the delayed restraint j set equal to unit.

Our restraint are hinges that becomes full restraints so the "reactions" are bending moments.

We have 3 delayed restraints so we will have 3 functions $\alpha_j(z)$.



The creep increases the rotation here

When I close the key, the structure wants to have more deformation, but this is ostaculated by the concreting of the key → reactions (bending moments)

Evaluation of functions $\xi(t,t_1,t_0)=\int_{t_1}^{t} R(t,\tau)dJ(\tau,t_0)$

$RH\%=50 \qquad f_{ck}=40MPa \qquad t_0=28 \quad days$

this value is half than $\xi(45,30,28)$ → if there would be a time, we could remove the scaffolding after 1-2 year, so the effect of the creep would be $=0$.

2nd restraint – key 2 $\qquad t_1=45 \quad days$

$$\left.\begin{matrix}2A_c/u=200mm \Rightarrow \xi(60,45,28)=0.25\\ 2A_c/u=400mm \Rightarrow \xi(60,45,28)=0.22\end{matrix}\right\}\frac{2A_c}{u}=369mm \Rightarrow \xi(60,45,28)=0.22$$

$$\left.\begin{matrix}2A_c/u=200mm \Rightarrow \xi(\infty,45,28)=0.67\\ 2A_c/u=400mm \Rightarrow \xi(\infty,45,28)=0.68\end{matrix}\right\}\frac{2A_c}{u}=369mm \Rightarrow \xi(\infty,45,28)=0.68$$

3rd restraint – key 3 $\qquad t_1=60 \quad days$

$$\left.\begin{matrix}2A_c/u=200mm \Rightarrow \xi(\infty,60,28)=0.64\\ 2A_c/u=400mm \Rightarrow \xi(\infty,60,28)=0.63\end{matrix}\right\}\frac{2A_c}{u}=369mm \Rightarrow \xi(\infty,60,28)=0.63$$

4. Apply 5th principle of linear viscoelasticity

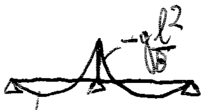
Evaluation of the internal actions.

Phase 1 (phase with scaffolding) $M=0 \quad \forall z$

Phase 2 (we remove the scaffolding) Simply supported beam moment $M_1=M_2=M_3=0$ ($t=28$ days)

Phase 3 t=30 days Simply supported beam moment $M_1=M_2=M_3=0$

we have just introduced the closing of the connection in correspondence of 1st pier, but in that time nothing happens in terms of M_1 ($M_1=0$)



t=45 days $\quad M_1=M_1^{el(1)}\cdot\xi(45,30,28)=-\frac{gl^2}{8}\cdot 0.40$

$M_2=M_3=0$

this is the 4th principle

Phase 4 t=60 days $\quad M_2=\boxed{M_2^{el(2)}\cdot\xi(60,45,28)}=-\frac{gl^2}{10}\cdot 0.22$

$M_1=\boxed{M_1^{el(1)}\cdot\xi(60,30,28)}+\alpha_{1,2}\cdot M_2^{el(2)}\cdot\xi(60,45,28)=$

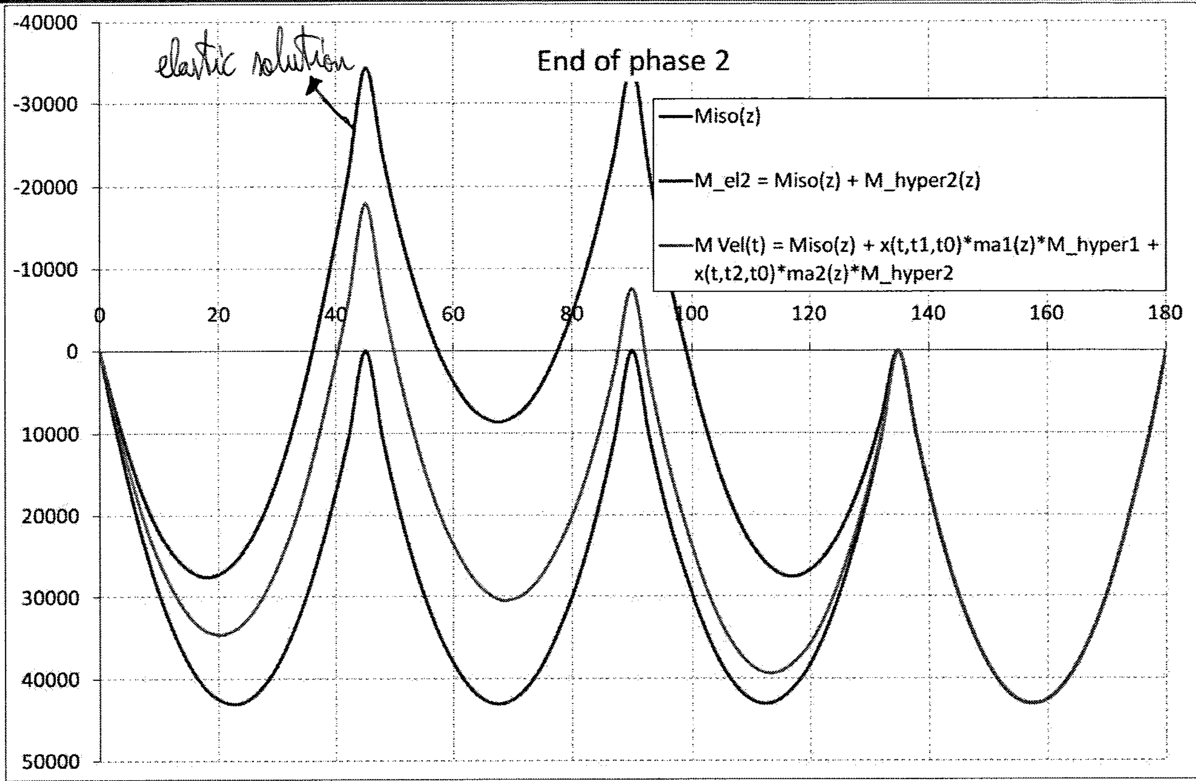
$=-\frac{gl^2}{8}\cdot 0.46+\frac{1}{4}\cdot\frac{gl^2}{10}\cdot 0.22$

elastic solution but this is the solution if the beams were born already with this configuration, but our configuration was: △ △ △) initial

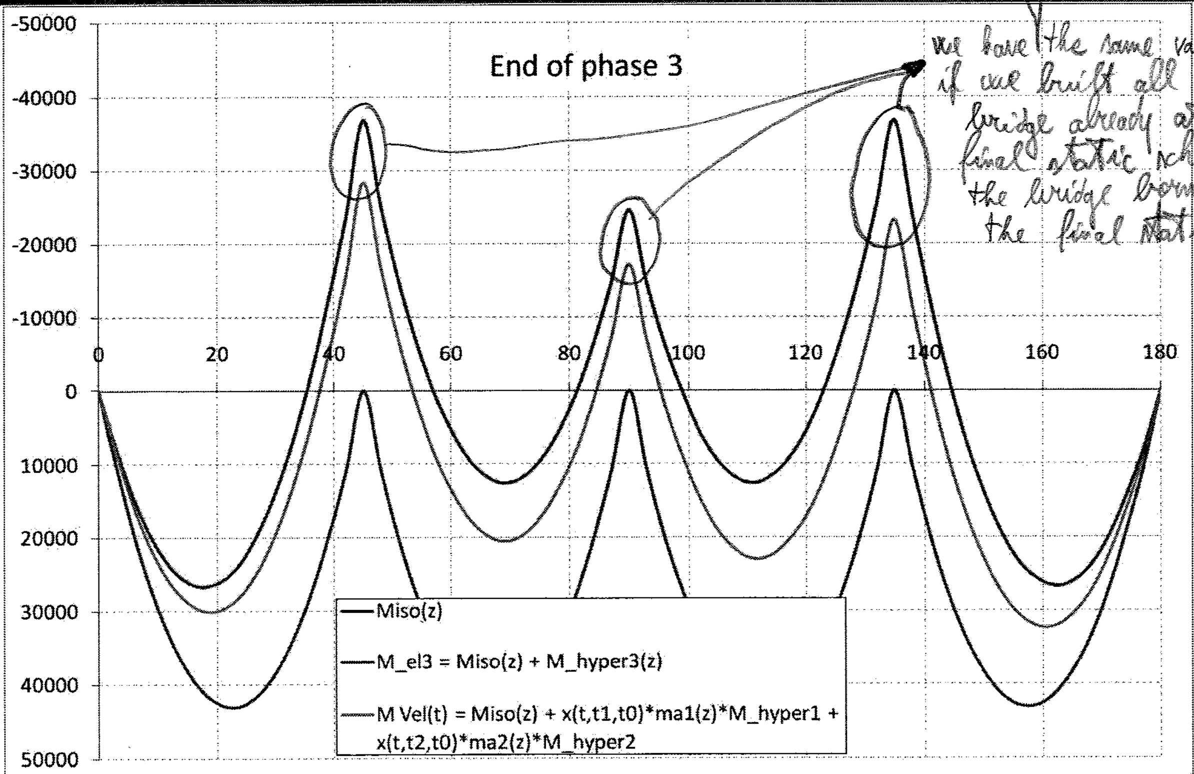
effect in the 1st delayed restraint due to the introduction of the 2nd delayed restraint

this is the 5th principle

The first delayed restraint is like a grandfather: he sees all the effects of the



we have (almost) the same value if we built all the bridge already at the final static scheme (if the bridge borns as in the final static scheme



TEORIA E PROGETTO DEI PONTI 21-11-2013

SLAB BRIDGE (see the next slides)

3/63

Castia bridge: very small bridge close to Asti.

It's irregular bridge (irregular shape) because there is a road (→ there is a turn).

6/63

F: only vertical reaction is present ⇒ free to move in longitudinal and transverse direction

L: vertical and transverse movements are fixed

The situation in the image is the minimum bearing situation: DON'T PUT TOO MANY BEARINGS!!!

7/63

139 mm^2: the area of 0.6" strands

9/63

There was an old masonry bridge in that position. Before demolished the old, the foundations of the new one were built.

10/63

Abutments: 2 blocks of concrete on 2 bored piles.

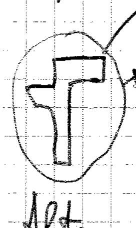
11/63

Steel part: scaffolding

The part where you put concrete: mould

(in italian we have "cassero" for both.)

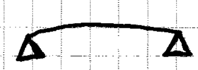
12/63



→ earth retaining wall

→ in normal bridges is ≃ 10 m tall. Here is not tall, because the abutments are very small.

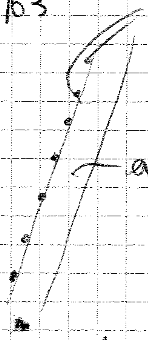
After prestressing ⇒ the bridge is supported by the external bearings ⇒ you can remove the scaffolding.



24/63

black: prestressing tendons: they are connected to the ordinary reinforcements just with "fascette" → (dell'elettricista)

25/63 vertical bars placed for spalling



end of the scaffolding

Big mistake in this image: there not spiral confinement over the initial part of the prestressing tendon → (armatura di frettaggio)

↓

no bursting reinforcement

26/63

Leave about 1 m of reinforcements out of the jack in order to tend if something will not be ok at the end of construction.

30/63

Dead anchorage of the New tendon.

Yellow thing: schiuma (" ")

31/63

Candy shaped tends in grey.

Ciano circles: longitudinal tendons.

32/63

Red: reinforcements used for bursting.

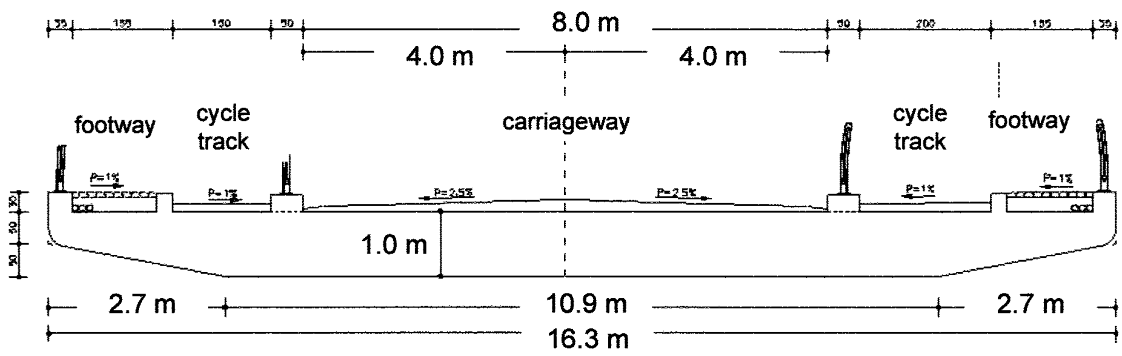
BRIDGE DESIGN

SLAB BRIDGE

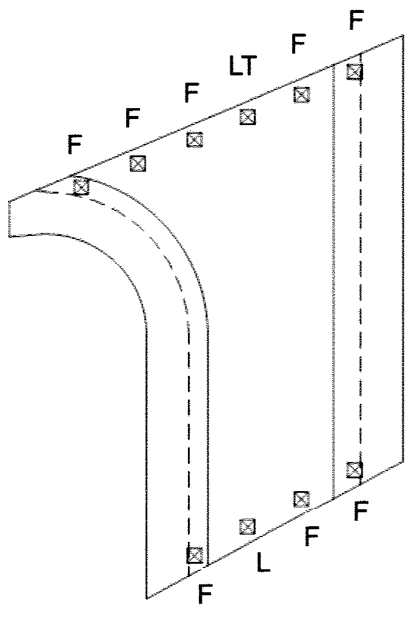
RANTIVA BRIDGE

General overview

Cross section A-A



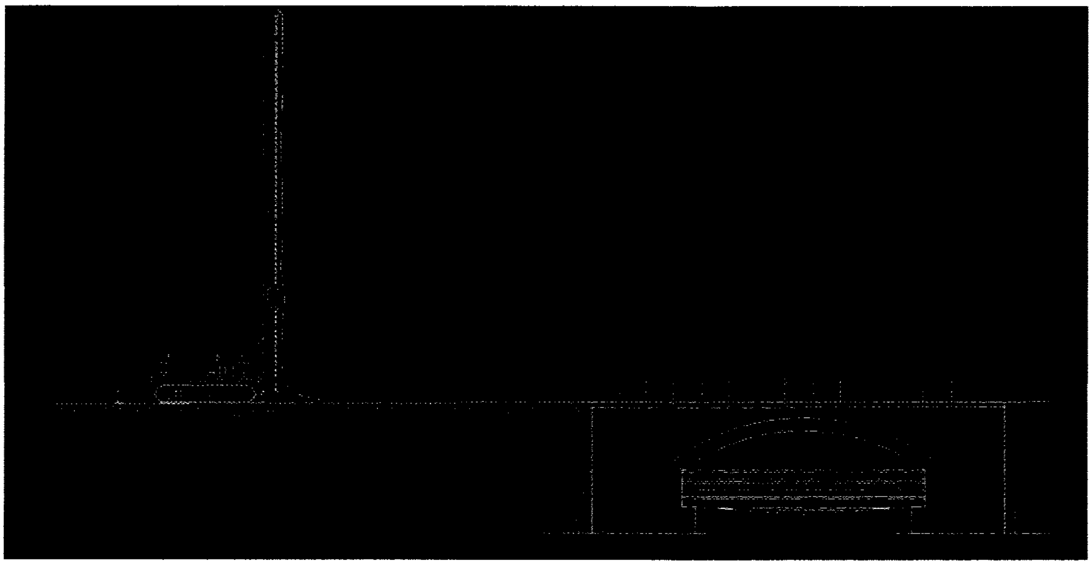
Top view - Bearings



F	free in both directions
L	fixed in transverse direction
LT	fixed in both directions

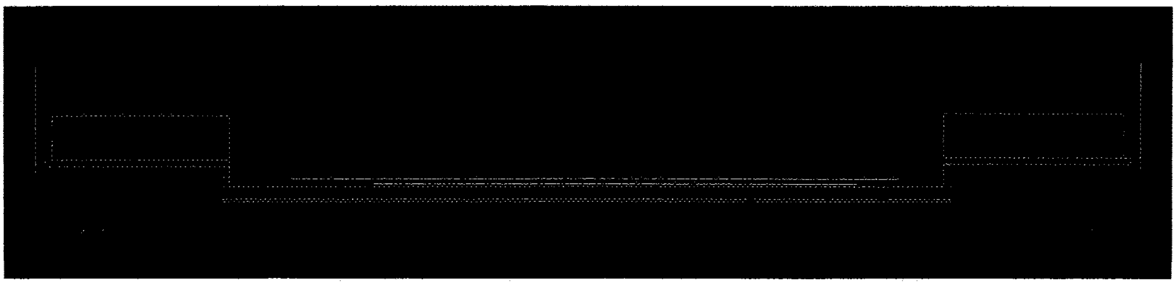
1st Phase:

boring of piles for the new abutments and demolition of old bridge



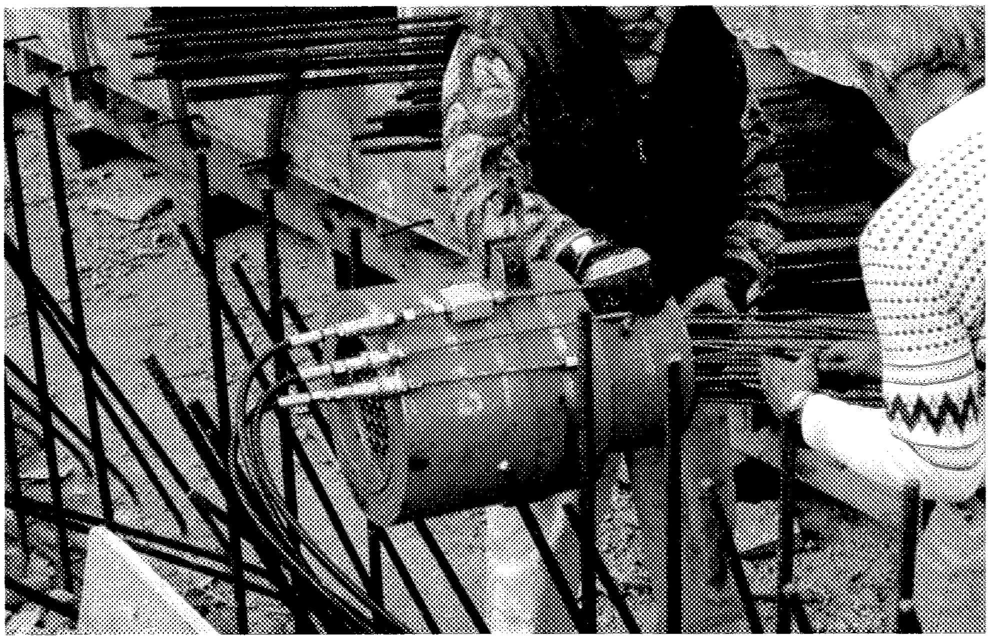
2nd Phase:

Construction of abutments foundations
Widening of the channel bed
Construction of new channel banks

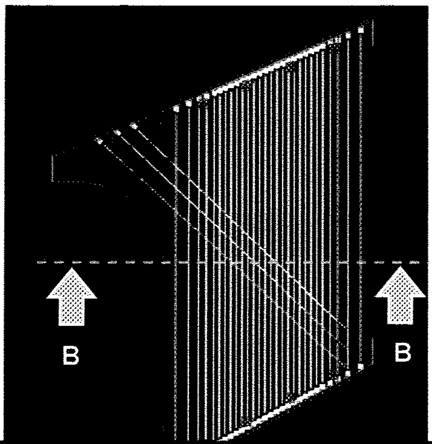


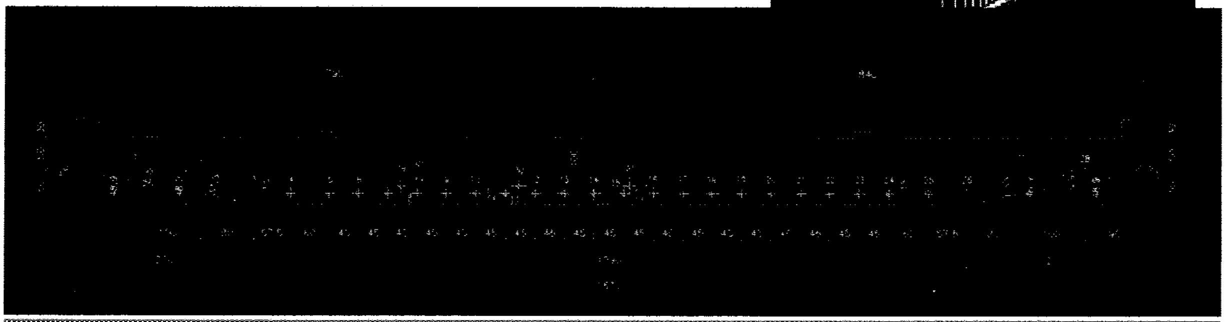
Scaffolding

Prestressing jack positioning

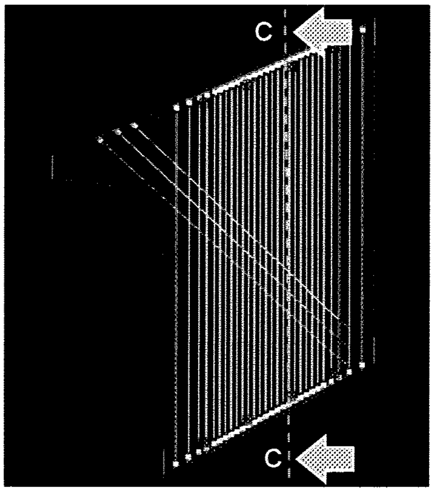


Prestressing tendons layout: cross section B-B

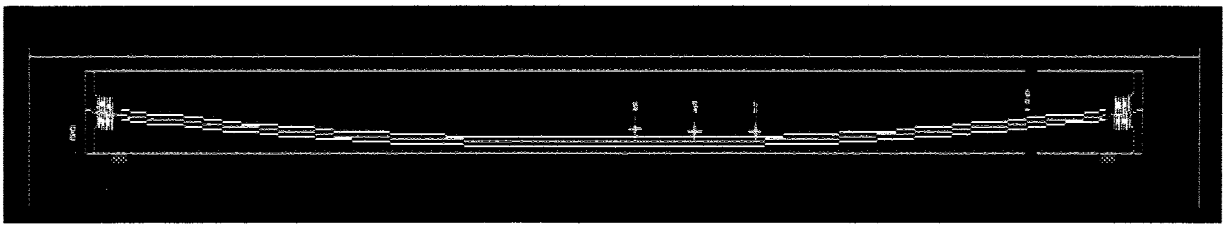




Prestressing tendons layout: longitudinal section C-C

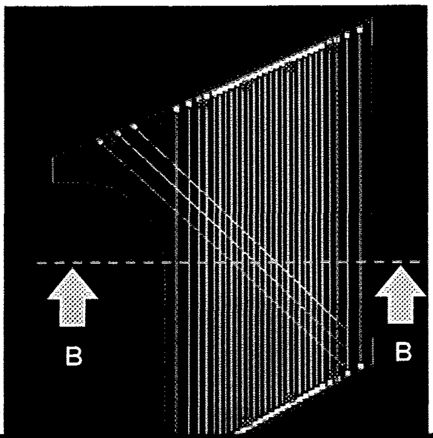


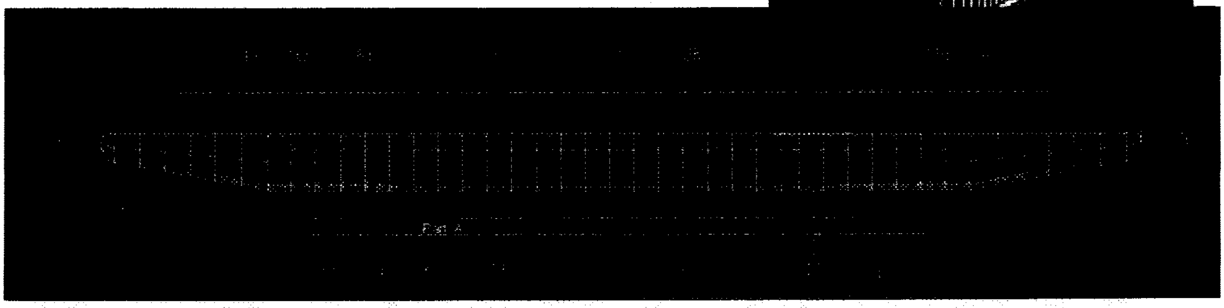
N.B. Vertical scale doubled



Ordinary reinforcement:

Cross section B-B



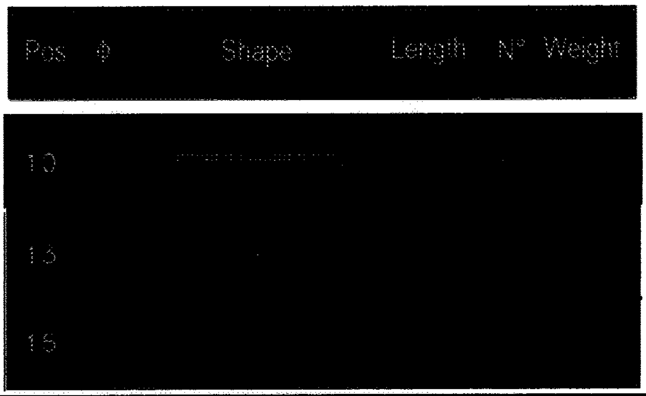


Longitudinal main reinforcement:

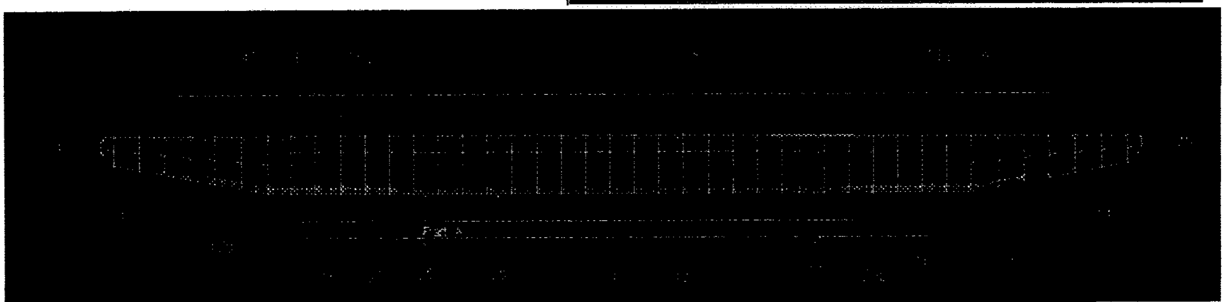
Cross section B-B

Top

Bottom

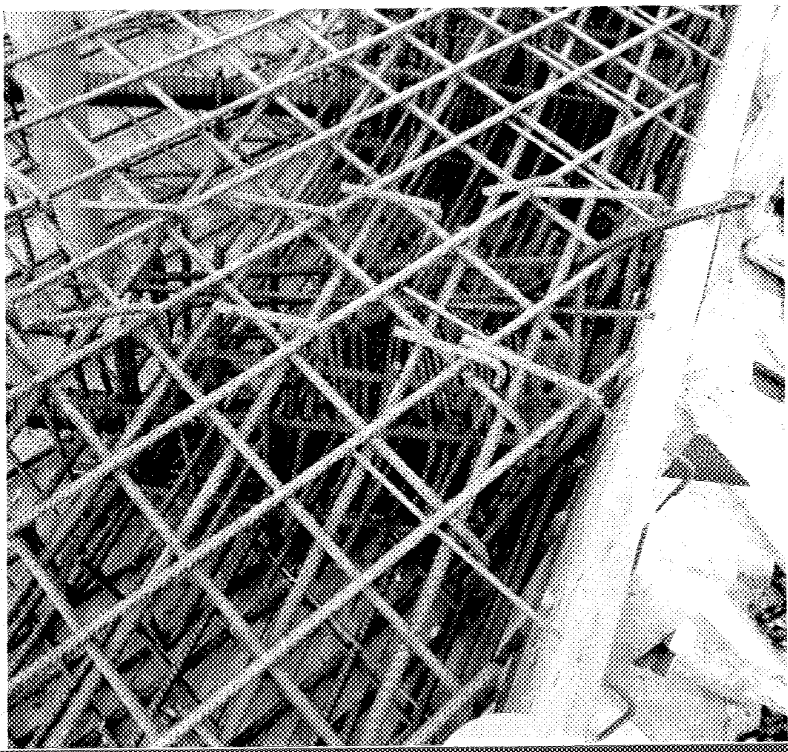


Pos	φ	Shape	Length	N°	Weight
10					
15					
15					

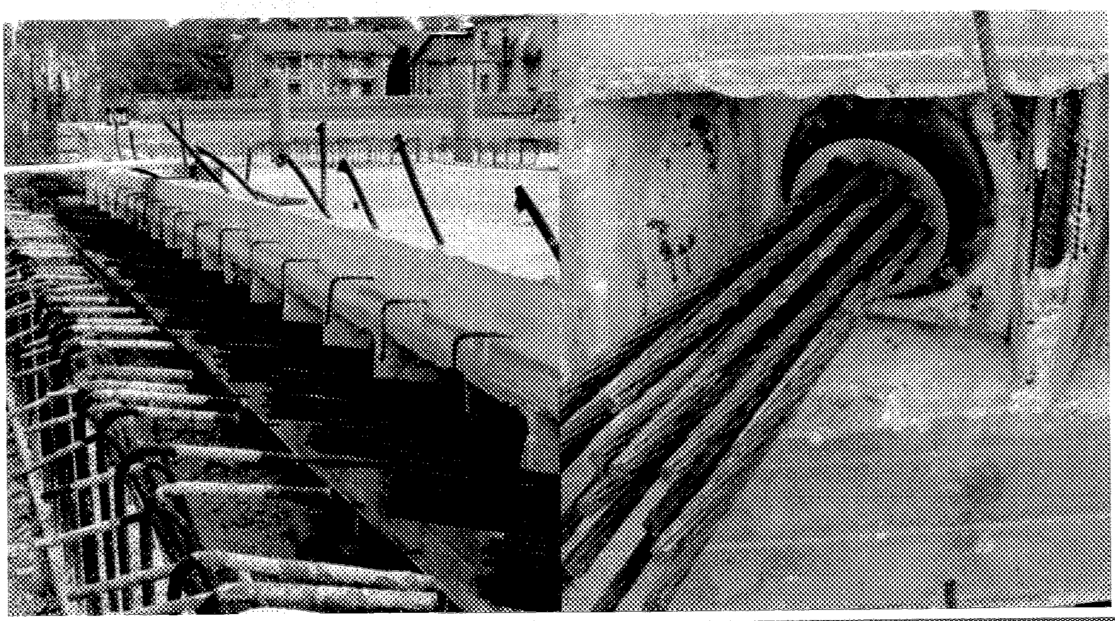


Live anchorage detail

Confinement reinforcement



Live anchorage detail – anchorage heads



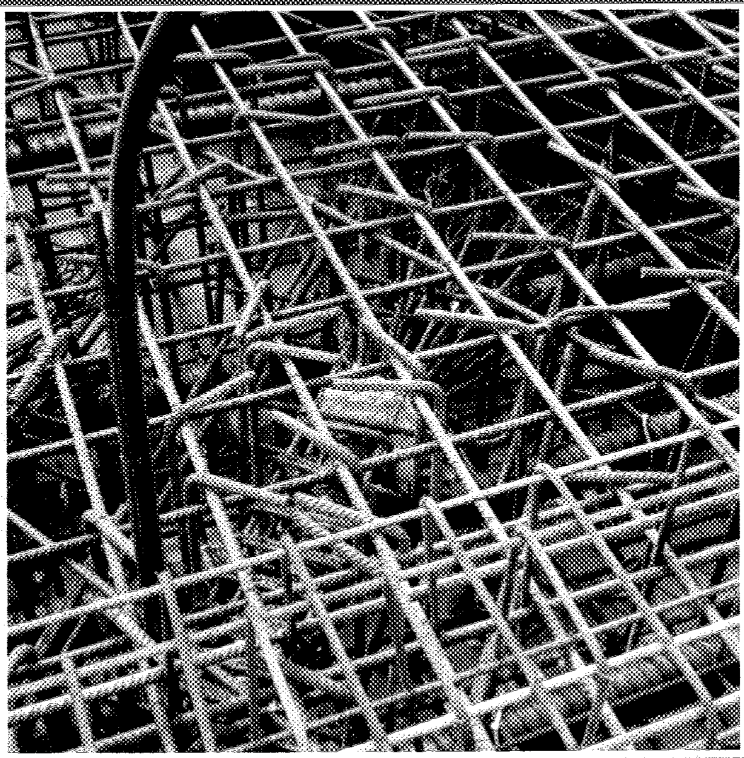
Live anchorage detail

Reinforcement description

Pos	Φ	Shape	Length	N°	Weight
8					
10					
12					
23a					
42					
42a					

Pos	Φ	Shape	Length	N°	Weight
44					
45					
46					
47					
47a					
48					

Dead anchorage detail



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Dead anchorage detail

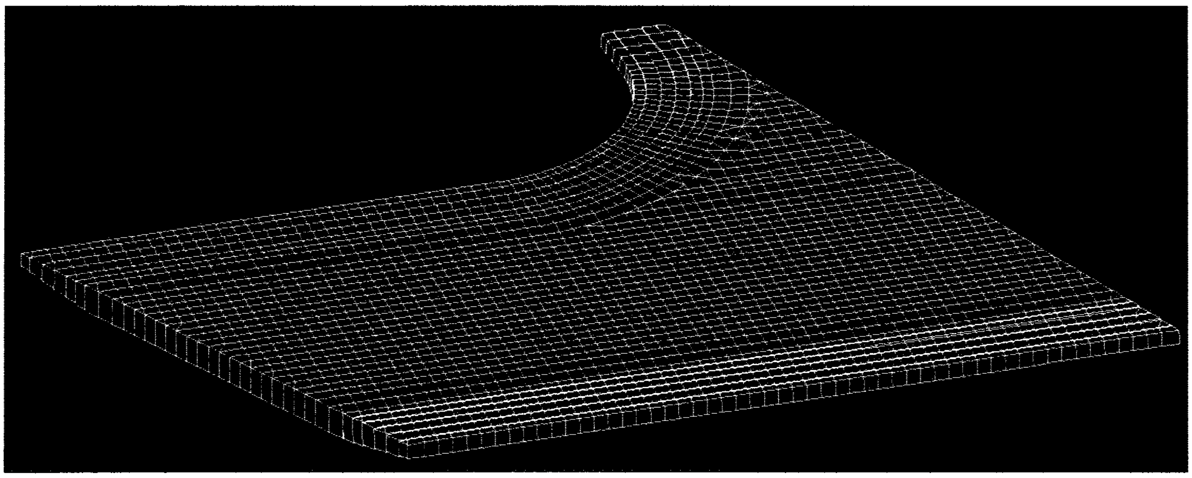
Reinforcement description

Pos	Φ	Shape	Length	N°	Weight
49	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
50	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
51	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
52	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
52a	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
53	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
54	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
55	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
56	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
58	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

RANTIVA BRIDGE

Finite element model

3D view of mesh



Guidelines to choose the mesh layout

1. Fit properly the geometry of the structure
   1.1 Avoid skew irregular elements
   1.2 Follow thickness changes and possibly main reinforcement distribution
2. Place a number of element able to describe properly the deformation of the structure
3. Choose a regular spacing between the nodes in order to be able to place live loads and prestressing equivalent loads as nodal loads (if you don't dispose of an advanced pre-processor)

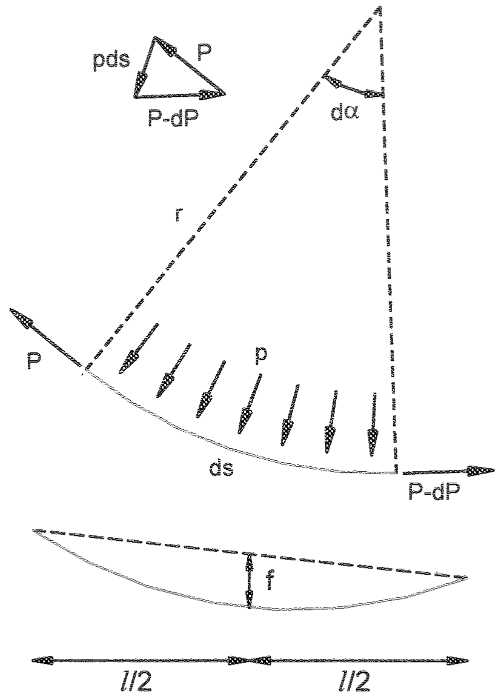
Equilibrium of an infinitesimal portion of curved tendon. The variation of axial force dP at tensioning is due to the friction between the tendon and the duct. The force exchanged between concrete and tendon is :

$$pds = p \cdot rd\alpha$$

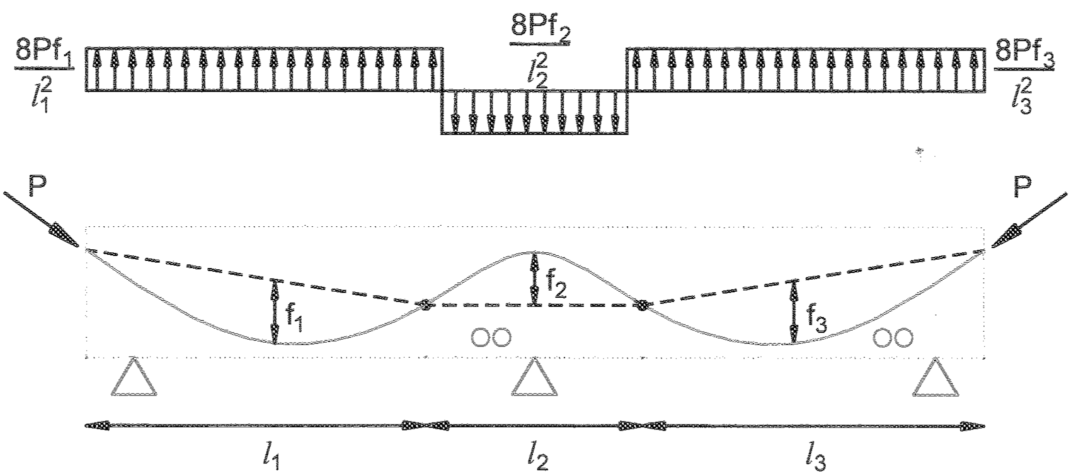
If we neglect friction we get: $p = \frac{P}{r}$

And if the tendon is parabolic we get:

$$\frac{1}{r} \cong \frac{d^2y}{dx^2} = \frac{8 \cdot f}{l^2} \qquad p = \frac{8 \cdot f \cdot P}{l^2}$$



For a simple continuous beam with a 3 parabolic tendon paths we get:



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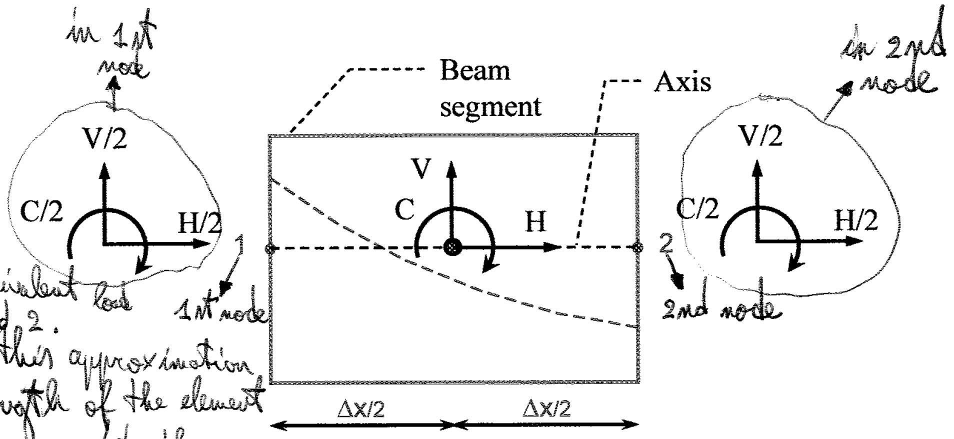
The actions H, V and C placed in the middle of the segment can be substituted by equivalent ones placed at the extremities.

It can be useful if we have finite element nodes coincident with the extremities of the segment



We split the equivalent load in nodes 1 and 2.
You can do this approximation if the length of the element is small compared to the total length of the bridge.

With shell elements things become more complicated….

With rectangular elements we can have 3 different situations:

1. Tendon passing trough two nodes of the same side (this is the situation of the 28 longitudinal tendons of our slab) (look at slide 47/63)
2. Tendon passing trough two opposite nodes of the same element (look at slide 48/63)
3. Tendon crossing the element in a general position. (slide 49/63)

the worst situation

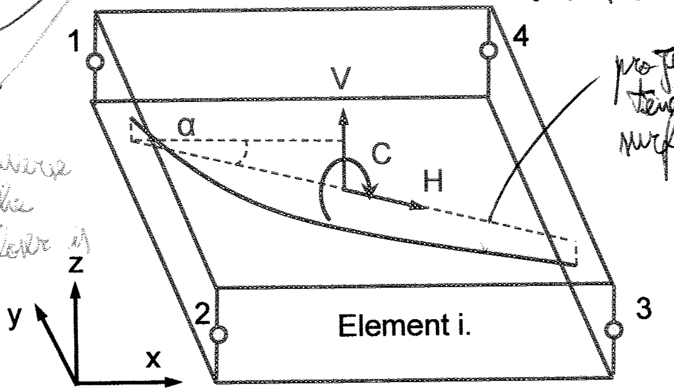
not on a face, not on a diagonal, but

3. Tendon crossing the element in a general position

The actions H, V and C placed in the middle of the projection of the tendon segment on the centroid surface of the element should be firstly decomposed in their x y and z component and then distributed on the nodes according to the shape functions of the element, or, by simplification, as a function of the distance from the nodes. "we'll see in calcolo automatico "course".

general we don't know it (the software doesn't give to us)

the relation is inverse proportional to the distance (the closer is the distance from the nodes the bigger are the actions in that node).

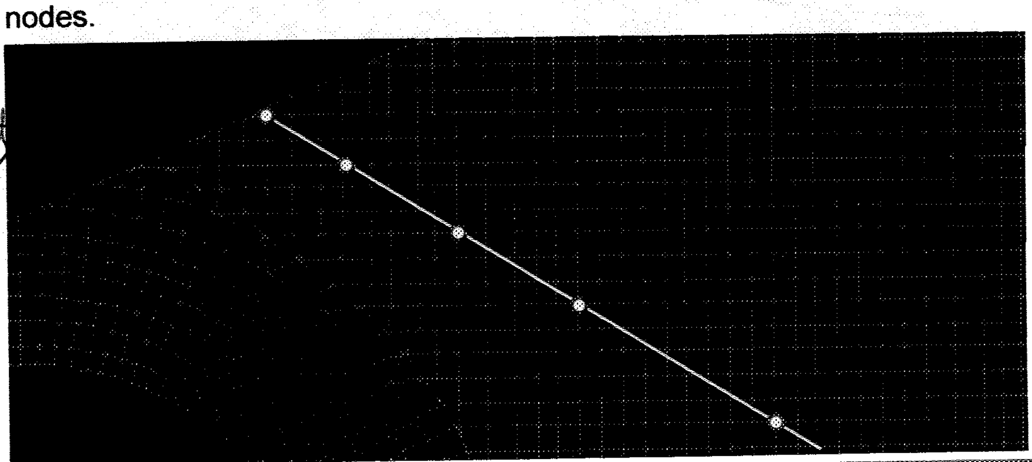


projection of the tendon on the centroid surface of the element.

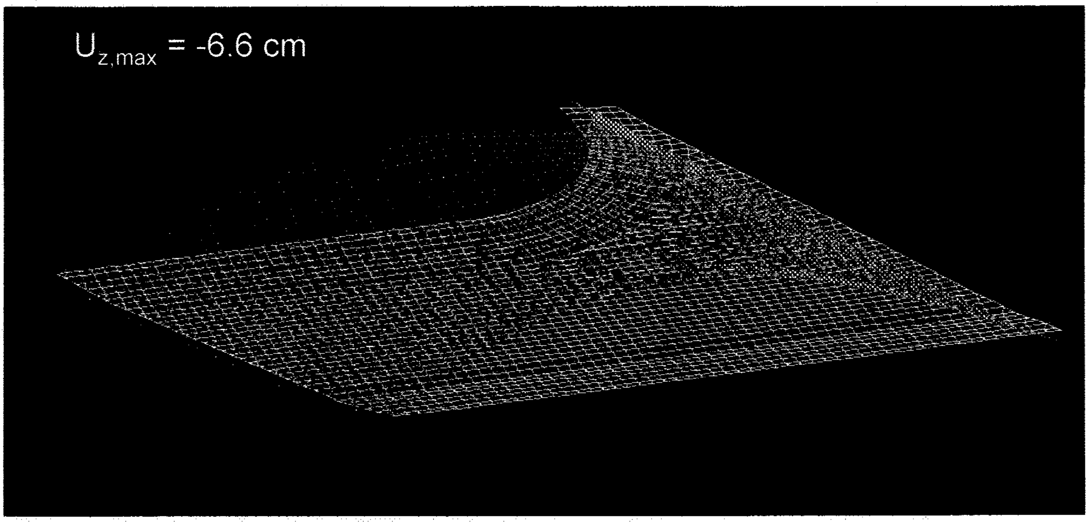
3. Tendon crossing the element in a general position

A rough approximation (but not a big mistake) of the result achieved with the method proposed in the last slide can be achieved by adopting the solution 2 only in the nodes effectively crossed by the tendon "jumping" the elements not crossed along nodes.

Equivalent loads weren't calculated element by element but on every segment in the slide and then splitted in every node (•).

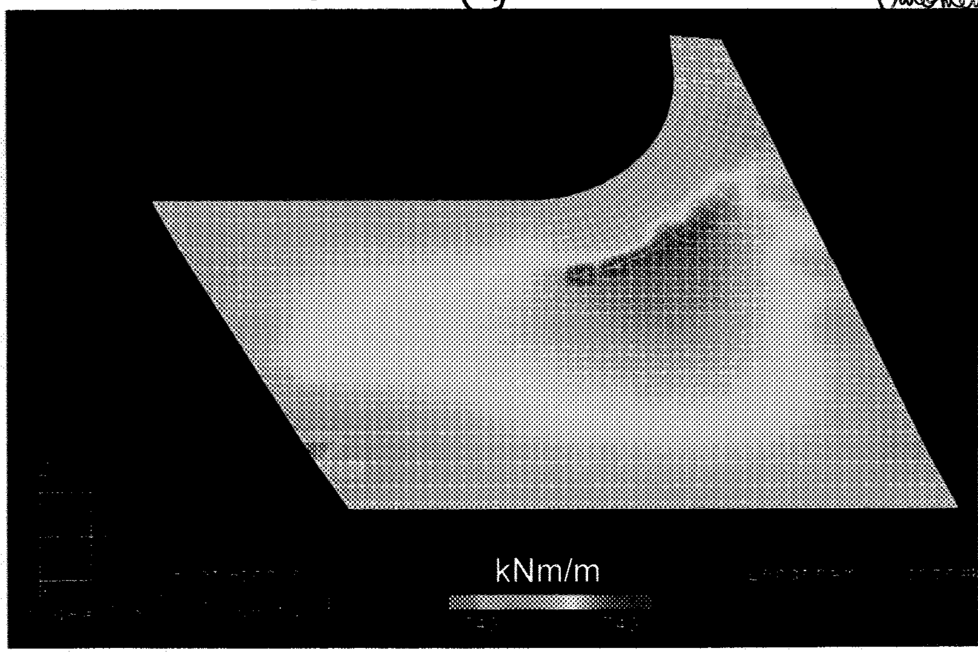


Deformation due to prestressing (end of construction)



Bending moment m_x due to dead weight

give rise to σ_x stresses (x is ↑ ⇒ m_x is transverse bending moment)

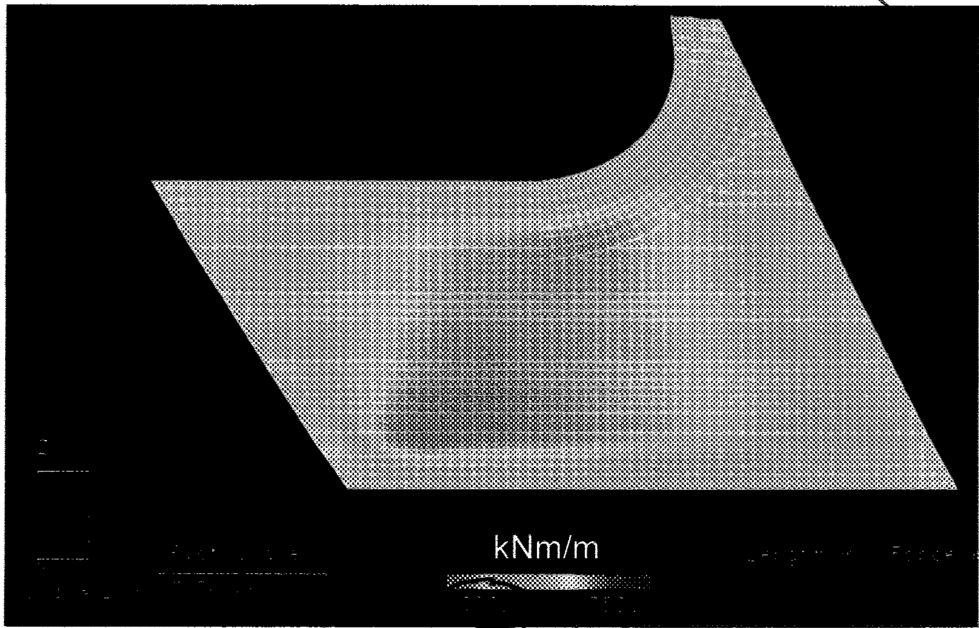


The biggest transverse bending moment is in the skew part.

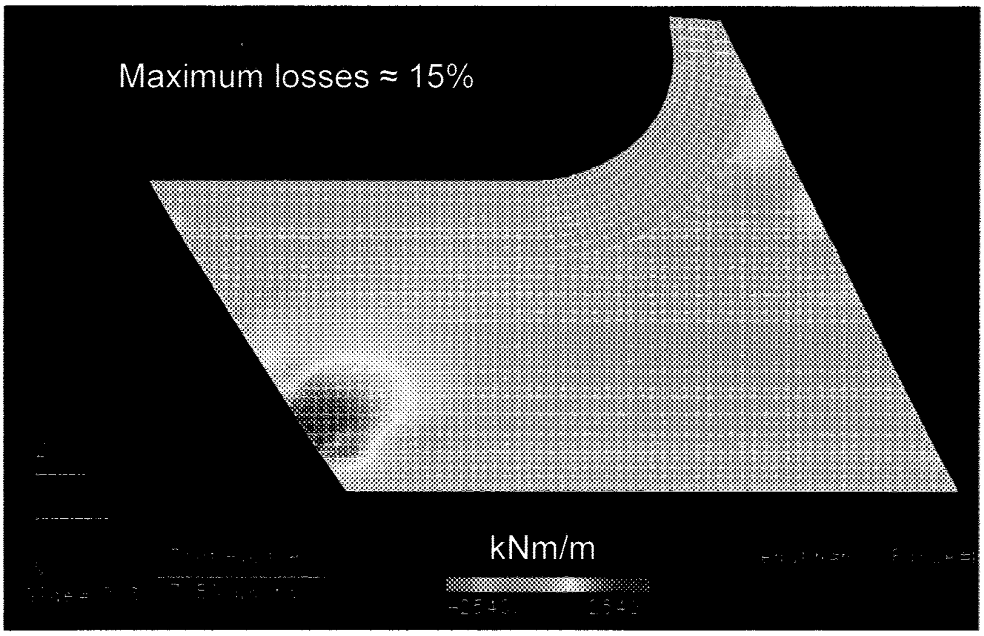
The m_x decreases where the thickness (⇒ the stiffness) of the slab decreases.

As almost all longitudinal

Principal bending moment m_2 due to prestressing (end of construction)



Principal bending moment m_1 due to prestressing ($t = \infty$) with prestressing losses



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SECONDINO VENTURA BRIDGE (ASTI)

Incremental launching continuous beam

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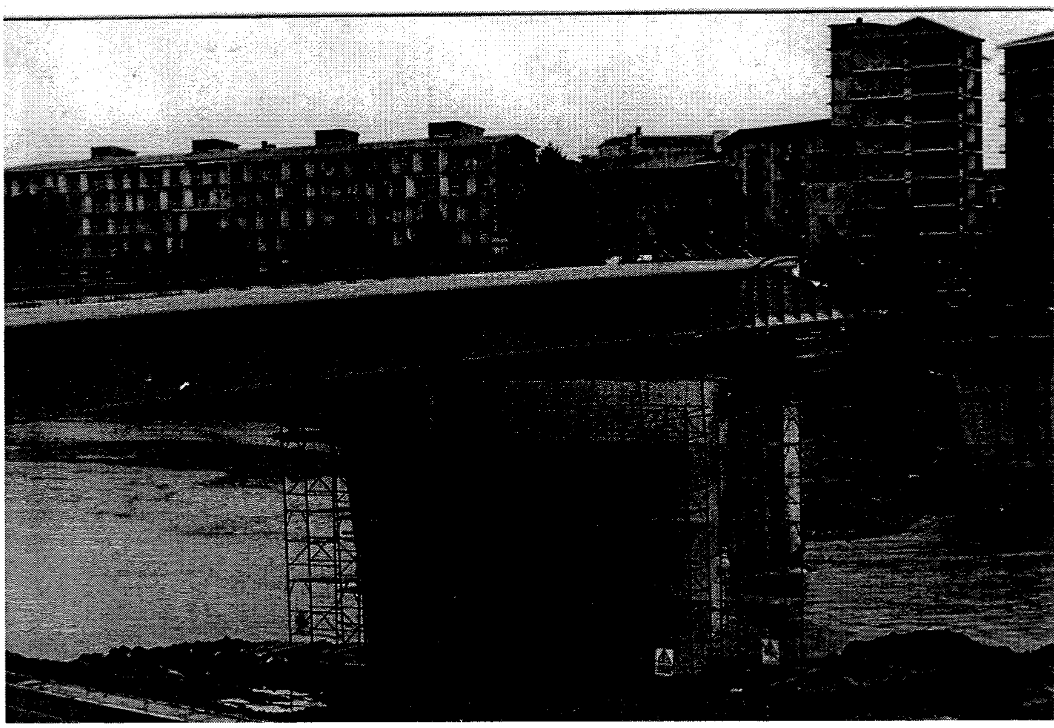

nose: It's made of steel. The bridge is pushed by one side by adding elements and the push.

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2 railway tracks

2 possible solutions: bridge is working both for railway and car traffic, or we need to separate bridges.

In the past there was a bridge built on 2 level (1 for cars and 1 for railways:). It was destroyed by the river ⇒ rebuilt.

SECONDINO VENTURA BRIDGE

What could it have been the typical solution?

SECONDINO VENTURA BRIDGE

A little history

A LITTLE HISTORY ...

- Flooding in Piedmont in 1994 concerned principally Tanaro basin with a flow measured in Alessandria of about 3800 m^3/s
- Old Corso Savona bridge in Asti was made of a upper way road deck, realized with 4 prestressed precast concrete beams with cast in situ slab of about 20 m span, and a lower railway deck made of steel
- Both the decks were supported by huge masonry piers that left very little free span between them.

huge quant of water

furthermore:
- 6 m/s the speed
- density of 1,5 ton/m3 (not only 1 ton) because there were debris inside water.

SOLUTION to respect all the previous restraints.

Both road and railway decks made of prestressed concrete.

Two continuous beams with 5 spans each

(end spans 29.70 m and central spans 33.20 m),

Incremental launching was used ⇒ only the piers were built with construction workers in the river bed (⇒ only small time in the river bed)

Total depth of the beams = 165 cm (l/h≈20).

Diaphragm piers with a transverse thickness of 150 cm.

smaller beams and bigger clearance for water than l/h = 15.

30 cm of clearance means 8 % of the total amount of the water that can pass under the bridge.

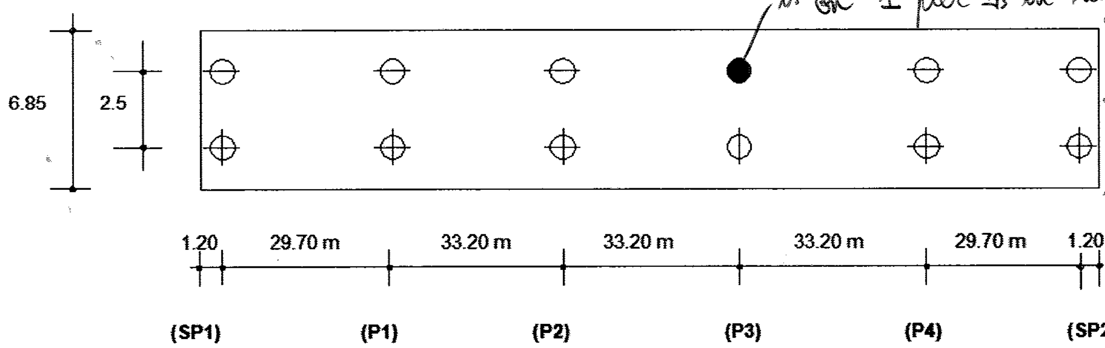
BEARINGS

in both direction

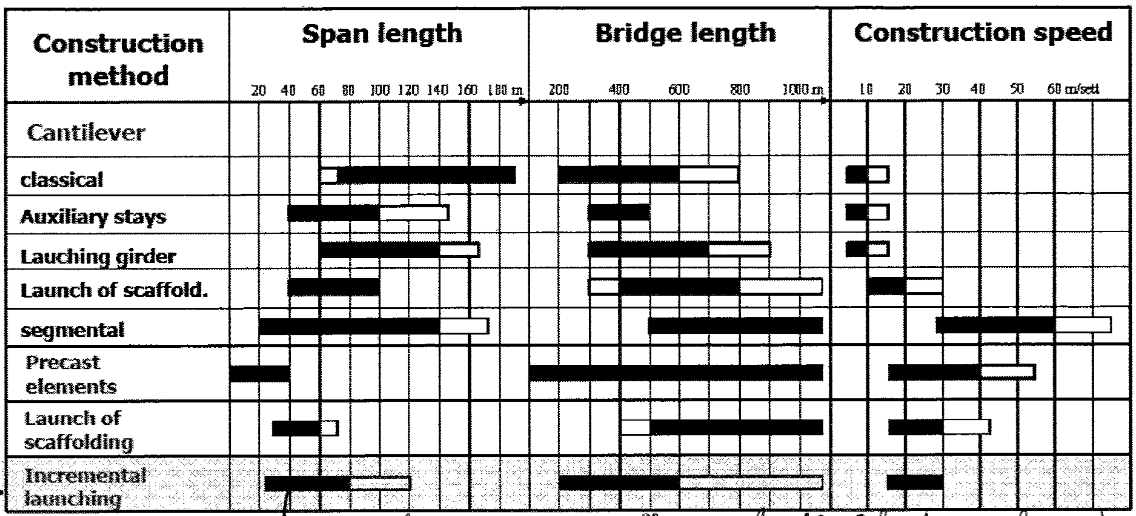
Free = ⊕ Long. Free / Transv. fixed = ⊖

Fixed = ● Long. fixed / Transv. free = ⦶

is on 1 pier ⇒ in the 2 abutment we don't have fixed bearings this is a way to minimize thermal elongation, in order to don't have track interaction problem with the joints



Comparison between construction techniques



the railway imposes a small span length (if it was only road, it could be much bigger.)

- Construction of one span (33 m) in ten days → 2 working weeks
- for 33 m span
- Lauching time: 3 hours → the launching speed was about 10 m in an hour

SECONDINO VENTURA BRIDGE

Launching technique

Geometrical limitations:

In vertical plane	In horizontal plane
horizontal	straight or circular
circular	straight
linear inclination	circular
circular	circular

(in our case it was made linear inclination (the inclination of the railway, that is very small ⇒ almost linear/horizontal deck)

In the last two cases the projections on the horizontal plane are ellipse circles

NOSE DESIGN

We can assume

$$L_n \cong 0,65\ L$$

The longer the nose, the biggest are the saves in the concrete, reinforcement

L_n= nose length
L = typical span of the bridge (temporary or final)

$$q_n = k\ L_n^2$$

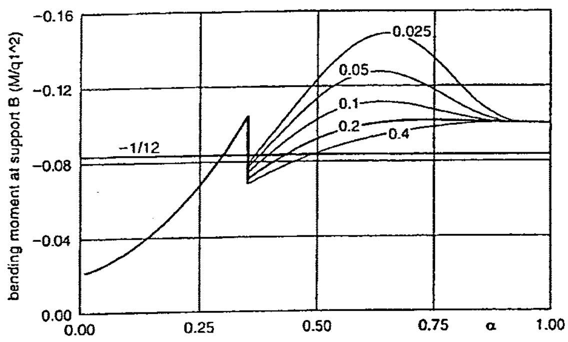
General you reuse the same nose fore more bridges.

q_n = dead weight of nose
k = 0,012 ÷ 0,020 for road bridges
0,018 ÷ 0,030 for rail bridges

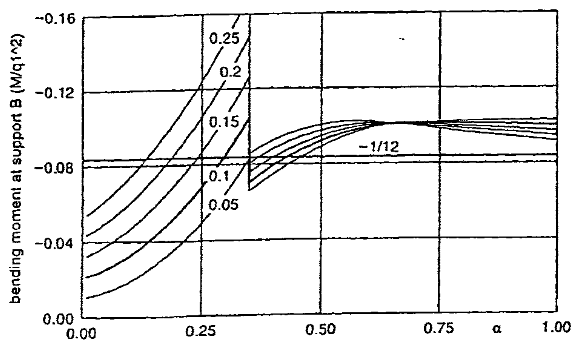
The ratio between dead weight of nose and deck can be assumed, at a first approximation, as:

$$q_n/q = 0,10$$

The effect of relative flexural rigidity E_nI_n/EI on the limitation of stress variation during the launching should be evaluated.



With q_n/q = 0,10 the bending moment at maximum cantilevering is equal to EOL for L_n/L = 0,65



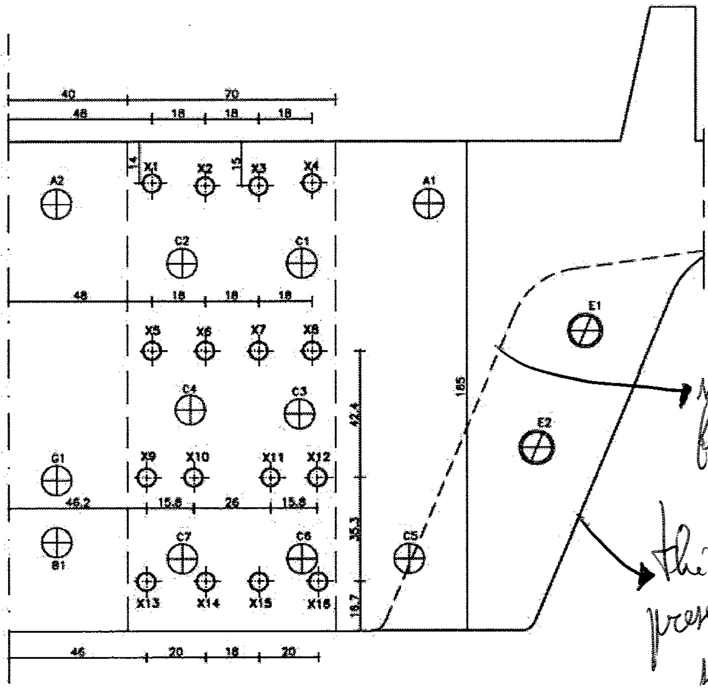
Variation of M_B for L_n/L = 0,65 and E_nI_n/EI = 0,200 as a function of the ratio q_n/q.

SECONDINO VENTURA BRIDGE

Launching nose

Section S3 (look at slide 27/66)

(2m from the nose)



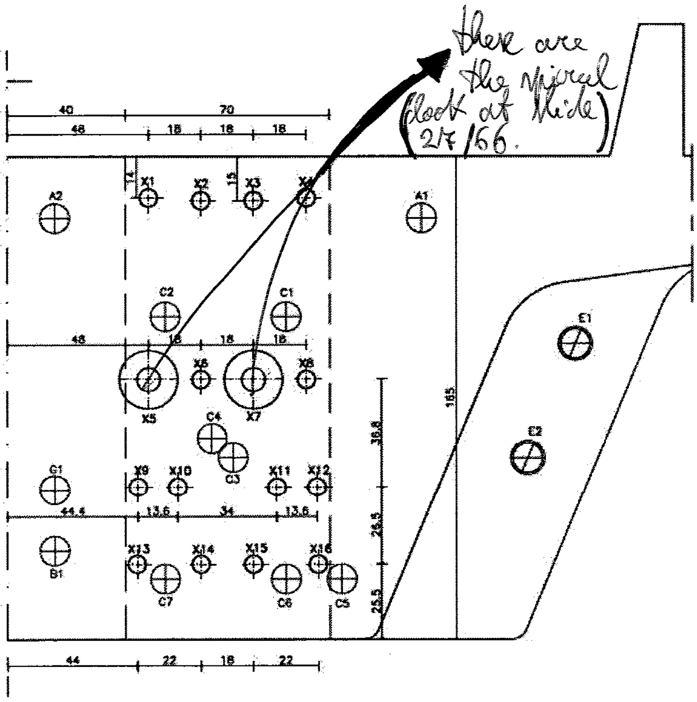
section far away from the piers.

this is the section in presence of the piers (bigger section)

Section S5

(4m from the nose)

there are the original (look at slide 27/66.)



INTERNAL ACTIONS DURING THE LAUNCHING: BENDING MOMENT

- Static scheme :

Temporary piers were present when the bridge were launched.

▲ Definitive restraint

▲ Temporary restraint

During launching, the static scheme has a span that is 1/2 of the span of the final bridge (≅ 15m)

- Actions:
  - ➢ Self weight
  - ➢ Temperature variation between intrados and extrados of ± 5° (that can be positive or negative)
  - ➢ Maximum differential settlement between two consecutive bearings of 5 mm

5mm over 15metres is not small quantity.

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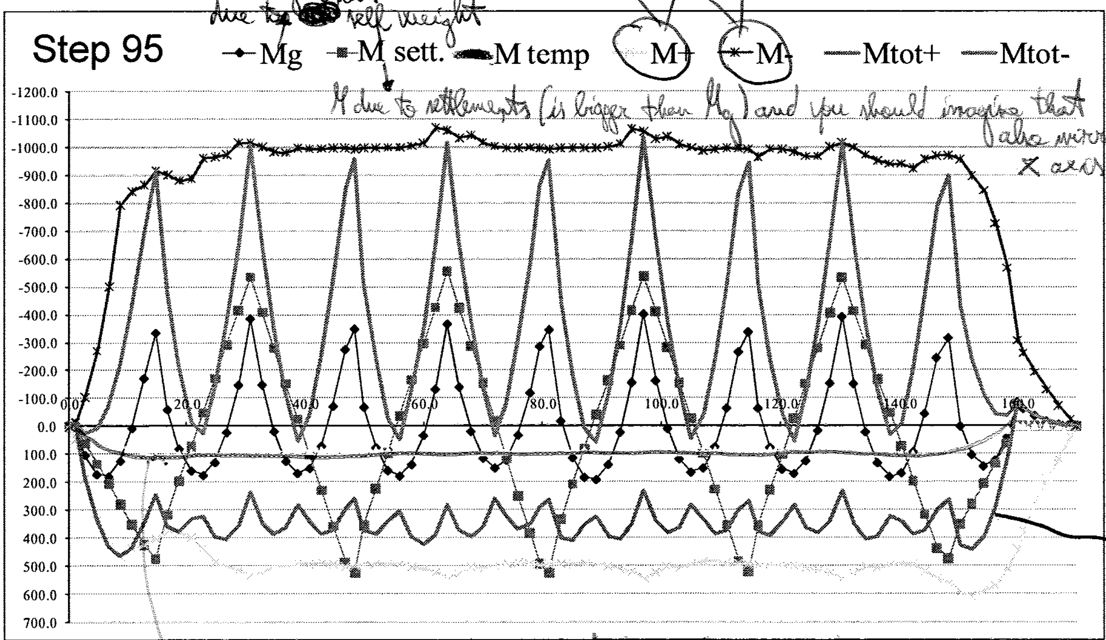
They don't refer to the end of launching (like the other diagrams)

they came from the fact that, during launching, each section was moving (look at next slide)

Bending moment at end of launching (values in kN*10*m)



bending moment due to self weight

M due to settlements (is bigger than Mg) and you should imagine that Msett. is also mirrored about x axis

maximum M+ (factored)

is for 1 sign of temperature ⇒ it should be mirrored

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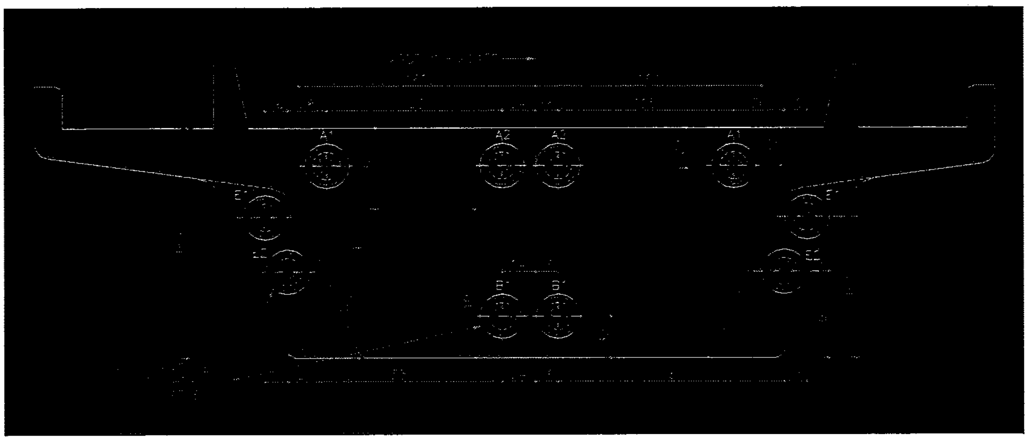
As the bending moments are almost constant in all the sections and the positive values are only half of the negative ones baricentric prestressing is introduced for the launching phases.

This layout of tendons is more or less baricentric: we need big axial force to contrast M+ and M-.



Enlarged section				
A [m²]	$W_{sx,sup}$ [m³]	$W_{dx,sup}$ [m³]	$W_{sx,inf}$ [m³]	$W_{dx,inf}$ [m³]
7.897	-2.828	-2.631	2.171	2.237

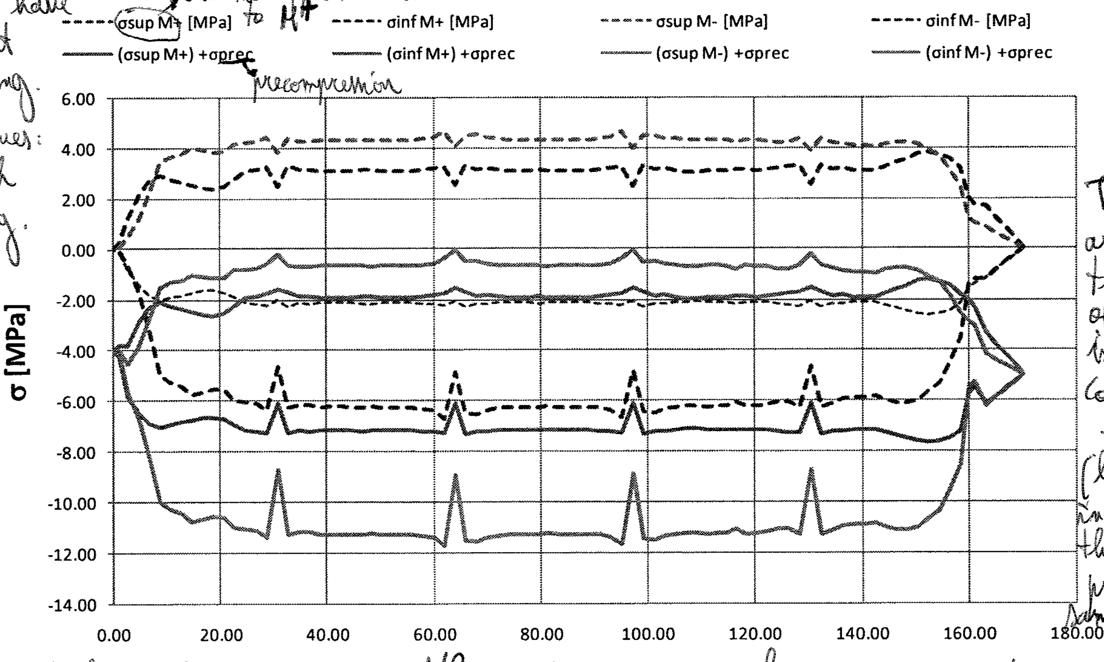
Current section				
A [m²]	$W_{sx,sup}$ [m³]	$W_{dx,sup}$ [m³]	$W_{sx,inf}$ [m³]	$W_{dx,inf}$ [m³]
6.458	-2.498	-2.290	1.590	1.629

Longitudinal stresses during launching (max. ≤ 1:20)

Dashed lines: stresses that you would have without prestressing.
Continuous lines: stresses with prestressing.

σ at the upper side due to M+

precompression

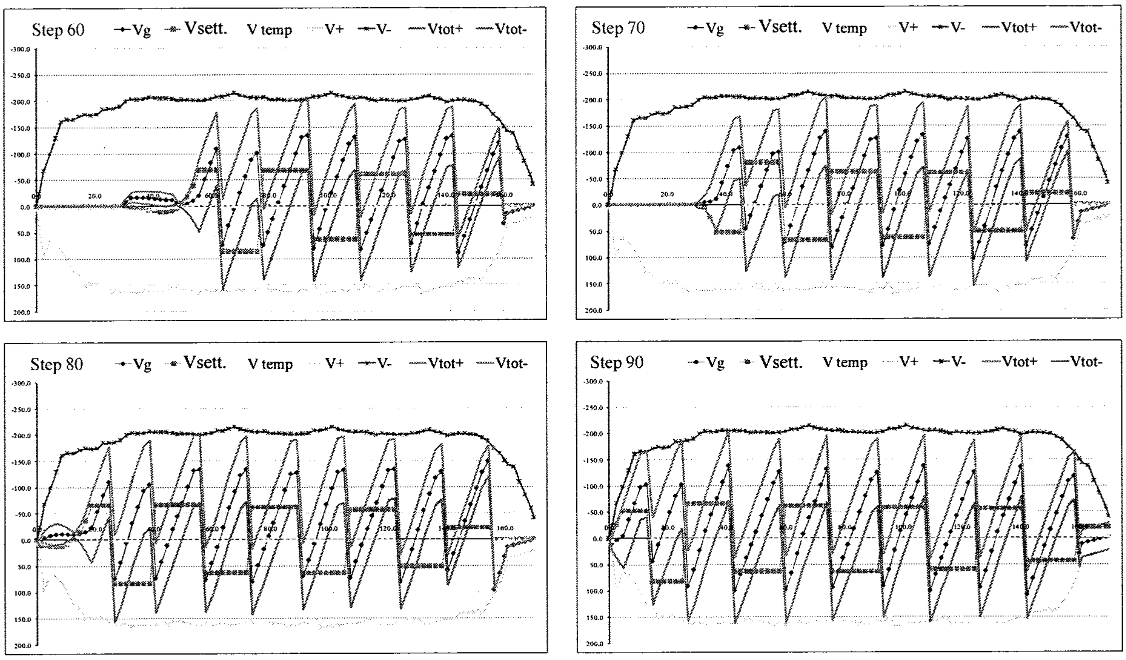


The peaks are because the section of the bridge is not constant everywhere (bigger section in presence of the pier) → if prestressing is the same → the effect of prestressing decreases where the section is bigger

The effect of prestressing is 4,5 MPa added everywhere in compression
4,5

Shear during launching

We have shear constant almost everywhere (like bending moment): this is typical of incremental launching bridges)

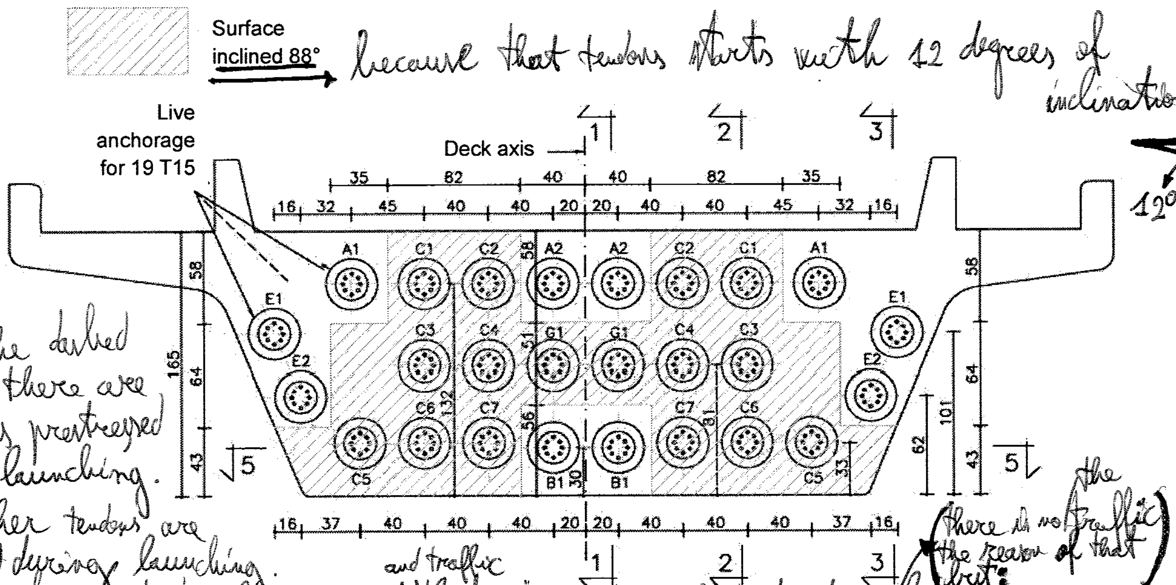


SECONDINO VENTURA BRIDGE

SLS Verifications

Prestressing layout – section AA

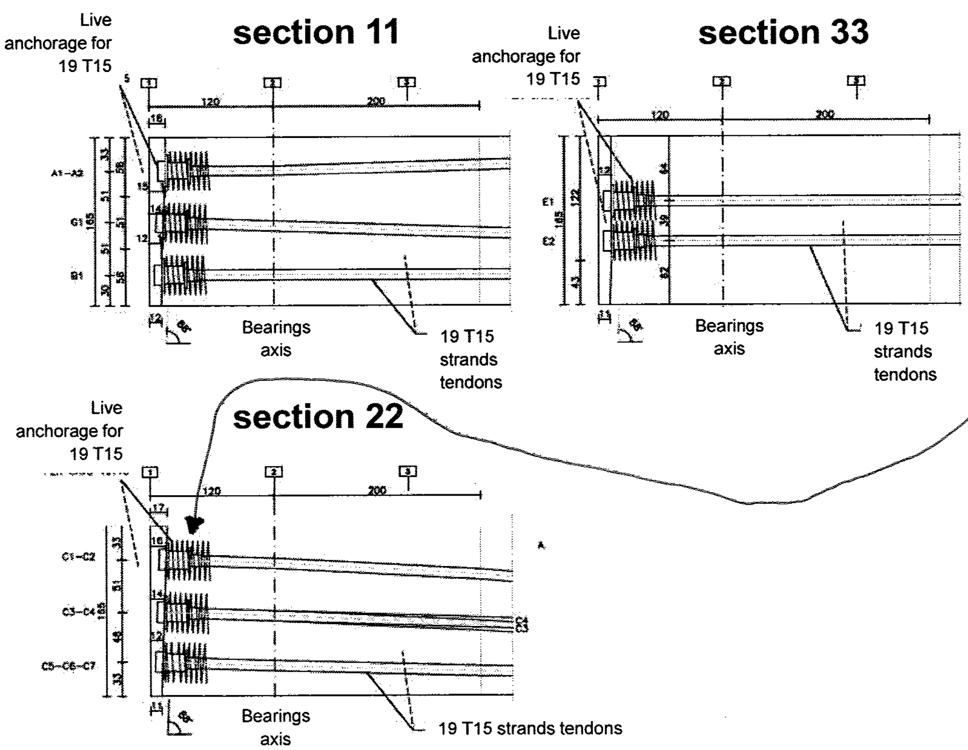
because that tendons starts with 12 degrees of inclination.



In the dashed zone there are tendons prestressed after launching. The other tendons are used during launching.

(there is no traffic the reason of that but:)

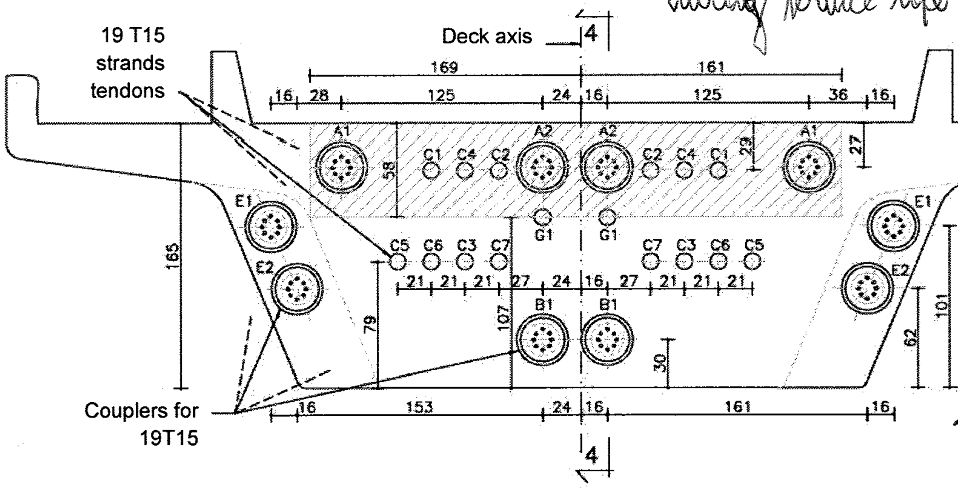
There are much more tendons for permanent loads and traffic than for the construction, because during launching we have 15 m span, whereas after construction we have 30 m span (the span double, the bending moment increases 4 times)



Prestressing layout – section CC

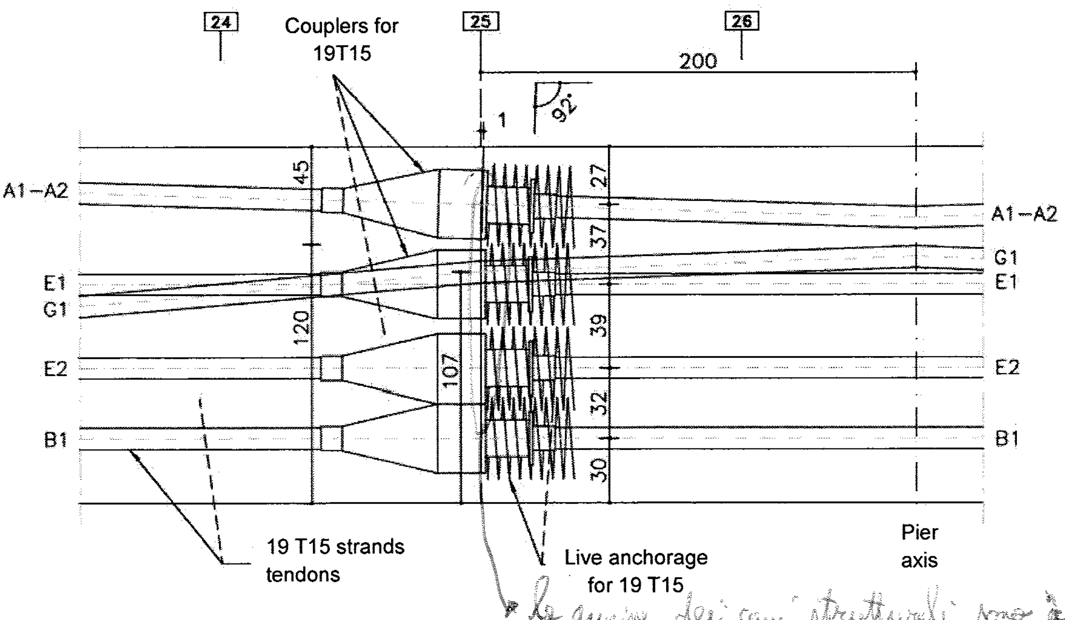
(look at slide 44/66)

the tendons used during launching are coupled. The tendons used during service life are not coupled. All the launching tendons are coupled at the end of every segment (you save money because you use the same scaffolding for every segment).



section 44

The final tendons (length 180m)



Le guaine dei cavi strutturali sono accoppiate qui con un po' di scotch

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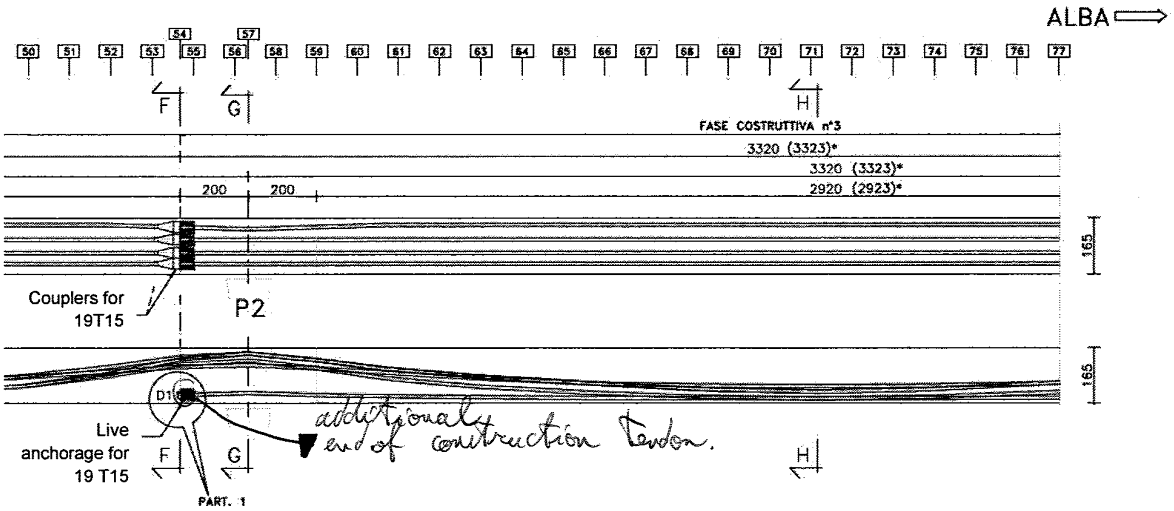
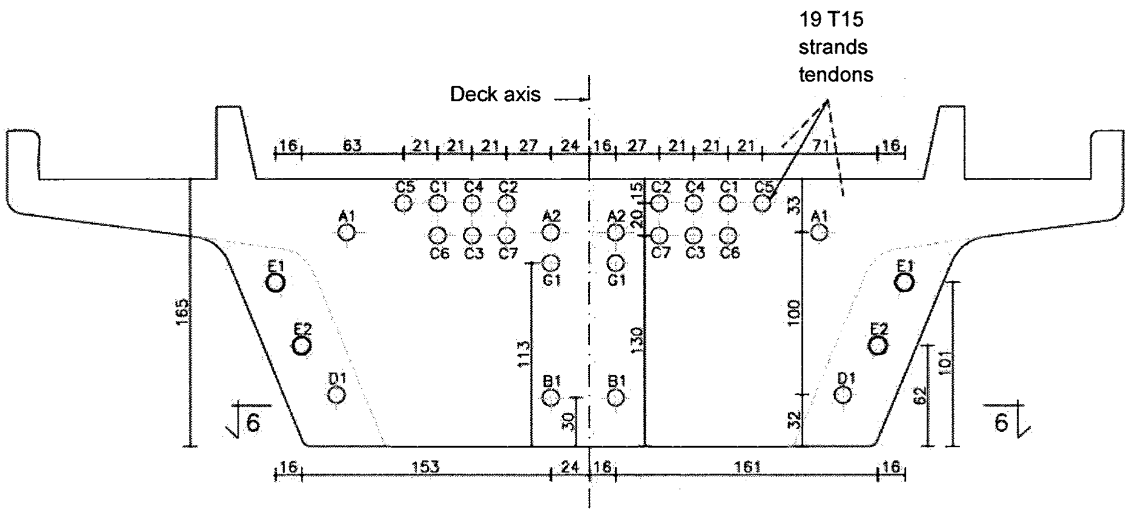
Prestressing layout – 3rd span

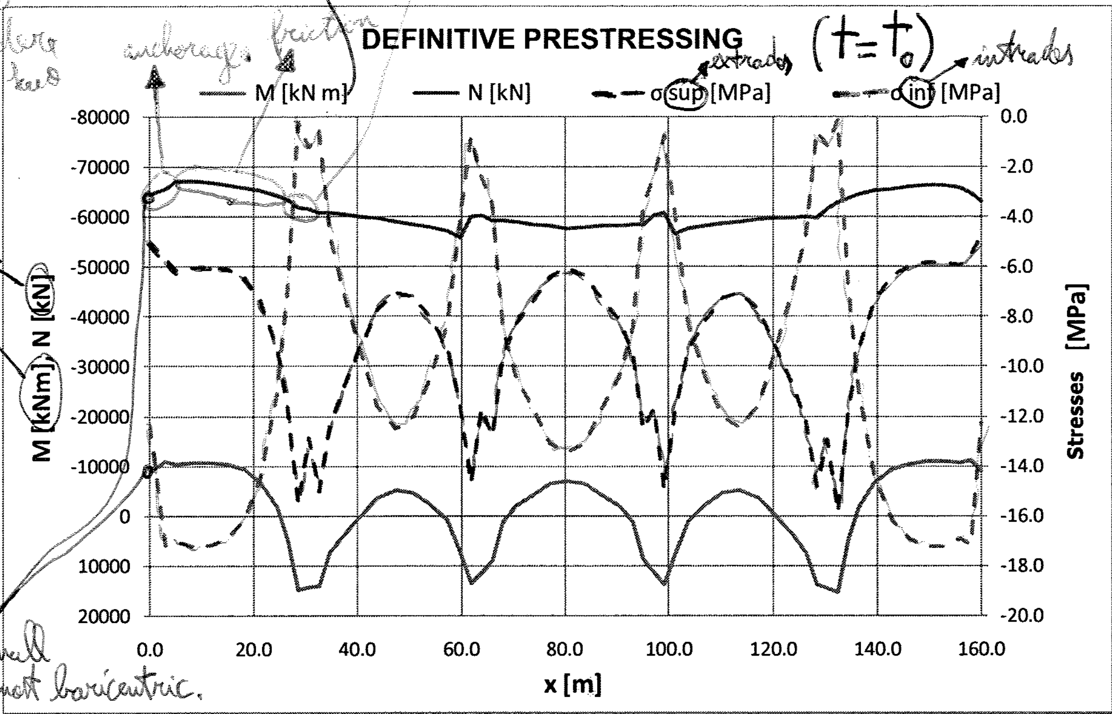


Prestressing layout – section GG



gives you idea of the bending moment due to tendons that are curved negative quickly → rapid decrease

Internal actions (M,N) and relative stresses



It's possible to plot together KN and KN·m if the internal level arms of the internal actions are about 1m.

All the stresses are in compression

10/65 = very small → the N is almost baricentric.

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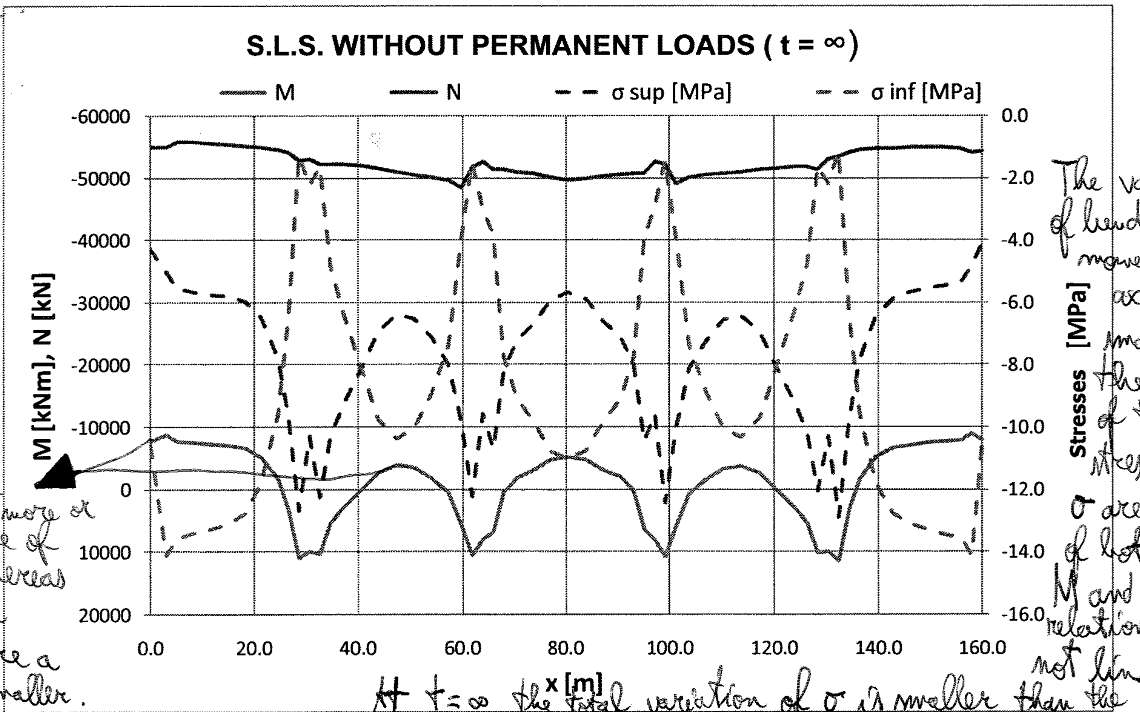
Internal actions (M,N) and relative stresses



The negative values are more or less the same of before, whereas the positive values are a bit smaller.

The variation of bending moment and axial force, smaller than the variation of the stresses because σ are combination of both N and M and this relationship is not linear.

At t=∞ the total variation of σ is smaller than the variation of σ at t=t0

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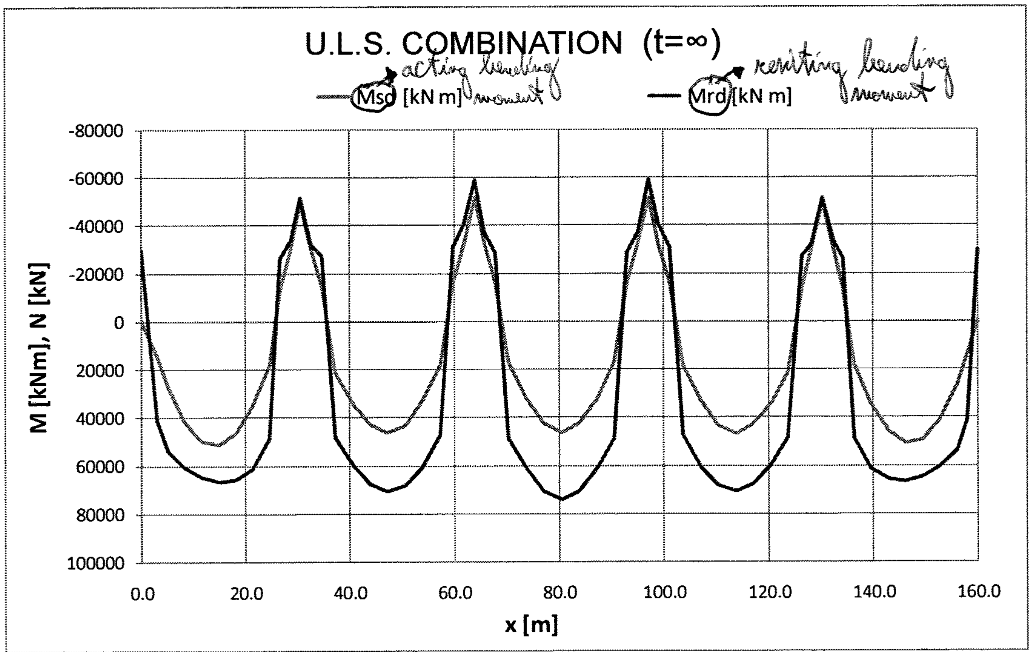
SECONDINO VENTURA BRIDGE

ULS Verifications

Bending moment diagram
(excluded isostatic internal actions due to prestressing)



Resistance of web compression fields ($V_{Rd,max}$) is modified to take into account the interaction between longitudinal and inclined compression:

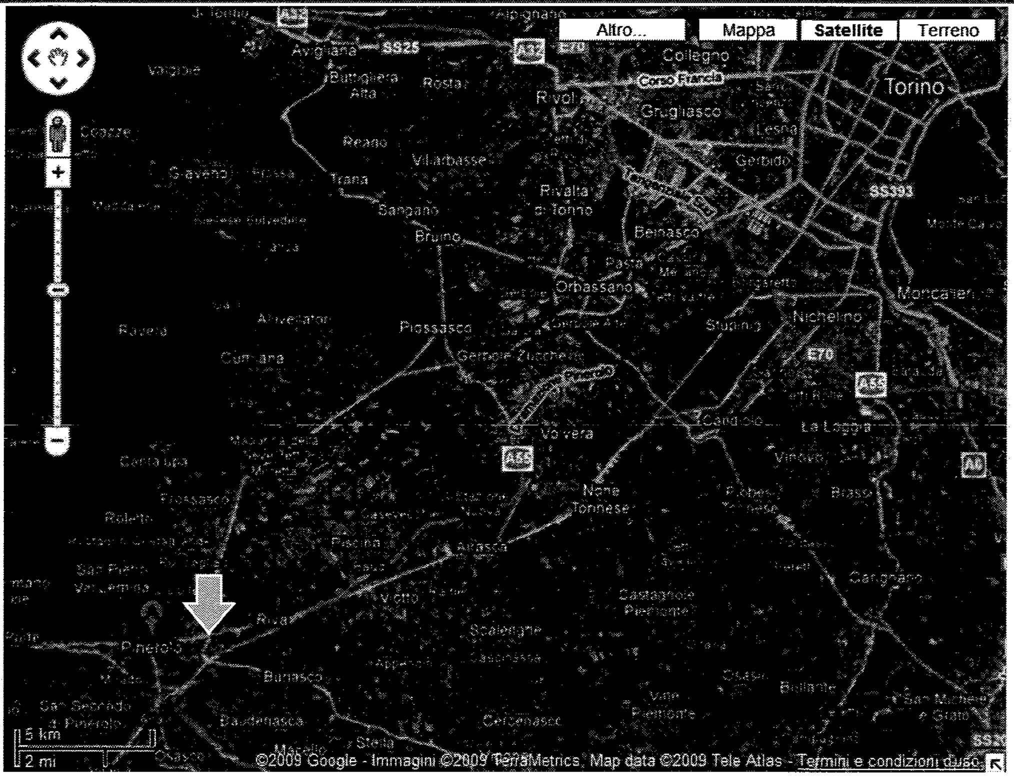
$$V_{Rd,max} = \alpha_{cw}\, b_w\, z\, \nu_1\, f_{cd}/(\cot\theta + \tan\theta)$$

$\nu_1 = 0{,}6$	for $f_{ck} \leq 60$ MPa
$\nu_1 = 0{,}9 - f_{ck}/200 > 0{,}5$	for $f_{ck} \geq 60$ MPa
$\alpha_{cw} = 1$	for non prestressed structure
$\alpha_{cw} = (1 + \sigma_{cp}/f_{cd})$	for $0 < \sigma_{cp} \leq 0{,}25\, f_{cd}$
$\alpha_{cw} = 1{,}25$	for $0{,}25\, f_{cd} < \sigma_{cp} \leq 0{,}5\, f_{cd}$
$\alpha_{cw} = 2{,}5\,(1 - \sigma_{cp}/f_{cd})$	for $0{,}5\, f_{cd} < \sigma_{cp} < 1{,}0\, f_{cd}$

Prestressing reinforcement can also be used to carry the increment of the tensile force in the tensed chord due to shear.

References

- *CEB-FIP Model Code 1990*, Thomas Telford – 1990
- *Eurocode 2 Design of concrete structures, Part 1-1: general rules and rules for buildings* - 2003
- *Eurocode 2 Design of concrete structures – Part 2: concrete bridges* - 2004



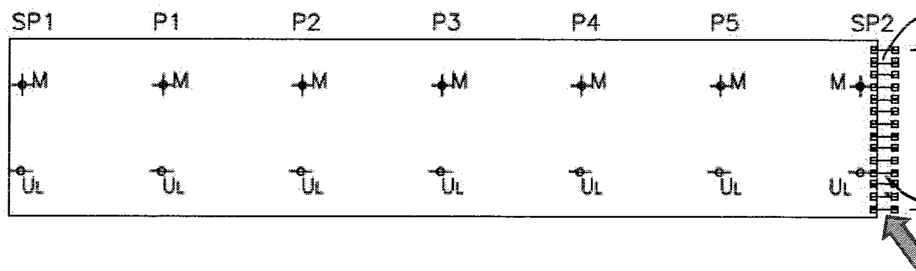
Road to Pinerolo

Static scheme: Bearings

Rubber expansion joint with 320 mm excursion

Rubber expansion joint with 50 mm excursion

It's 1 deck:



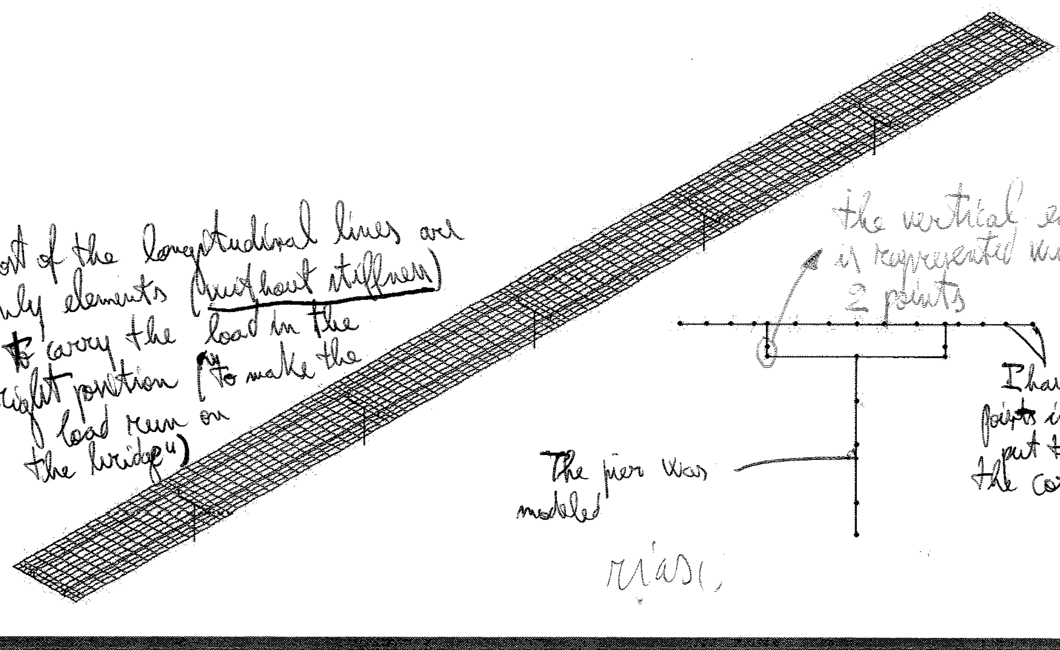
Joints with tie together the deck and the abutment.

These were the longitudinal restrains for seismic actions.

M	free in both directions
UL	fixed in transverse direction

Restrain in longitudinal direction
14 bars ϕ=70mm i=65cm

Finite element model: only beam elements were used.



Most of the longitudinal lines are only elements (without stiffness) to carry the load in the right position ("to make the load run on the bridge")

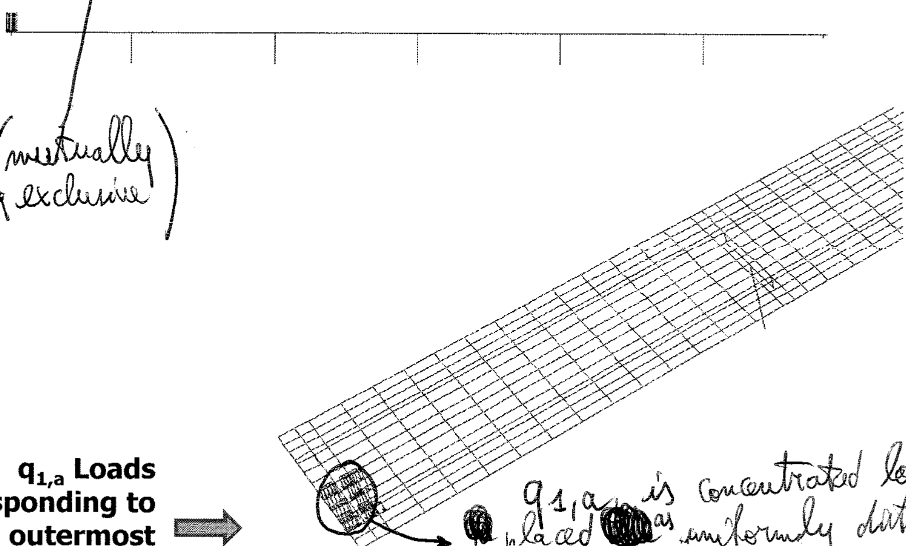
the vertical eccentricity is represented with this 2 points

I have a lot of points in order to put the load in the correct position

The pier was modeled

$q_{1,a}$ has been placed 64 times in different longitudinal positions on the girder model with stepping of 3 m (64x3 = 192m ≅ length of the bridge 189)

(mutually exclusive)

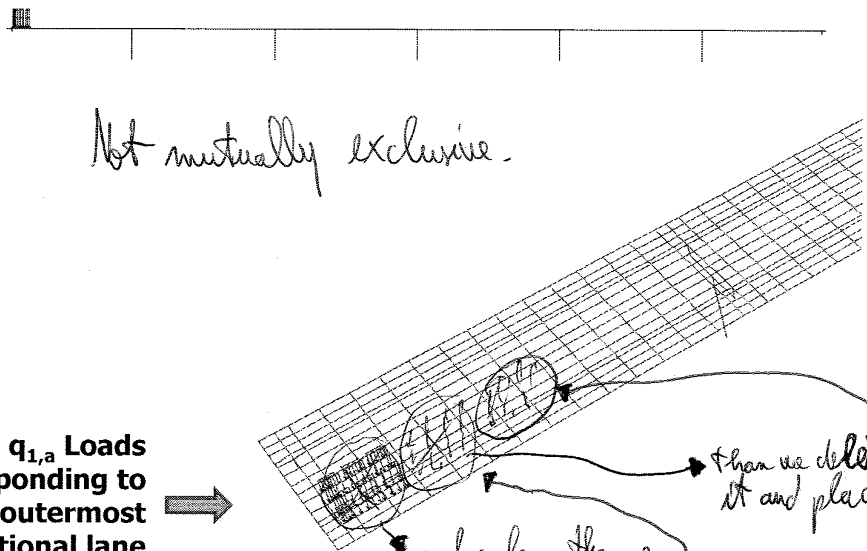


$q_{1,a}$ Loads corresponding to the outermost notional lane

$q_{1,a}$ is concentrated loads, but is placed as uniformly distributed load, because of dispersion.

$q_{1,b}$ has been divided into 3x3m squared areas and placed 64 times in different longitudinal positions on the girder for each notional lane

Not mutually exclusive.

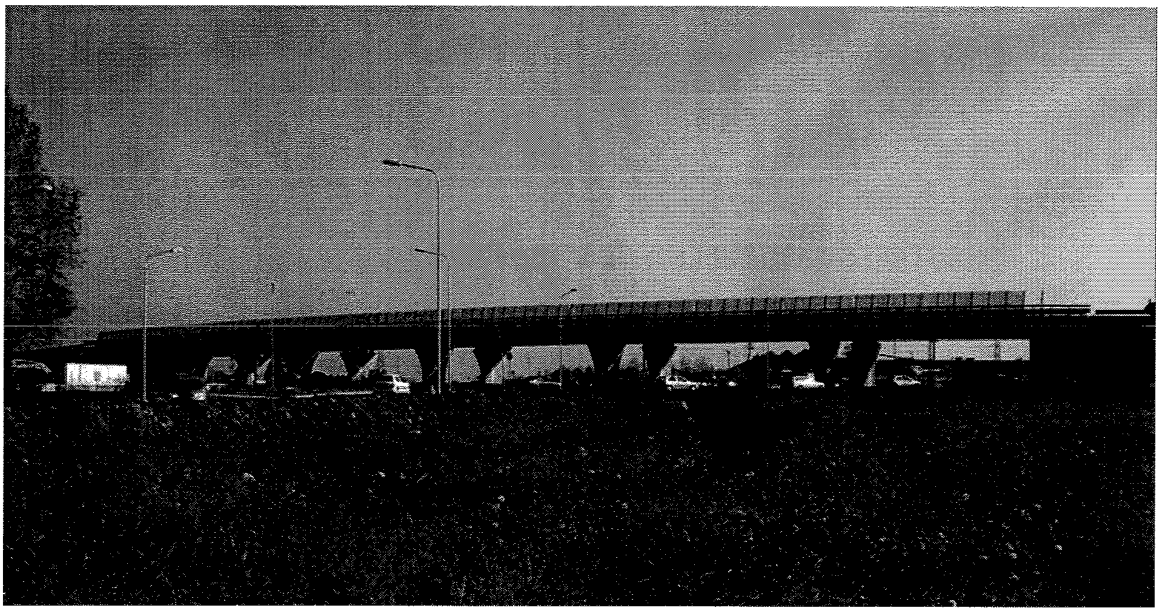


then we delete it and place

we place here then we delete it and place

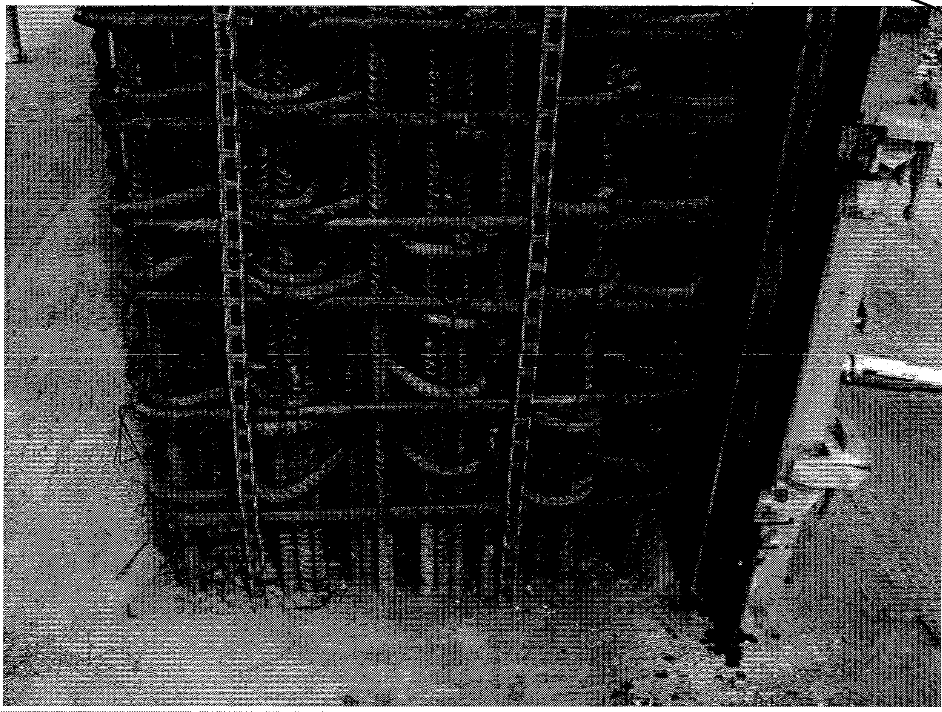
$q_{1,a}$ Loads corresponding to the outermost notional lane

Completed bridge



Completed bridge

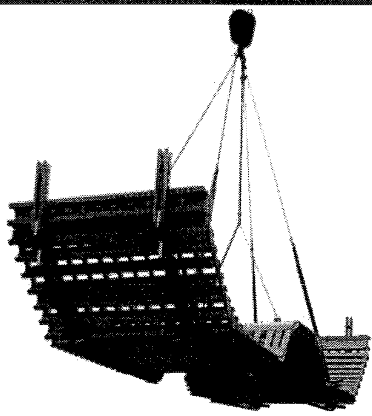
Detail of reinforcement cage at the foot ~~of the pier~~ Marc.



The base of the pier

We have so much reinforcement at the base of the bridge, because the pier is not tall (is short) → very rigid.

Scaffolding realization

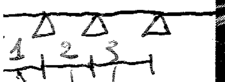


Steel formwork

Reinforcement just before concreting

Reinforcement cage.

3 steps of scaffolding:



1 2 3

scaffolding in these 3 parts

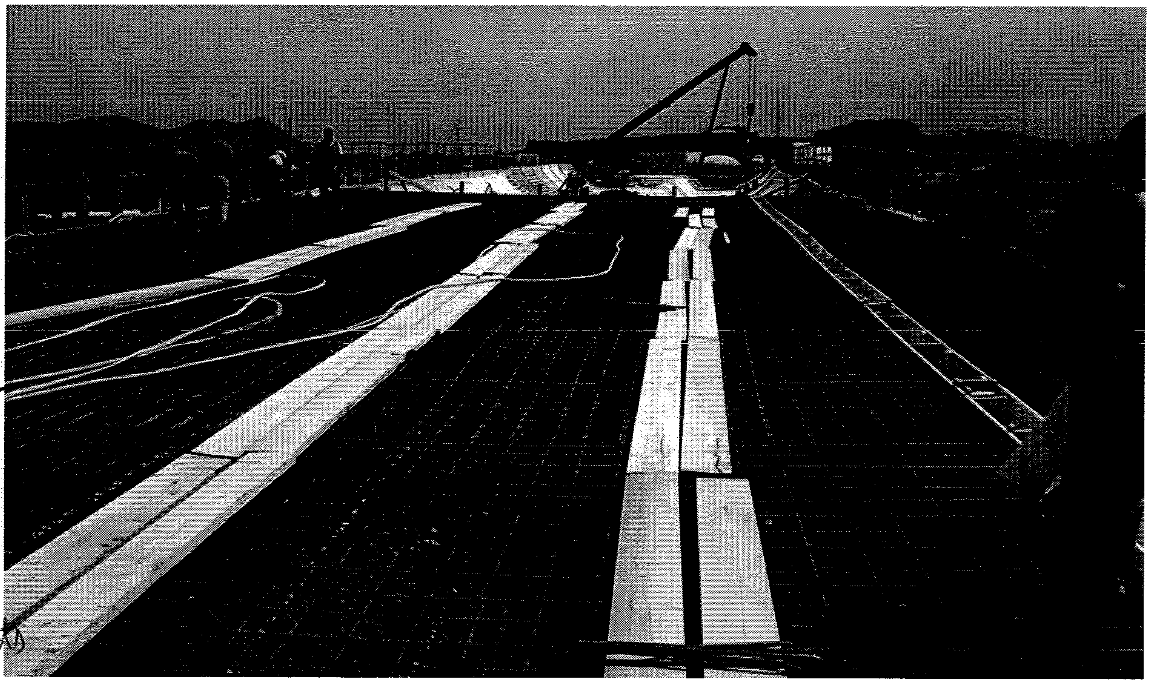
: concreting

: place reinforcements

: start to build scaffold...

...

here 3 scaffolding of 1 step (this technique is called "MOVING SCAFFOLDING")



Construction joint between casting i and i+1 before concreting

prestressing tendons

there are the couplers (to couple the prestressing tendons)

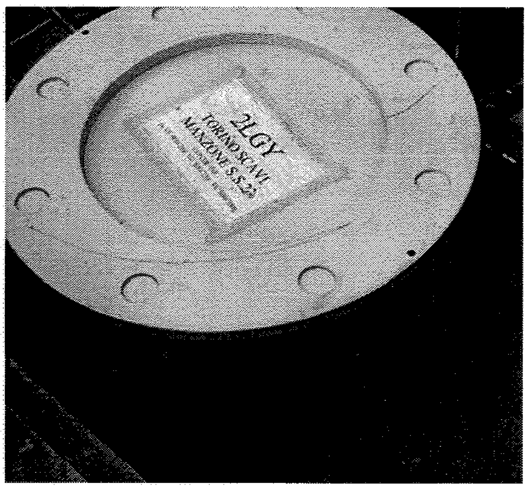
Cutting tendons strands after tensioning in the coupler

... At the end of that deck (33 m after).

couplers for the tendons for each span.

These tubes are used to continue the tendons.

Bearing detail



day 1 of the structural analysis

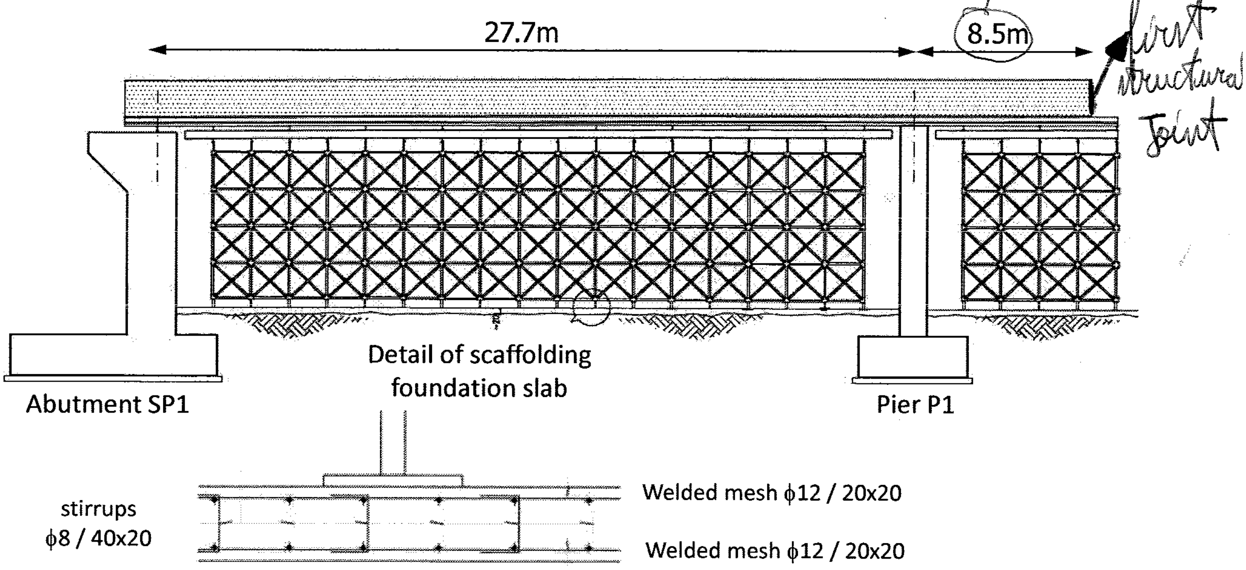
Day 1 for the structural analysis is not day 1 in the working site.

more or less 1/4 of the span

Phase 1 & 2:

Day 1 - Concreting of span SP1 – P1

Day 14 – tensioning of tendons "I" and removal of scaffolding

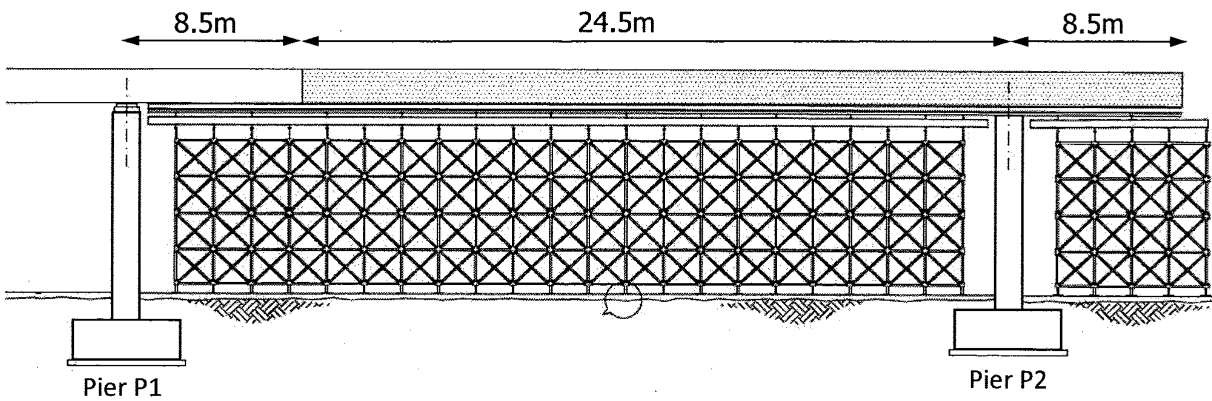


Phase 3 is the removal of scaffolding

Phase 4:

Day 45 - Concreting of span P1 – P2

Day 58 - tensioning of tendons "I" and removal of scaffolding



Table

Fase n°	Tempo [gg]	Evoluzione struttura	Cavi tesati
22	190	Scaffolding removal P4-P5	I5
23	195	-	-
24	200	-	-
25	210	-	-
26	220	-	-
27	233.99	-	-
28	234	Scaffolding removal P5-SP2	I6
29	239	End of construction tendons	F1 & F2
30	244	Phases to reach the end of service life 25500 days = 70 years Pay attention to the dimension of the steps: short at the beginning, long at the end	
31	250		
32	260		
33	270		
34	290		
35	310		
36	330		
37	370		
...	...		
48	2410		
49	3410		
50	4410		
51	9410		
52	18820		
53	25550		

Each operation means an evolution, changing of the static scheme ⇒ ⇒ changing in the creep effects (according to 1st principle of linear viscoelasticity).

bigger step than first steps: steps used to calculate the creep.

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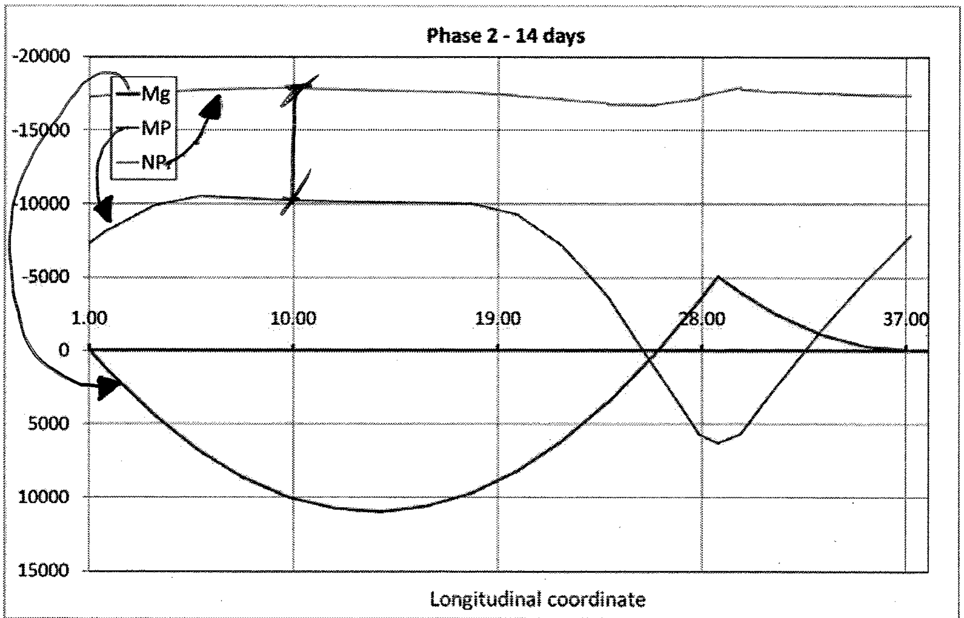
Phase 1 & 2:

span pier

Day 1 - Concreting of span SP1 – P1

Day 14 – tensioning of tendons "I" and removal of scaffolding

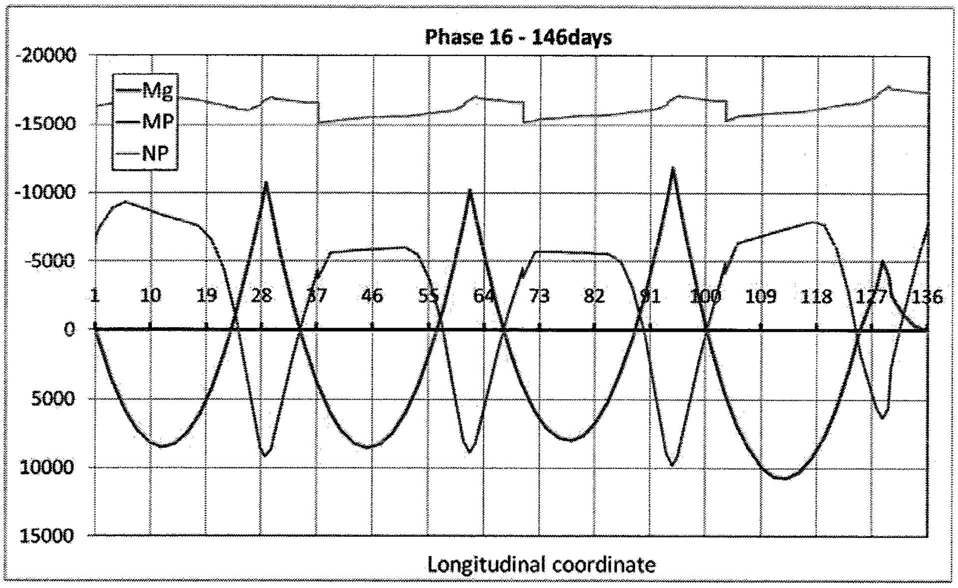


The eccentricity of the tendon is about 0,5 m, because Mp is about 1,5 times Np

Phase 16:

Day 130 - Concreting of span P3 – P4

Day 146 - tensioning of tendons "I" and removal of scaffolding

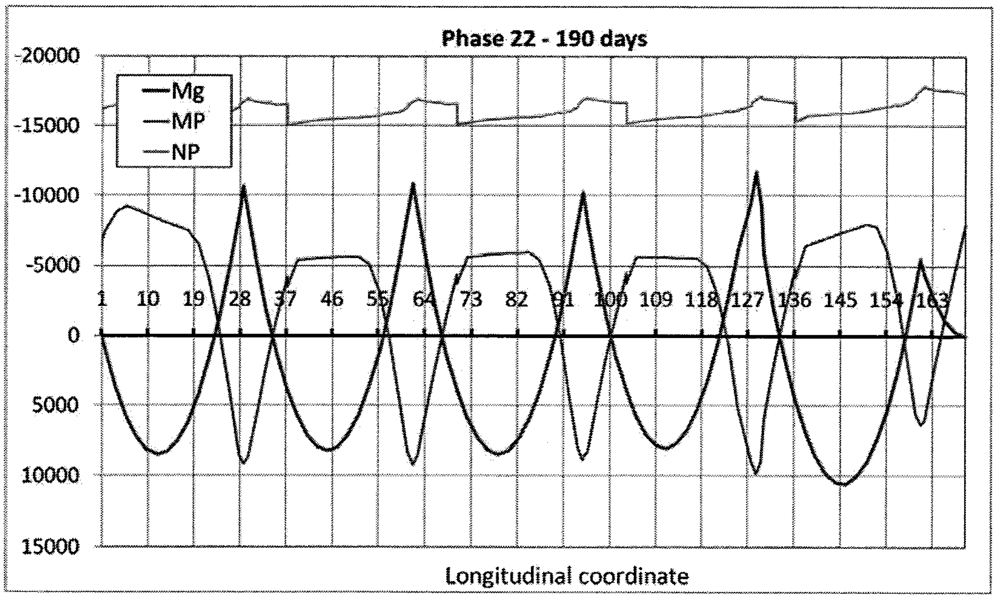


Phase 22:

pier 4 pier 5

Day 175 - Concreting of span P4 – P5

Day 190 - tensioning of tendons "I" and removal of scaffolding



PINEROLO BRIDGE

Prestressing layout

Internal bonded tendons

19 strands Ø 0.6" tendons A_t = 26.41 cm².

Prestressing stress:
$\sigma_{spi} = 0.85 \times f_{p(1)k} = 0.85 \times 1670 = 1420$ MPa

Maximum prestressing force in each tendon:
$T_{max} = 1419.5$ N/ mm² · 26.41 mm² = 3749 kN

Construction phases prestressing

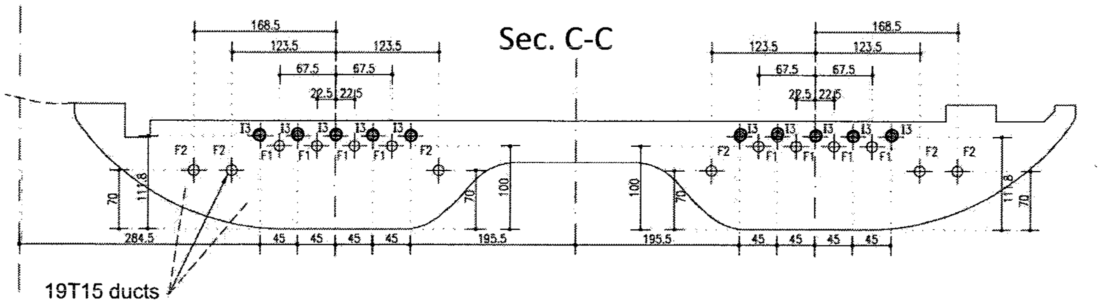
10 Tendons I1 coupled in each construction joint

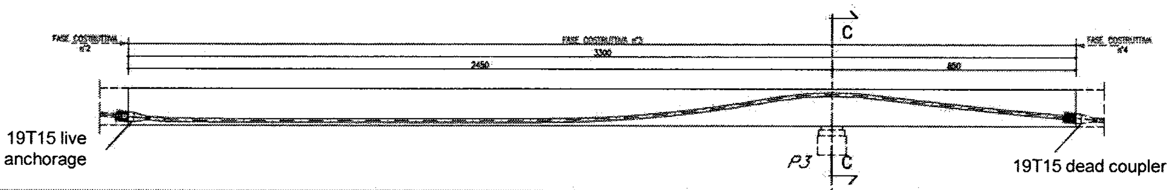
End of construction prestressing

8 tendons F1 for positive and negative moments +
\+ 6 baricentric tendons F2

placed to give only N, not M

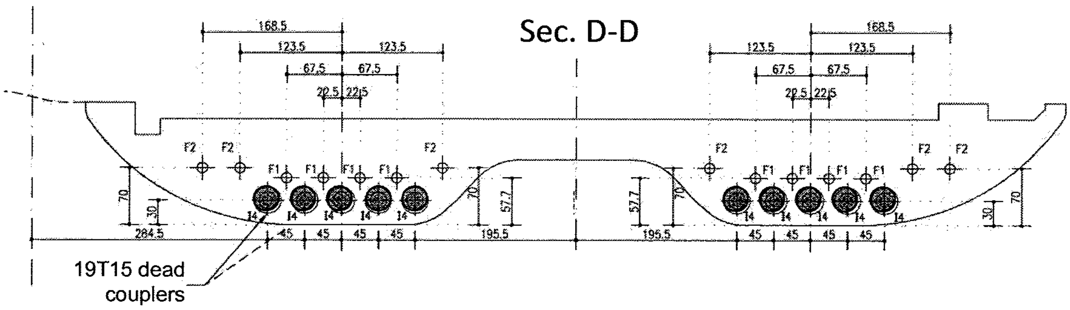
Phase 3: tendons I3

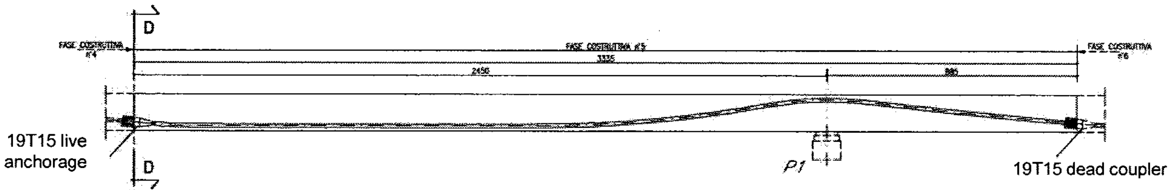




I3	X (cm) horizontal		50	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	50	The vertical Position Z of the tendon is taken from intrados
	Z (cm) vertical	30.0	25.6	18.7	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.1	19.9	23.8	30.1	38.4	49.2	61.9	77.1	94.1	107.1	111.8	107.4	96.4	84.5	73.2	62.6	52.6	43.2	34.3	30.0	

Phase 4: tendons I4





I4	X (cm) horizontal		50	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	85	The vertical Position Z of the tendon is taken from intrados
	Z (cm) vertical	30.0	25.6	18.7	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.1	19.9	23.9	30.1	38.5	49.1	62.0	77.1	94.1	107.2	111.6	107.9	97.8	86.6	75.8	65.5	55.7	46.3	37.4	30.0	

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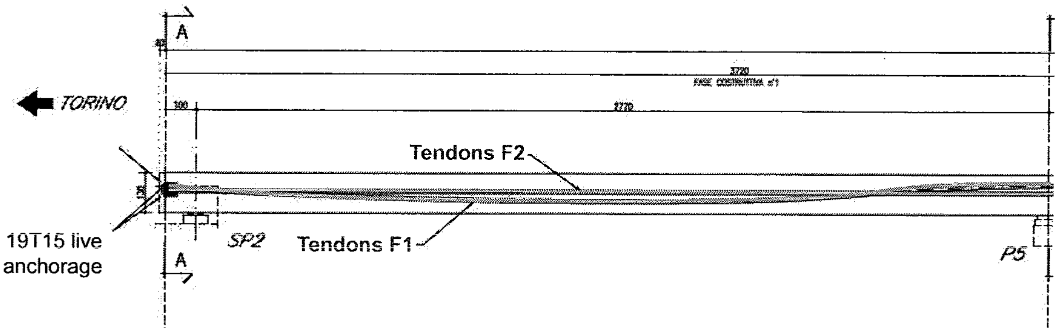
End of construction: tendons F1 & F2

Span SP2 – P5



final F1 tendons

F1	X (cm) horizontal		70	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
	Z (cm) vertical	80.0	75.3	71.2	66.5	62.0	58.1	54.3	51.2	48.2	45.9	43.8	42.3	41.1	40.4	40.0	40.0	40.0	40.0	41.0	42.6	46.5	51.1	58.4	66.1	74.2	82.0	87.0	91.5	93.5	94.7

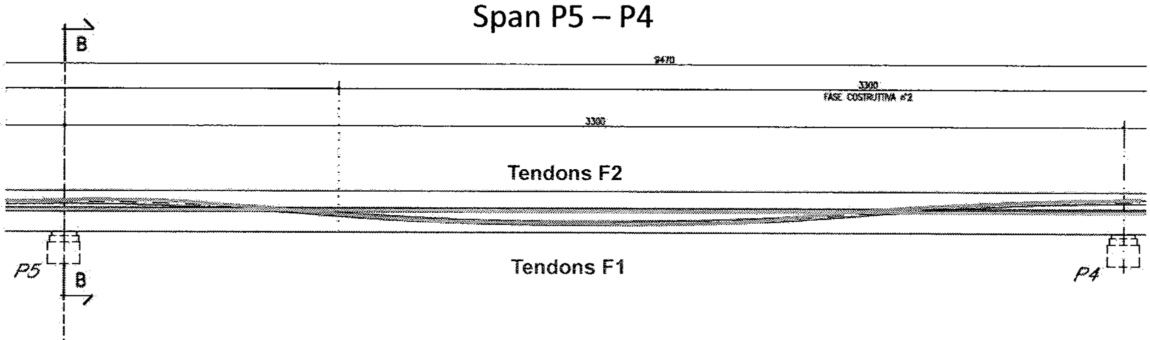
final F2 tendons

F2	X (cm) horizontal		70	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
	Z (cm) vertical	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0

See the colour of the slides!!!

End of construction: tendons F1 & F2

Span P5 – P4



100	100	100	100	100	100	100	100	100	100	50	50	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	10
93.5	94.7	94.0	92.4	89.6	85.7	80.7	74.5	67.5	59.9	56.0	52.1	46.0	40.3	36.5	32.9	31.4	30.0	30.0	30.0	30.3	31.3	33.6	36.9	41.4	47.2	54.0	62.0	70.6	77.9	84.9	89.8	94.5	97.0	99.3	99.4	[illegible]

100	100	100	100	100	100	100	100	100	100	50	50	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	10
70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	70.0	[illegible]

LEZ. 02-12-2013

VEROLENGO BRIDGE

Precast continuous beam

Very very cheap bridge.

VEROLENGO BRIDGE

Geographical positioning

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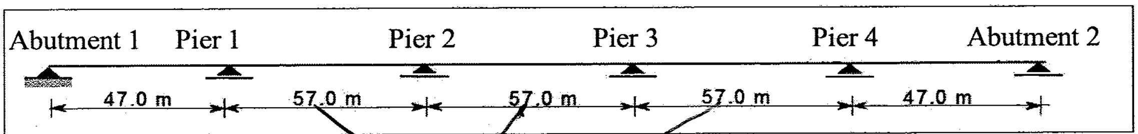
VEROLENGO BRIDGE

General overview

Static scheme: scheme of a continuos beam

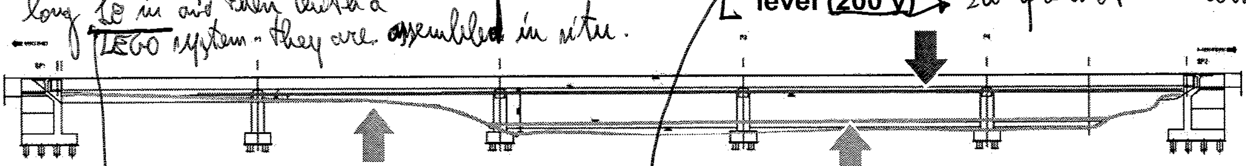
General dimensions



57 m: is too long to use prefabricated beams, but we used prefabricated beams anyway: they are long 10 m and then with a LEGO system - they are assembled in situ.

span unusual for concrete bridges (general is for steel or composite bridges).

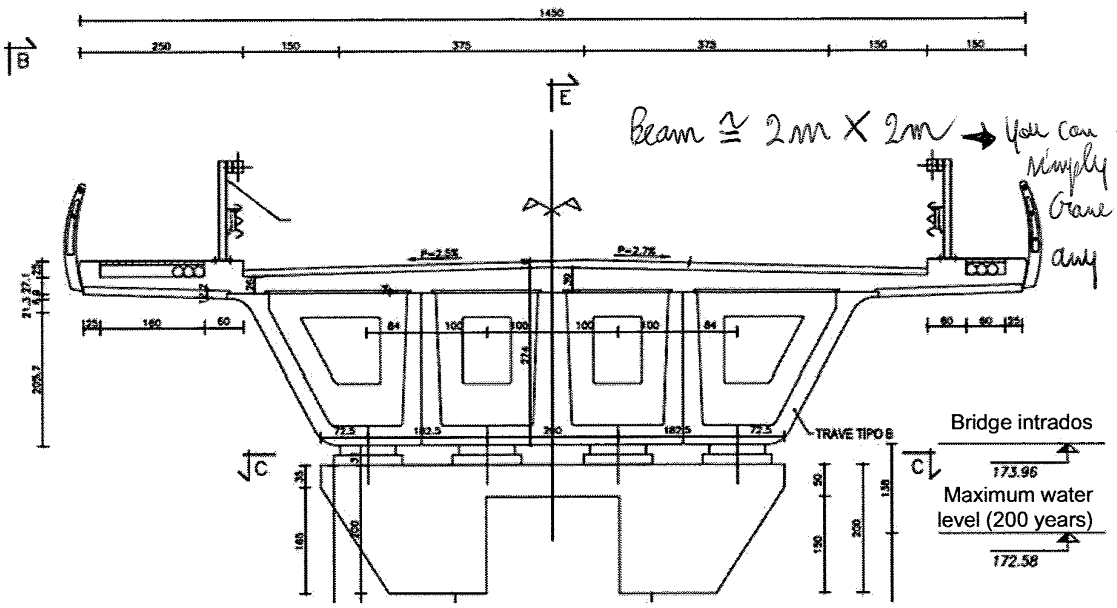
Maximum water level (200 y) → 200 years of return period



so you need 3 beams for each span.

almost at the intrados

Deck and upper part of the pier

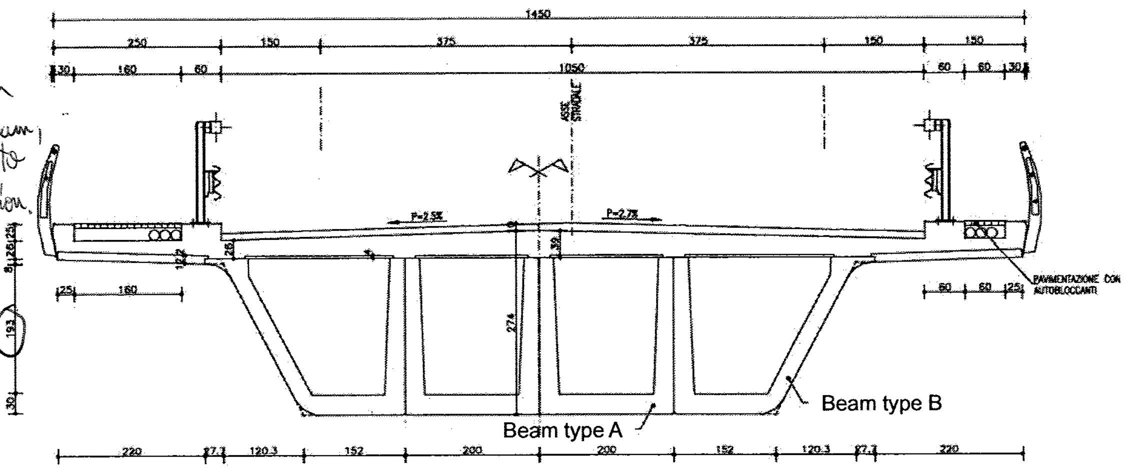


Beam ≅ 2m X 2m → You can move simply with a crane without any problem.

Deck cross section

This is the minimum dimension of box shaped beam, because you have to provide the inspection.

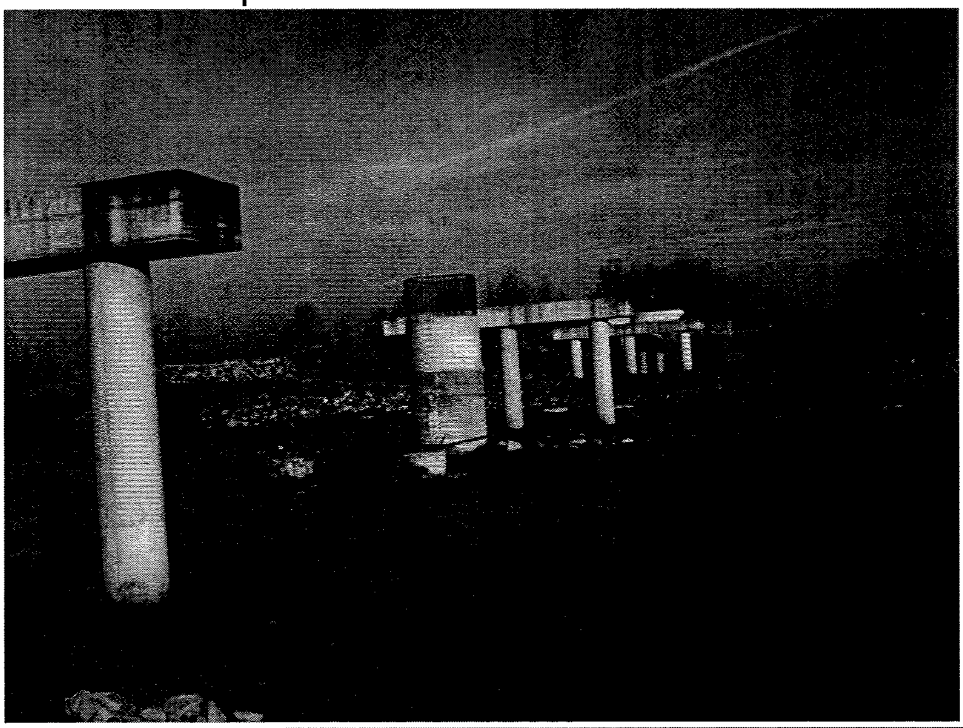
1,93m ⇒ a man can walk inside



Pier n°1 and old pier

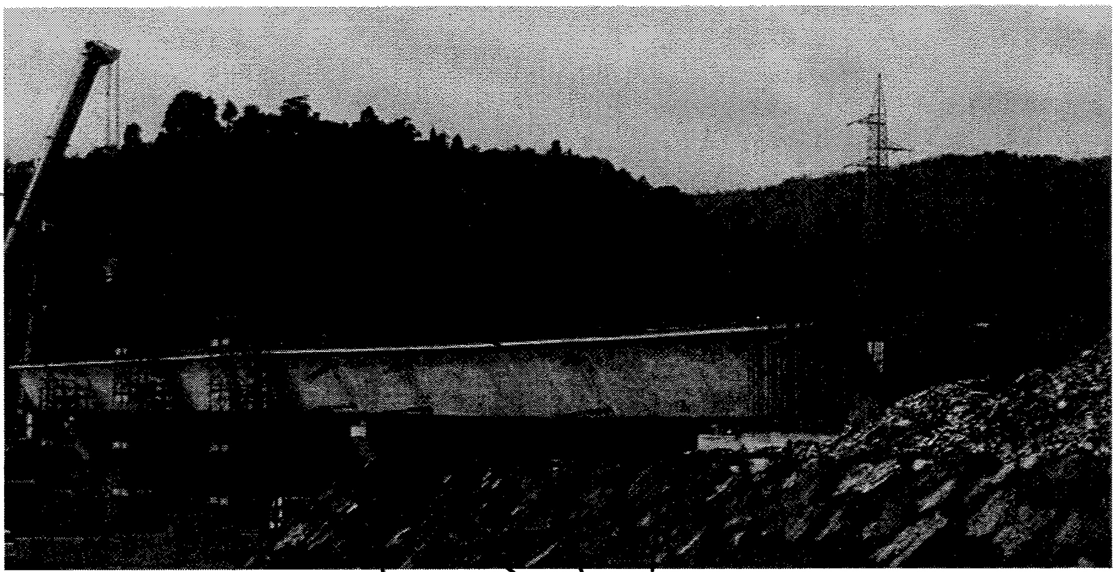
piers of the old temporary ~~pier~~ bridge (wooden bridge). These piers were used as temporary piers for the construction of the new bridge

Old and new piers within the river bed



Inflection of 1st span after the removal of its 2 temporary piers

We can see the deformed shape of the beam under the dead weight.



final pier.

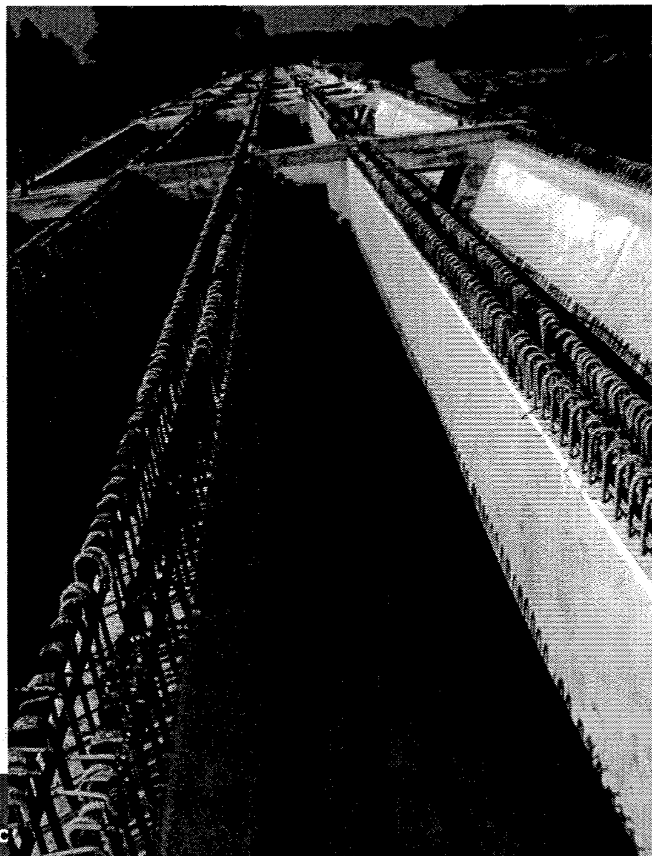
3 segments

External tendons within the beams

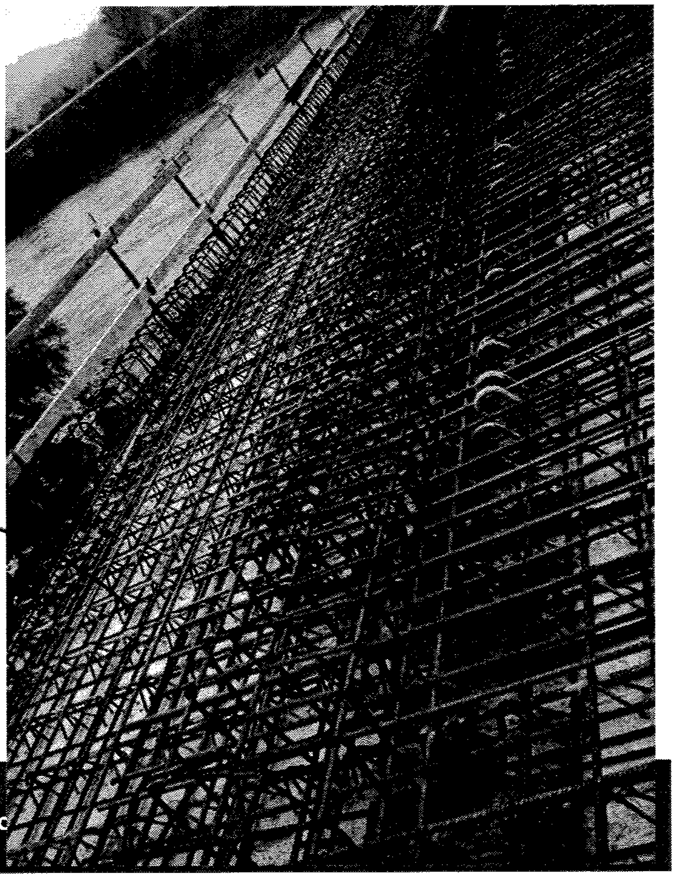
Phase.

In this bridge we have all kind of prestressing:

- pretensioned strands
-
-



Reinforcement of deck slab before concreting



predalles used as a scaffolding.

The cost per m^2 of the deck was 600 € (without the cost of the piers and foundations)

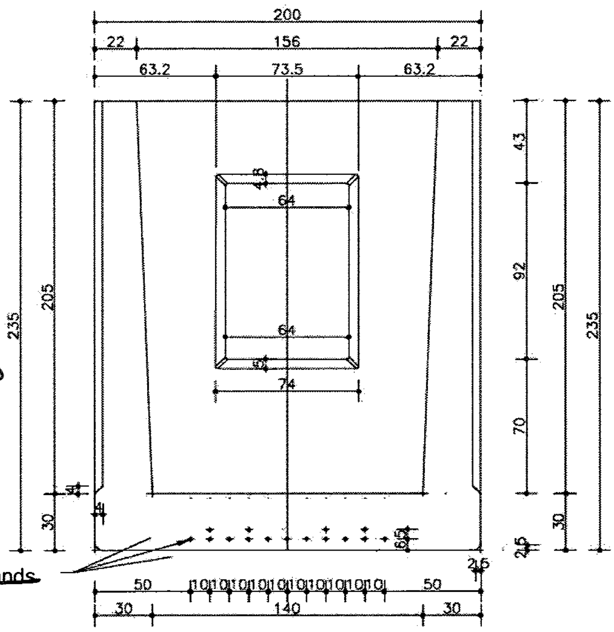
Completed bridge

Generally normal concrete bridges costs 1000/1200/m^2. With bigger spans → 2000/m^2. For stay cable bridge: 5000/m^2

Beam A → typical central beam

Cross section

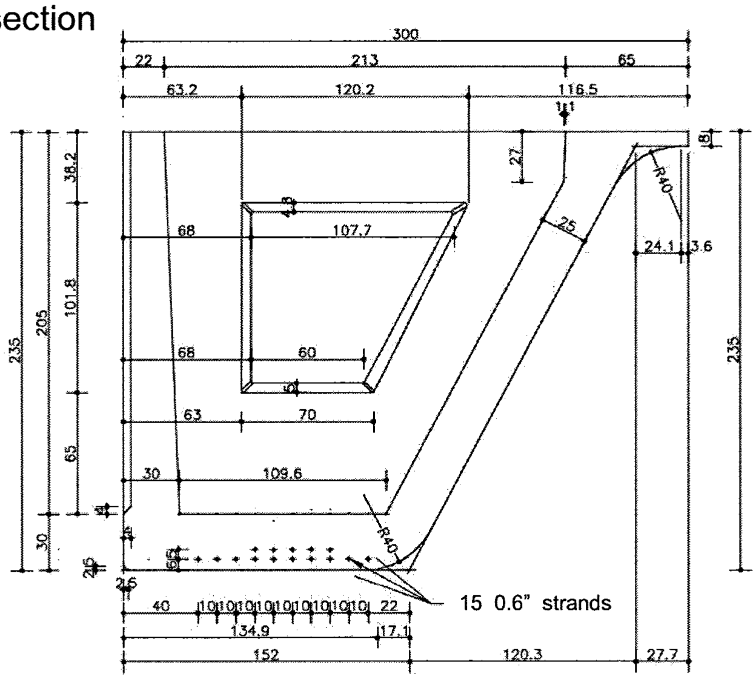
pretensed strands



they are present since the factory (la trave è pretesa, come tutte le travi dei capannoni).

Beam B

Cross section



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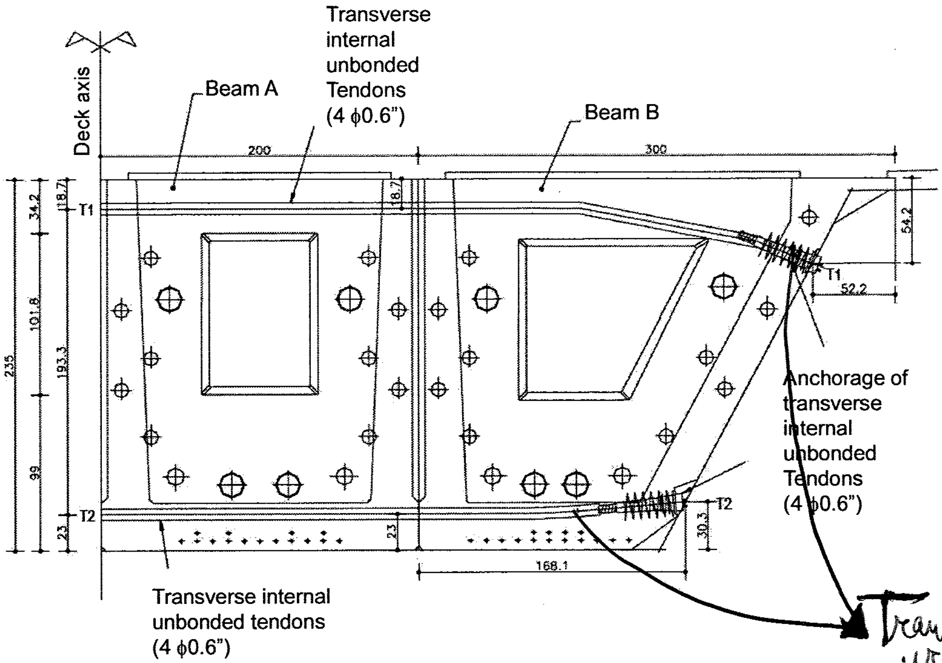
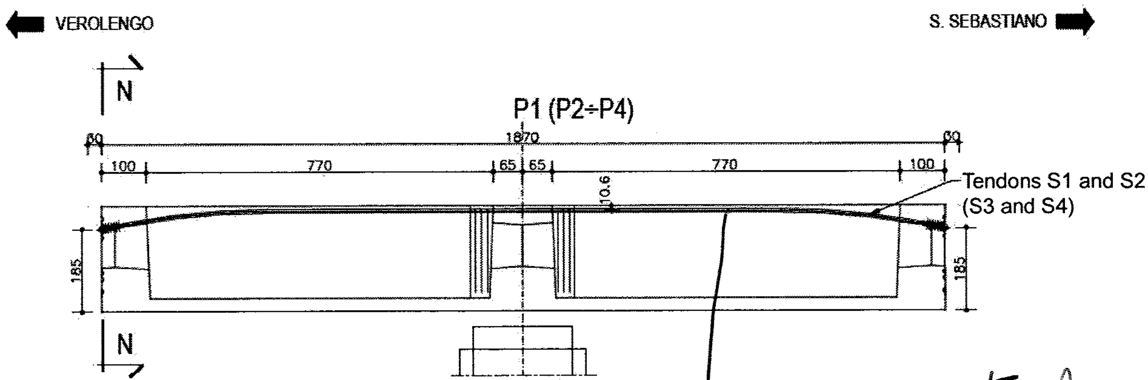
“Bridge design”

Typical transverse beam section



Transverse prestressing

Internal bonded prestressing of pier elements



This tendon is present only on the segment (beam) that is on the pier.

Cross section properties

	Area 1	Area 2	J3	J2	yg
	[m2]	[m2]	[m4]	[m4]	[m]
Beam A	1.66	1.66	0.86	1.01	0.87
Beam B	1.63	1.63	0.91	1.43	1.02
Beam A + its slab	2.31	2.25	2.09	1.21	1.31
Beam B + its slab	2.80	2.69	2.39	4.65	1.60
all Beams alone	6.58	6.58	3.54	39.20	0.93
Slab	3.63	3.28	0.02	45.55	2.53
all Beams + Slab	10.21	9.86	9.16	84.75	1.46

Area 1 pure geometric area (to calculate self weight)

Area 2 homogenized area (on the cls of the precast beams) → cross section of the slab is reduced.

J3 main longitudinal bending inertia

J2 transverse bending inertia

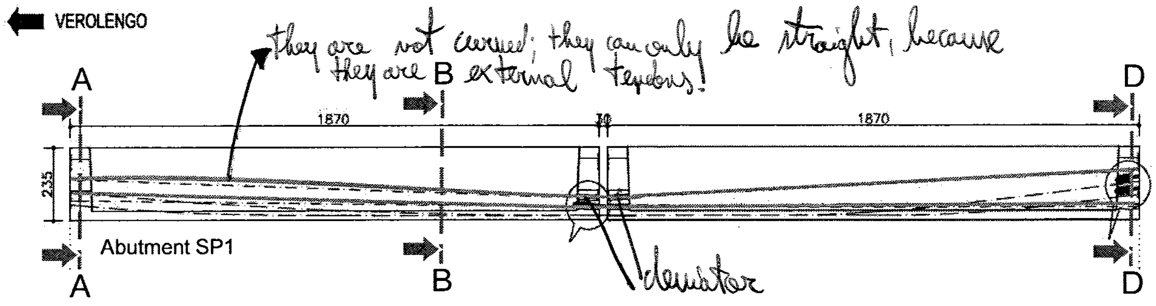
Yg centroid distance from deck intrados

RANTIVA BRIDGE

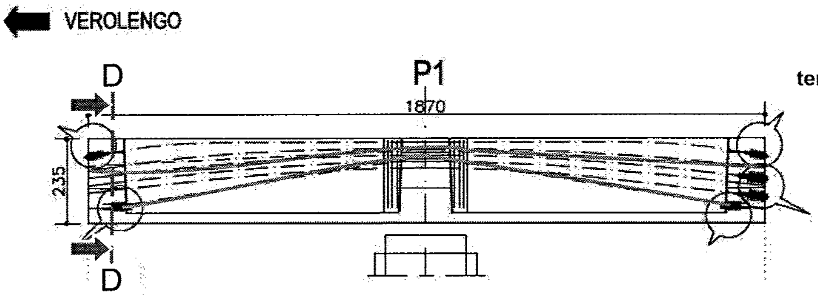
Prestressing &

Ordinary reinforcement

Prestressing of beam A: segments 1 and 2 – END of construction



Prestressing of beam A: segment 3 – END of construction



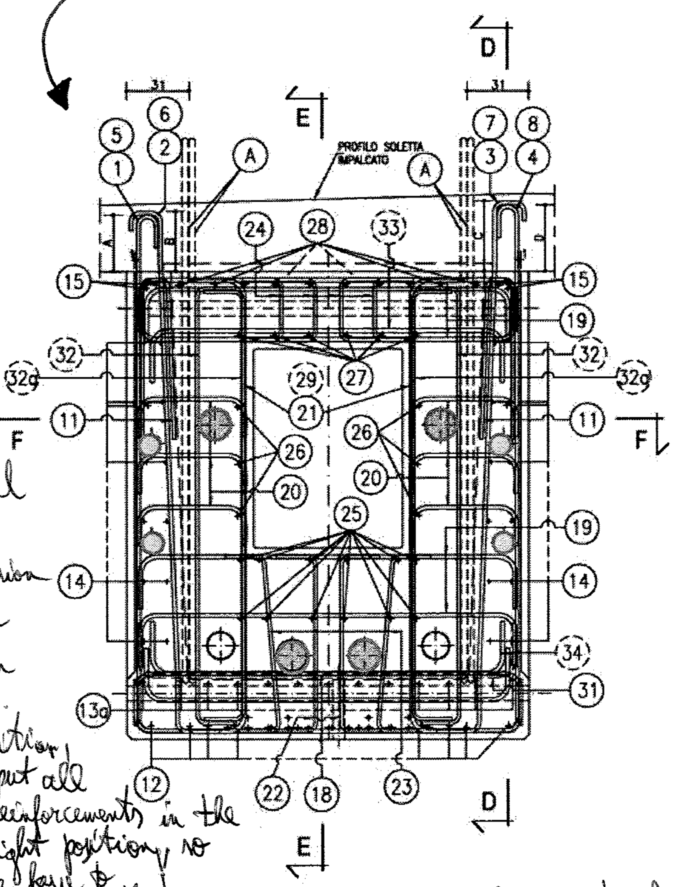
External tendons
F1-1 to F1-8 (9 f0.6")
4 for each beam
tensed at the end of construction

With the same colour seen before we have

Beam A
First segment
Cross section AA (abutment)

Internal tendons F

External tendons tensed at the end of construction



This is the top level of reinforcement layer (every dimension is the right one: we have the definition of all cm and mm).

This image is with no high definition, because it's really complicated to put all reinforcements in the right position, so we have to define everything with [illegible] of the external tendons)

Beam A
Second segment
Cross section CC:

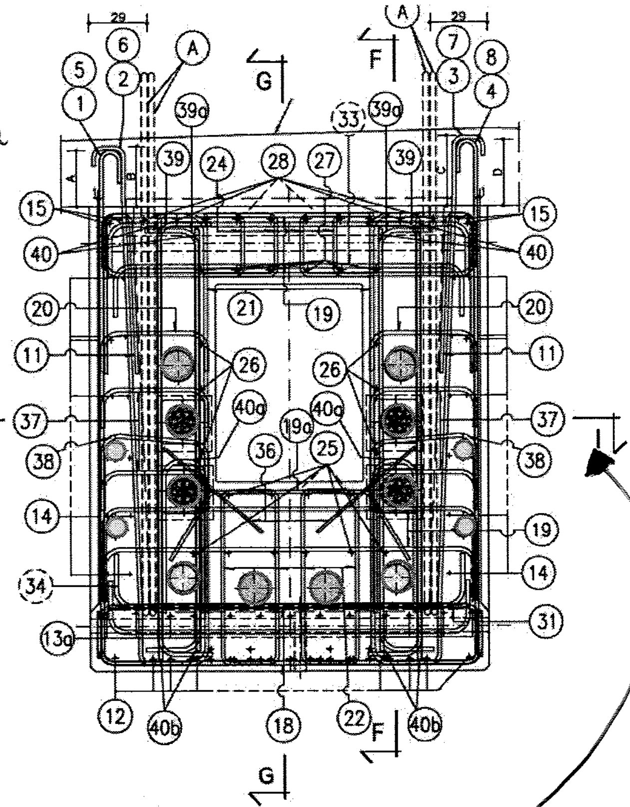
there is all the prestressing

Internal tendons F1-1 to F1-8 (1st span)

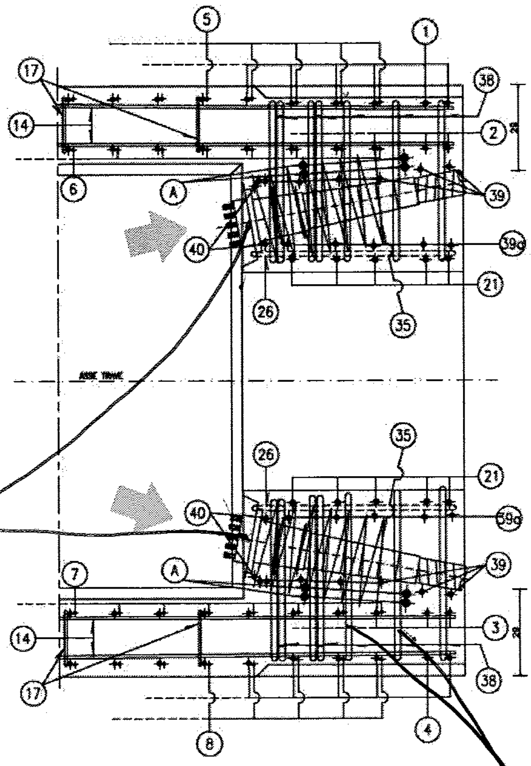
External tendons tensed at the end of construction

External tendons tensed during construction : straight external unbounded

Internal tendons F2-1 to F2-8 (2nd span)



Beam A
Second segment
Horizontal section II



Anchorages of internal tendons F2-1 to F2-8 (2nd span)

orange tendons

long shear reinforcement that tie together this part

2 different models:

1: for construction phase
2: for end of construction verification (traffic load)

1 Construction phases and creep effects:
one dimension model
the deck is a single continuous straight beam
deadweight does NOT give rise to torsion

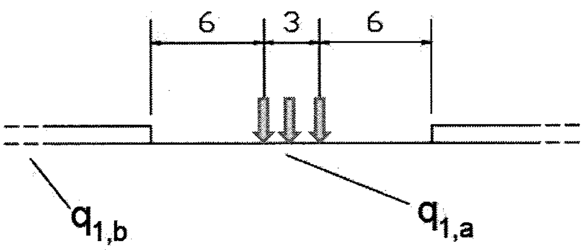
2 SLS and ULS with live load:
girder model:
the deck is a girder made of longitudinal and transverse beams and slab;
live load gives rise to torsion

In the 1st model we have symmetric condition (self weight, ...). At the end of construction we have big eccentricity of the traffic loads → we need 3D Model to take into account the torque moment.

Live loads on model 2:

3 notional lanes in transverse direction
Each lane is 3m wide in transverse direction

LM1 according to old Italian code



$q_{1,a}$ = 3 couples of concentrated loads 150kN each = 3 x 2 x 150 = 600 kN

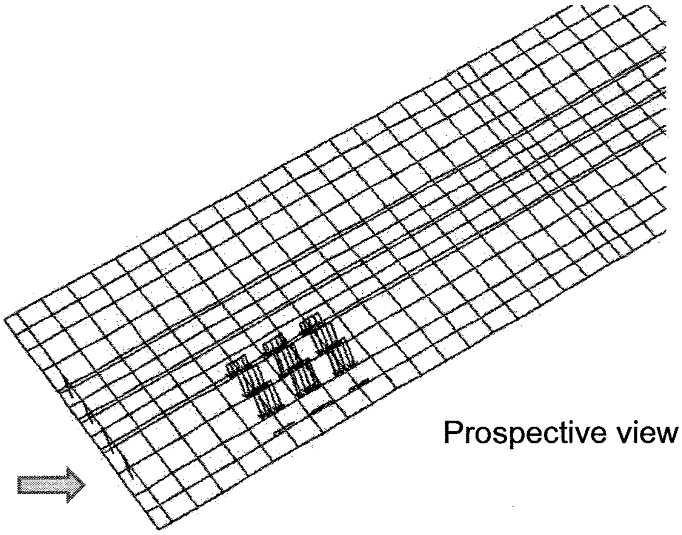
$q_{1,b}$ = uniformly distributed load of 3KN/m^2

The results given by model 1 for deadweight, prestressing, creep, shrinkage, construction phases are then divided between the 8 beams corresponding to the webs of the boxes

Longitudinal view

Position 3

$q_{1,a}$ Loads corresponding to the outermost notional lane

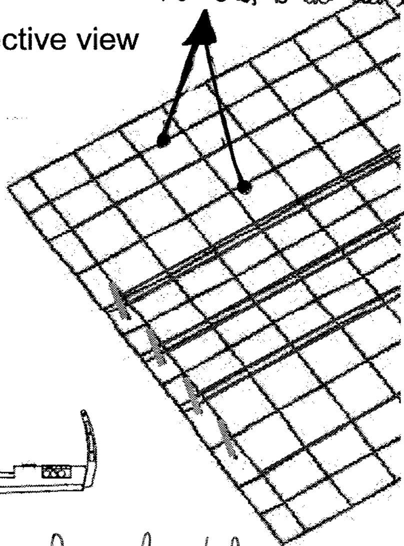


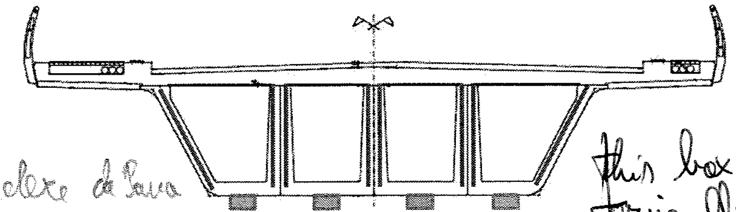
Prospective view

these beams only represent the slabs. They only transfer the loads to the red beams.

The webs are highlighted in red whereas the offset elements for the bearings are evidenced in green.

Prospective view





Rivedere da Sava

this box shaped beam is very rigid torsionally. (multi-cell deck)

9	69	Positioning of precast elements of span P2-P3and concreting of relative tranverse beams	Span 3
10	79	A. Removal of temporary piers of span P2-P3 B. Tensioning of tendons F3-1 to F3-8	
11	85	Tensioning of tendons E3-1 to E3-8	
12	86	Positioning of precast elements of span P3-P4and concreting of relative tranverse beams	Span 4
13	96	A. Removal of temporary piers of span P3-P4 B. Tensioning of tendons F4-1 to F4-8	
14	102	Tensioning of tendons E4-1 to E4-8	
15	103	Positioning of precast elements of span P4-SP2 and concreting of relative tranverse beams	Span 5
16	113	A. Removal of temporary piers of span P4-SP2 B. Tensioning of tendons F5-1 to F5-8	
17	122	Tensioning of tendons E5-1 to E5-8	
18	123	Slab concreting	
19	150	A. End of slab hardening - change of centroid positions B. Tensioning of tendons EF1 to EF8	

the centroid is moving up (also the moment of inertia changed).

So: [illegible] is effecting also the slab. The green and yellow no.

20	163	Phases to reach the end of service life 25500 days = 70 years Pay attention to the dimension of the steps: short at the beginning, long at the end	
21	183		
22	223		
23	263		
24	313		
25	363		
26	450		
27	550		
28	650		
29	850		
30	1050		
31	1250		
32	1750		
33	2250		
34	3250		
35	4250		
36	9000		
37	18000		
38	25500		

Phase 2:

Beam A – Pier element
Stress verification during storage lifting and transport
Internal actions

Section	x (long) [m]	Self weight			Prestress - strands			Prestress - tendons		
		N [kN]	M [kN m]	V [kN]	N [kN]	M [kN m]	V [kN]	N [kN]	M [kN m]	V [kN]
1	0	0.0	0.0	0.0	0.0	0.0	0.0	-2449.0	2288.0	476.0
2	0.35	0.0	-5.2	-418.1	-249.0	-196.4	0.0	-2450.6	2456.2	476.4
3	0.7	0.0	198.6	-397.3	-689.9	-544.4	0.0	-2453.2	2625.7	474.3
4	1.35	0.0	335.1	-382.7	-1508.8	-1190.6	0.0	-2494.4	2944.1	371.7
5	2.35	0.0	697.0	-341.1	-2768.7	-2184.7	0.0	-2541.5	3307.0	235.8
6	3.35	0.0	1017.2	-299.4	-2960.7	-2336.2	0.0	-2581.1	3525.4	94.3
7	4.35	0.0	1295.8	-257.8	-2960.7	-2336.2	0.0	-2604.0	3587.7	0.0
8	5.35	0.0	1532.7	-216.1	-2960.7	-2336.2	0.0	-2608.5	3593.9	0.0
9	6.35	0.0	1728.0	-174.5	-2960.7	-2336.2	0.0	-2613.0	3600.2	0.0
10	7.35	0.0	1881.6	-132.8	-2960.7	-2336.2	0.0	-2617.5	3606.4	0.0
11	8.35	0.0	1993.6	-91.2	-2960.7	-2336.2	0.0	-2621.9	3612.4	0.0
12	9.35	0.0	2063.9	49.5	-2960.7	-2336.2	0.0	-2617.3	3606.0	0.0

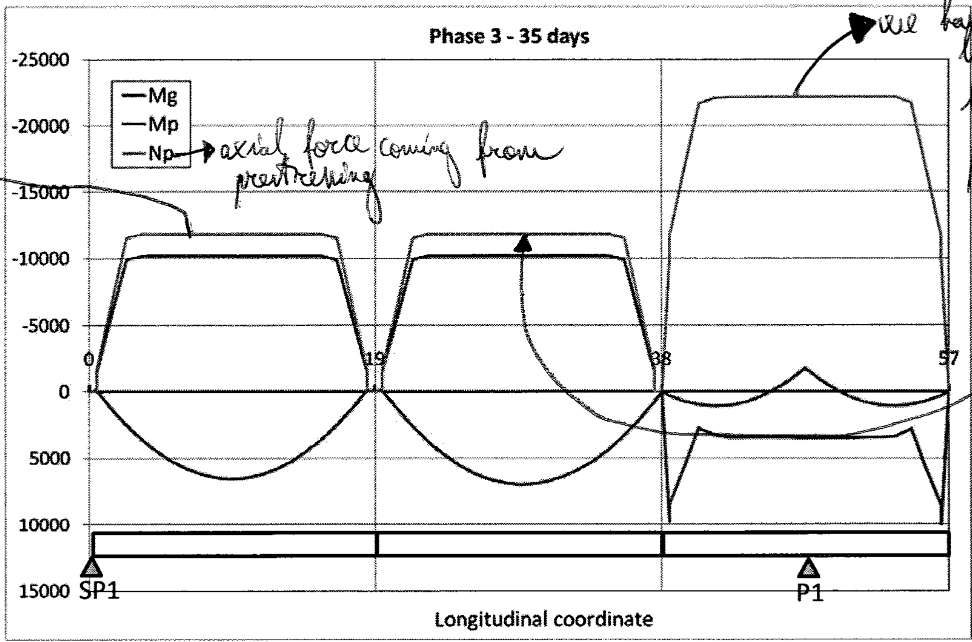
Phase 2:

Beam A – Pier element
Stress verification during storage lifting and transport
Only pre-tensioned strands present

Section	x (long) [m]	Self weight + strands			Stresses	
		N [kN]	M [kN m]	V [kN]	σ inf [MPa]	σ sup [MPa]
1	0	0.0	0.0	0.0	0.00	0.00
2	0.35	-249.0	-201.7	-418.1	-0.35	0.20
3	0.7	-689.9	-345.7	-397.3	-0.76	0.18
4	1.35	-1508.8	-855.4	-382.7	-1.77	0.57
5	2.35	-2768.7	-1487.7	-341.1	-3.17	0.90
6	3.35	-2960.7	-1319.0	-299.4	-3.11	0.50
7	4.35	-2960.7	-1040.4	-257.8	-2.83	0.02
8	5.35	-2960.7	-803.5	-216.1	-2.59	-0.39
9	6.35	-2960.7	-608.2	-174.5	-2.39	-0.73
10	7.35	-2960.7	-454.6	-132.8	-2.24	-0.99
11	8.35	-2960.7	-342.6	-91.2	-2.12	-1.19
12	9.35	-2960.7	-272.3	49.5	-2.05	-1.31

Phase 3: 35 days

Positioning of precast elements of span SP1-P1 and concreting of relative transverse beams



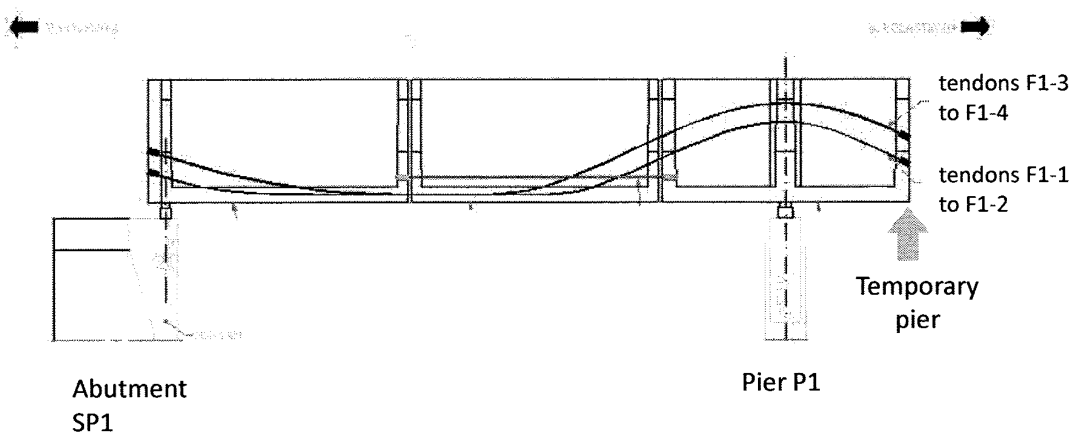
due to the prestressed tendons

axial force coming from prestressing

we have the strands below and the factory tendons above, so the Np is almost double of

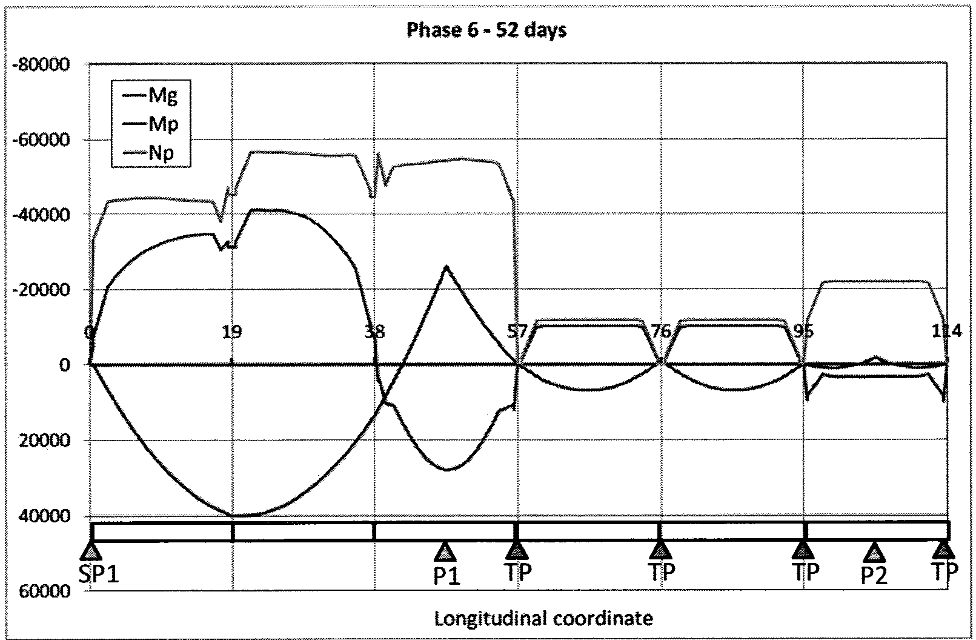
Phase 4:

A. Tensioning of tendons F1-1 to F1-8
B. Removal of temporary piers of span SP1-P1



Phase 6: 52 days

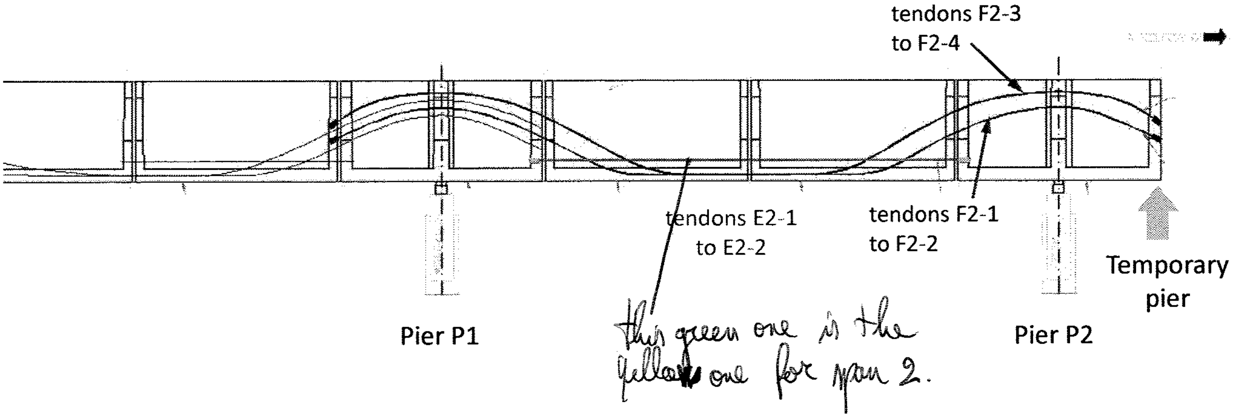
Positioning of precast elements of span P1-P2 and concreting of relative transverse beams



Phase 8:

A. Tensioning of tendons F2-1 to F2-8 P1-P2

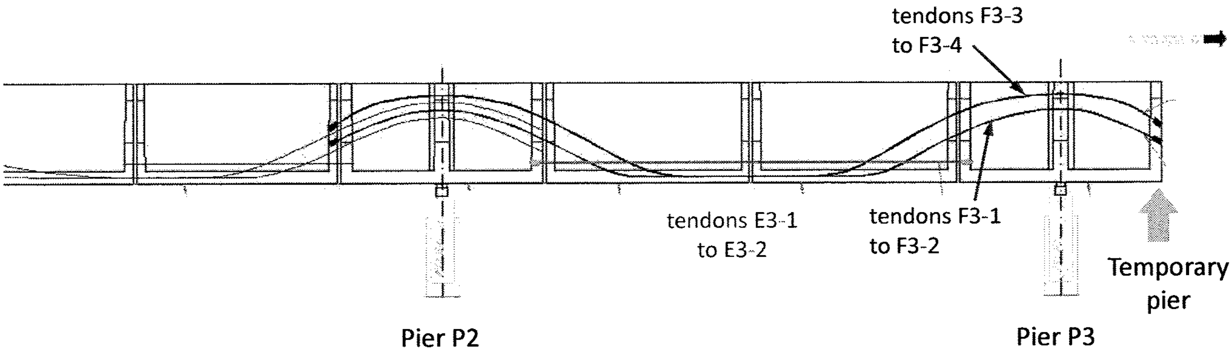
B. Removal of temporary piers of span 2 C. Tensioning tendons E2-1 to E2-8



Phase 11: 85 days

A. Tensioning of tendons F3-1 to F3-8 P2-P3

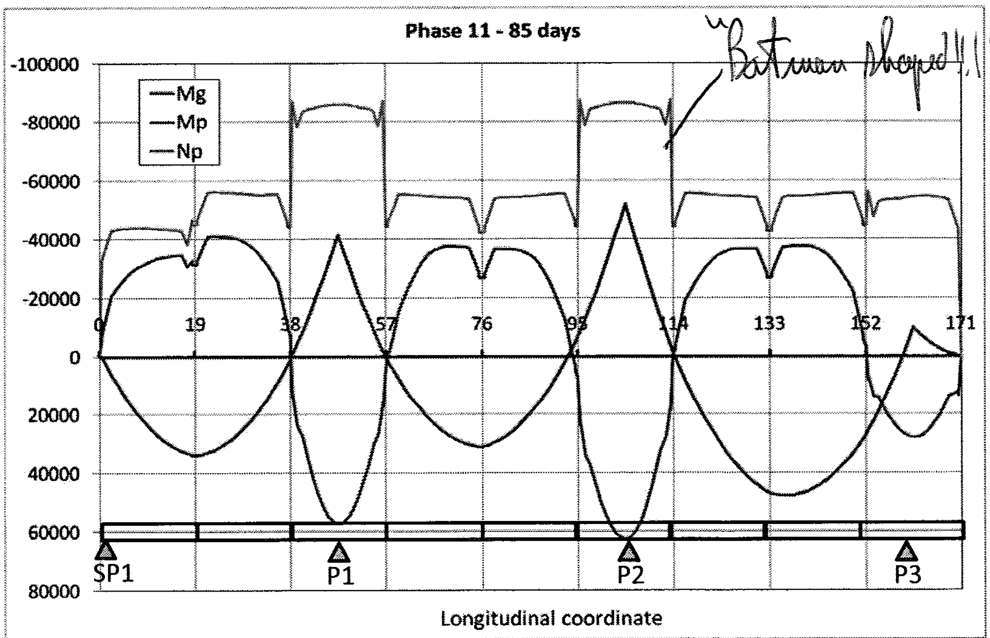
B. Removal of temporary piers of span 3 C. Tensioning tendons E3-1 to E3-8



Phase 11: 85 days

A. Tensioning of tendons F3-1 to F3-8 P2-P3

B. Removal of temporary piers of span 3 C. Tensioning tendons E3-1 to E3-8



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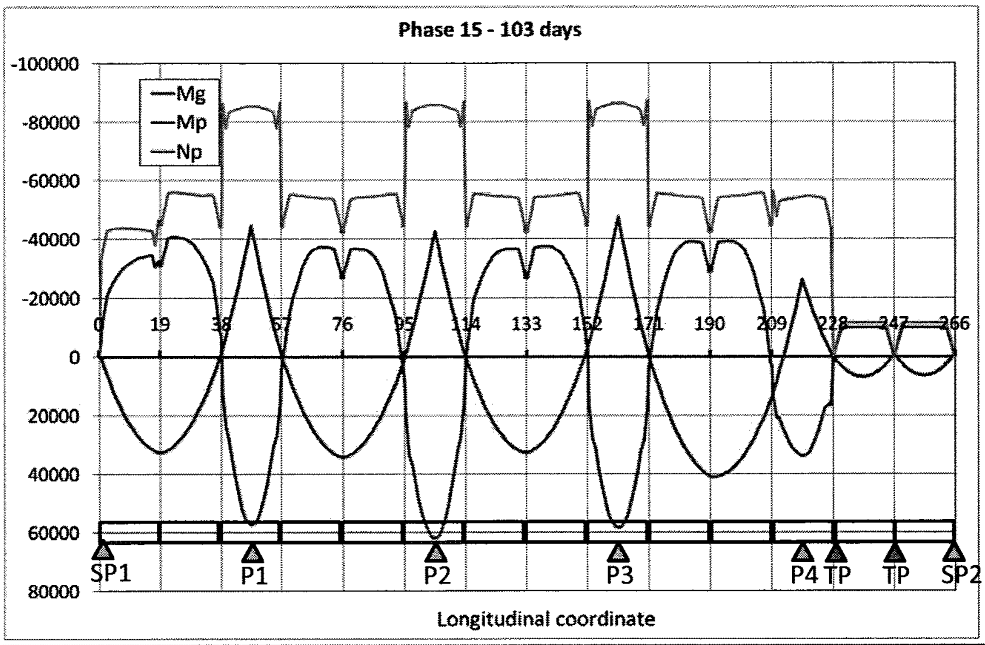
Phase 15: 103 days

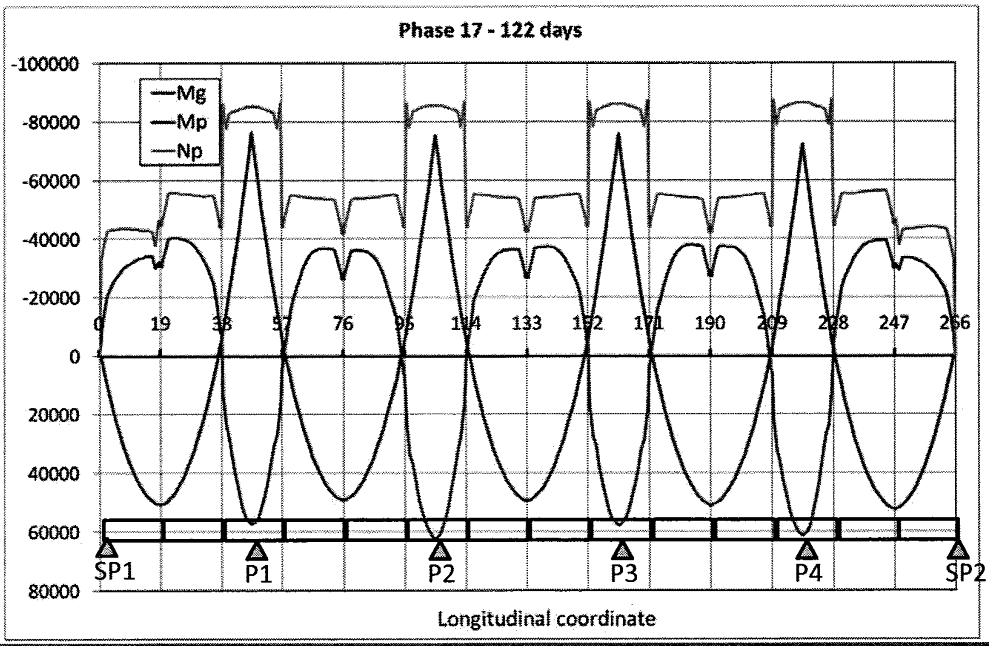
Positioning of precast elements of span P4-SP2and concreting of relative transverse beams



Phase 17: 122 days

A. Tensioning of tendons F5-1 to F5-8 P4-SP2

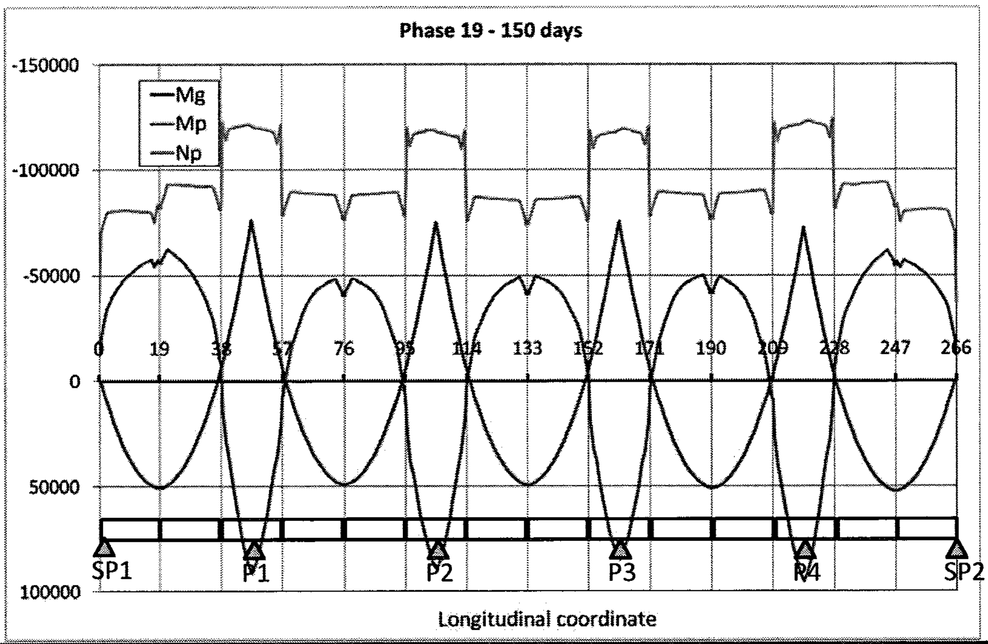
B. Removal of temporary piers of span 5 C. Tensioning tendons E5-1 to E5-8



Phase 19: 150 days

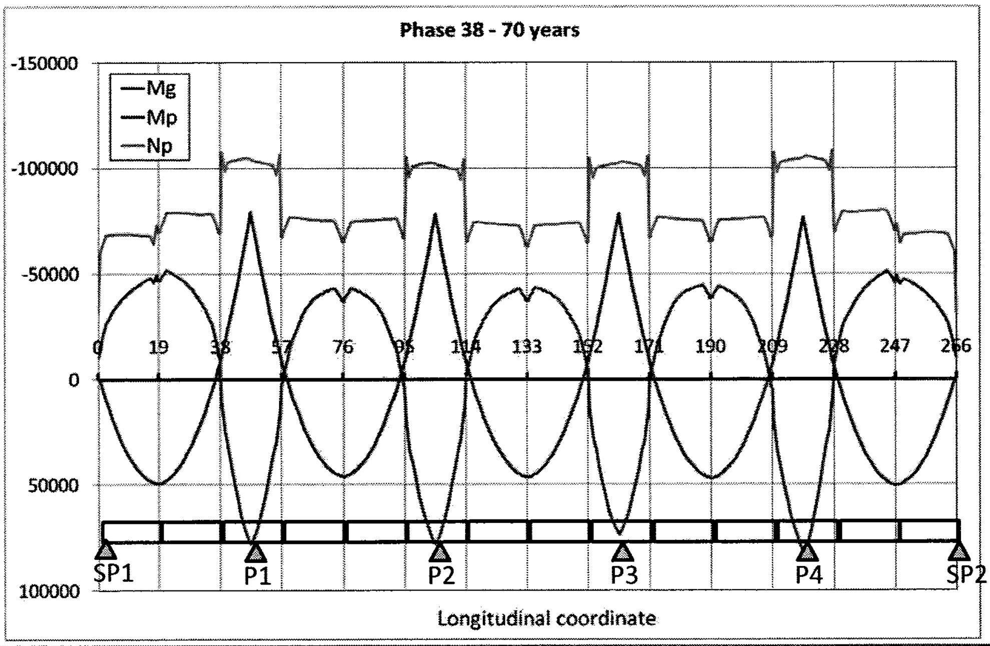
A. End of slab hardening - change of centroid positions

B. Tensioning of tendons EF1 to EF8



Phase 38: 70 years

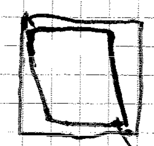
End of analysis for time dependant behaviour



References

- *CEB-FIP Model Code 1990*, Thomas Telford - 1990
- Comite Euro-International du Beton, *Bulletin d'Information n° 181 – "Anchorage zones of prestressing concrete members"*, 1987
- *Eurocode 2 Design of concrete structures, Part 1-1: general rules and rules for buildings* - 2003
- *Eurocode 2 Design of concrete structures – Part 2: concrete bridges* - 2004

In order to prevent the effect of thermal actions, shrinkage → the bridge is free to move longitudinally.



blister: enlargment of the section that allow you to put inside the prestressing devices.

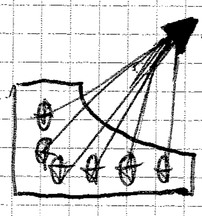
For each segment we have 3 kind of prestressing tendons.

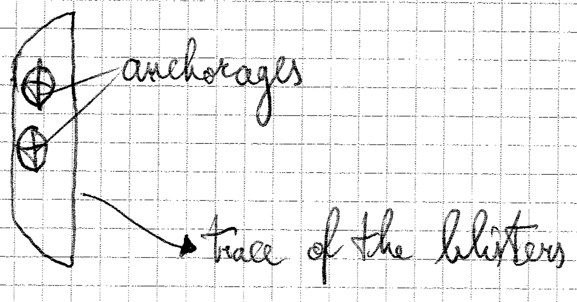
The dimension of the joint is more or less 30 cm, in another imagine is almost 1 m.

Let's see now the imagine of half of the box section.

Shear keys: prevent joint opening and allow the transmission of the shear between 2 adiacent segments.

holes for the prestressing for the positive bending moment





JOINTS AND BEARING DEVICES

For this bridge only a longitudinal direction is important → we'll use beam elements.
The effects in the transversal direction are negligible.

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Politecnico di Torino – Department of Structural and Geotechnical Engineering

"SEGNO" VIADUCT

Part 1- General Overview

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GENERAL OVERVIEW

PART 1

ABUTMENT SP3 – ABUTMENT SP2

ONE ROADWAY

12 SPANS
- 6 SPANS 46.00 m LONG
- 4 SPANS 43.60 m LONG
- 2 DIFFERENT SPANS NEAR THE ABUTMENTS

11 PIERS

PART 2

ABUTMENT SP2 – ABUTMENT SP1

BIFURCATION NEAR THE ABUTMENT SP2

DOUBLE ROADWAY BETWEEN SP2 AND SP1

2 SPANS WITH DIFFERENT SPAN LENGTH

1 PIER

TRANSVERSAL SECTION

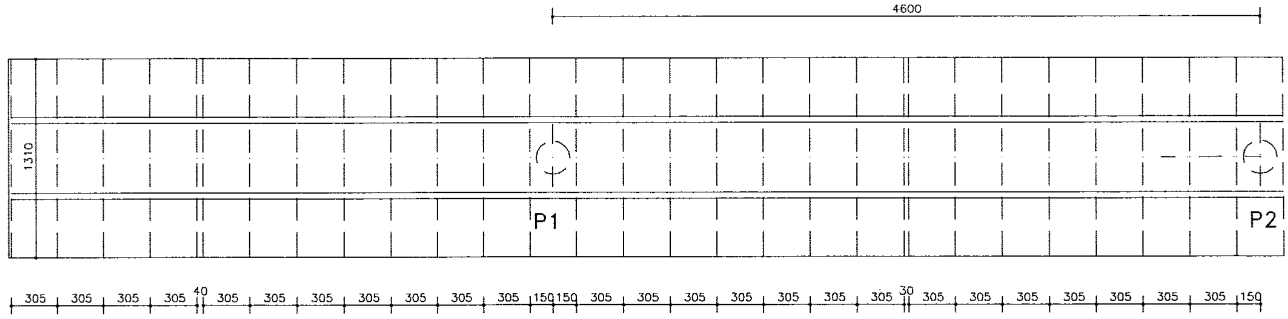
REINFORCED CONCRETE BOX SECTION WITH CONSTANT HEIGHT (2.24 m) AND TRANSVERSAL DIMENSION EQUAL TO 13.10 m

CONSTRUCTION PROCESS

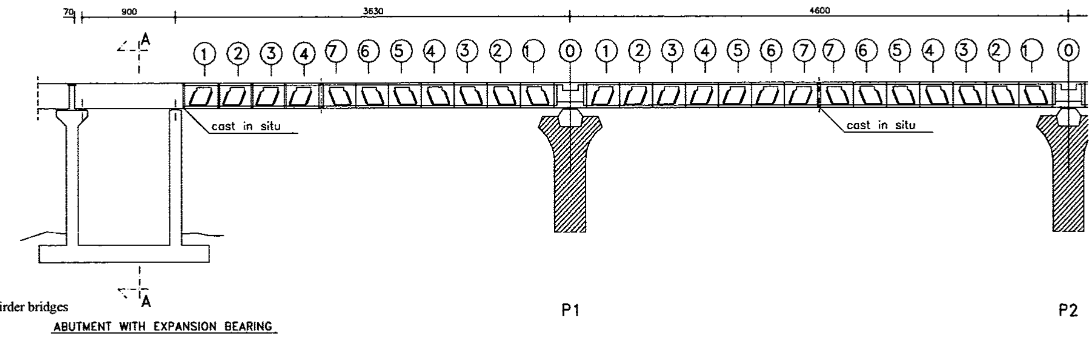
IN SITU ASSEMBLY OF SEGMENTAL PRECAST ELEMENTS WITH CLASSICAL BALANCED SYMMETRIC INCREMENTAL LAUNCHING CANTILEVER FROM THE PIERS.

CONSTRUCTION DONE WITH CRANE (NO LAUNCHING GIRDER) BECAUSE OF THE SMALL HEIGHT FROM THE GROUND.

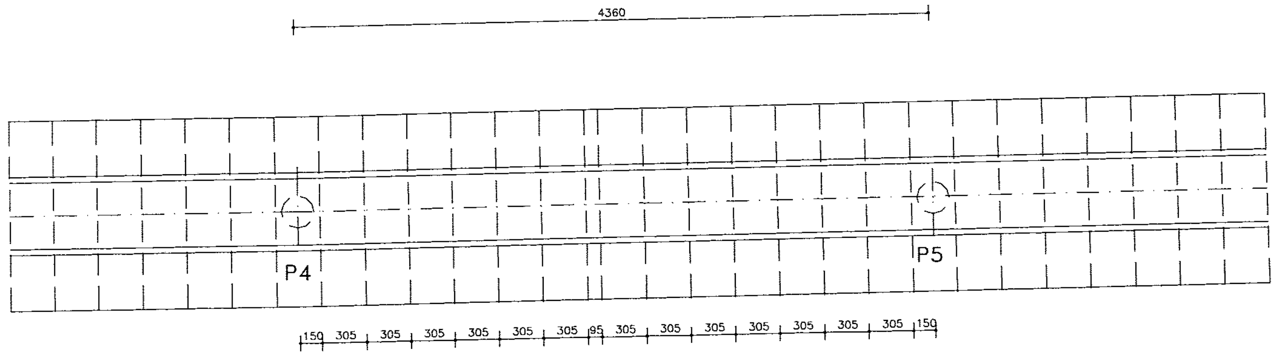
DECK ARRANGEMENT – 46.00 m SPAN WITH END SPAN FROM ABUTMENT SP1– TOP VIEW



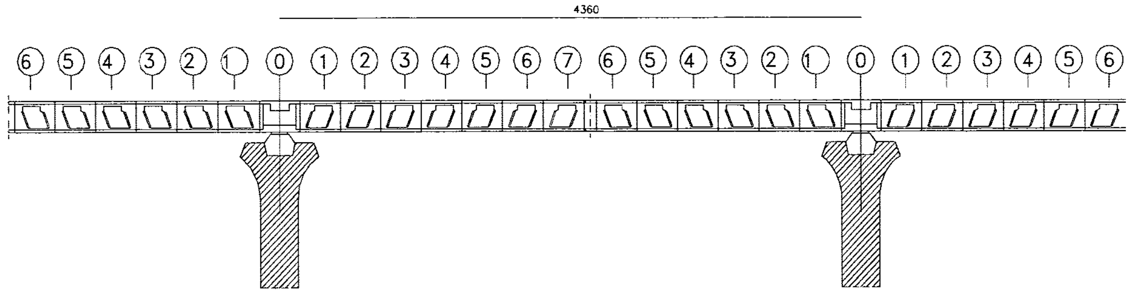
DECK ARRANGEMENT – 46.00 m SPAN WITH END SPAN FROM ABUTMENT SP1– LONGITUDINAL SECTION



DECK ARRANGEMENT – 43.60 m SPAN – TOP VIEW



DECK ARRANGEMENT – 43.60 m SPAN – LONGITUDINAL SECTION



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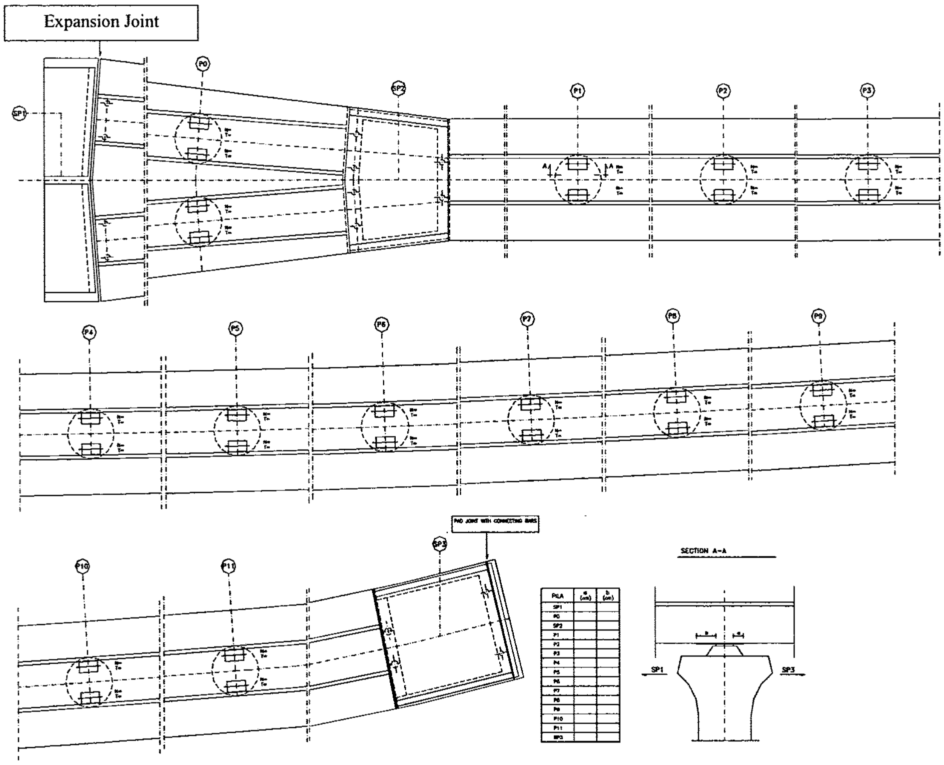
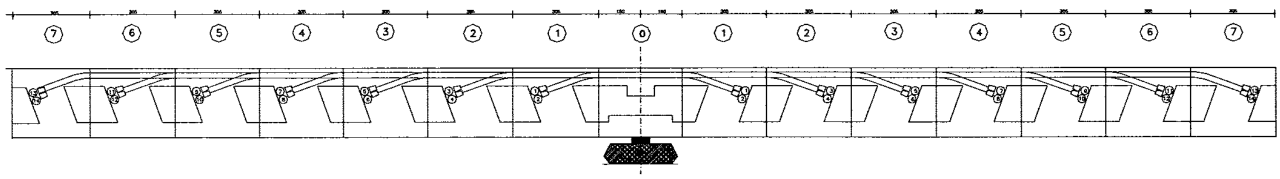
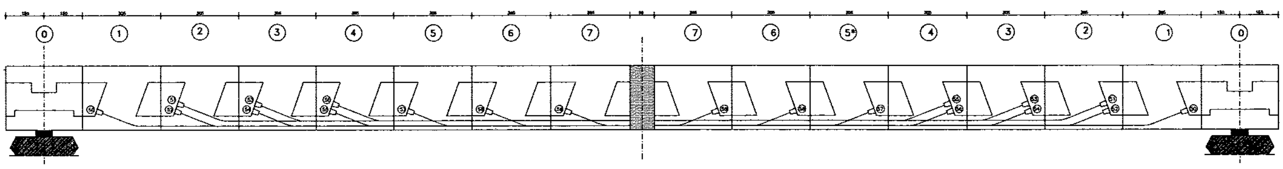
JOINTS AND BEARING DEVICES



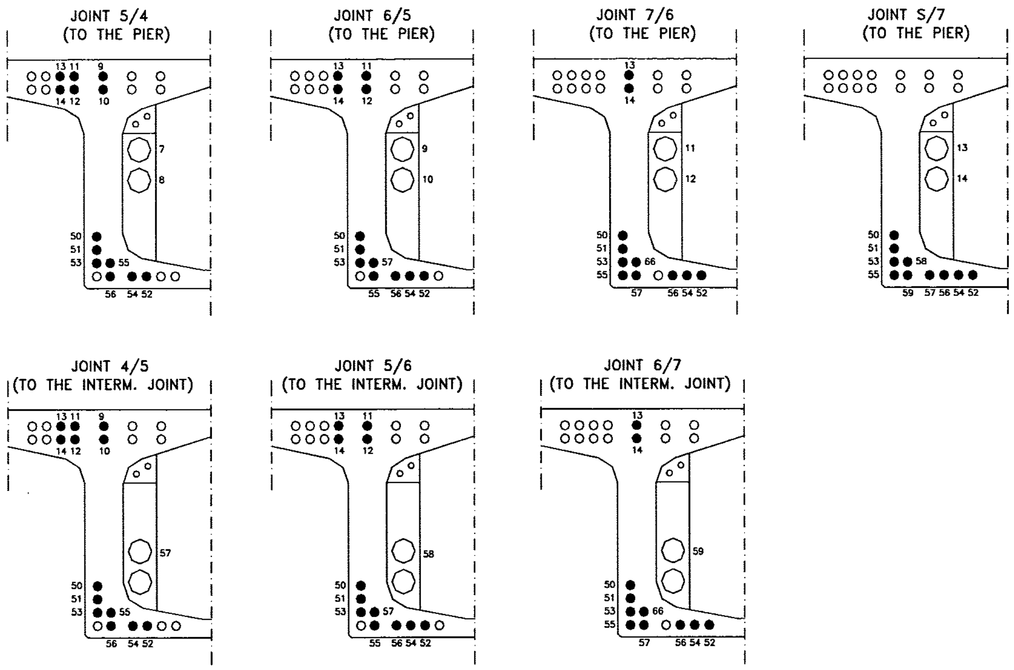
46.00 m HAMMER : TOP SLAB TENDONS LAYOUT



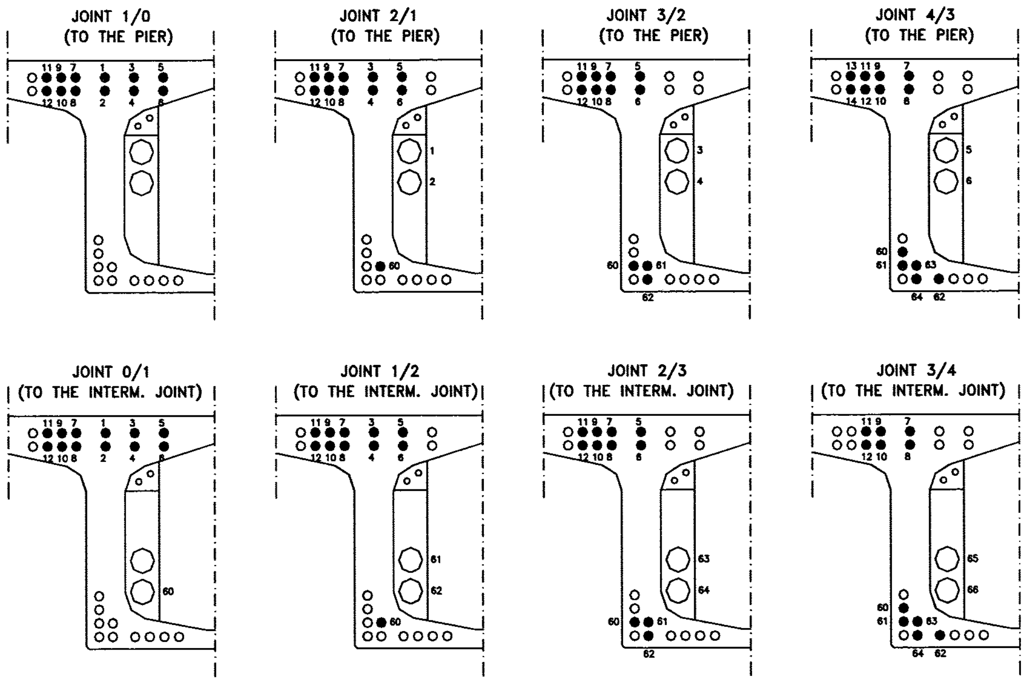
46.00 m HAMMER : BOTTOM SLAB TENDONS LAYOUT



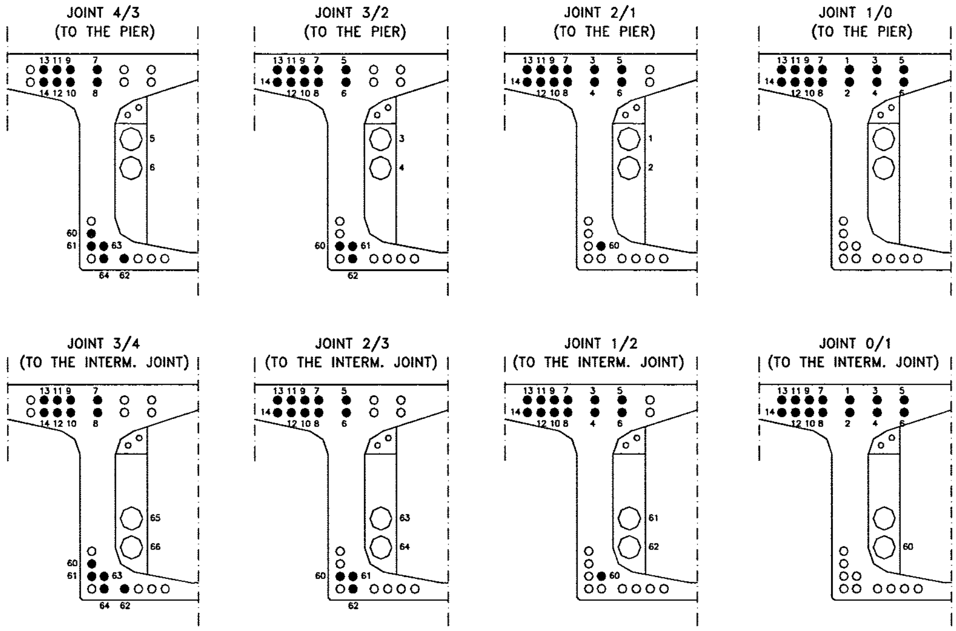
46.00 m HAMMER: TENDONS MASKS (PART 2)



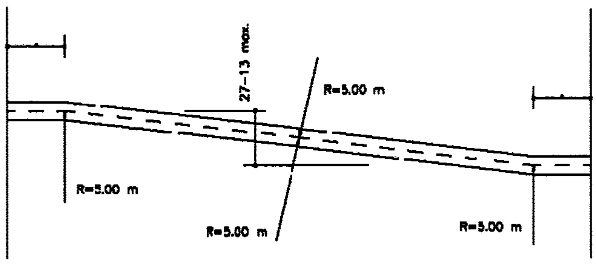
43.60 m HAMMER: TENDONS MASKS (PART 1)



43.60M HAMMER: TENDONS MASKS (PART 4)



TENDONS LAYOUT DETAILS: SCHEME FOR VERTICAL AND HORIZONTAL DEVIATIONS



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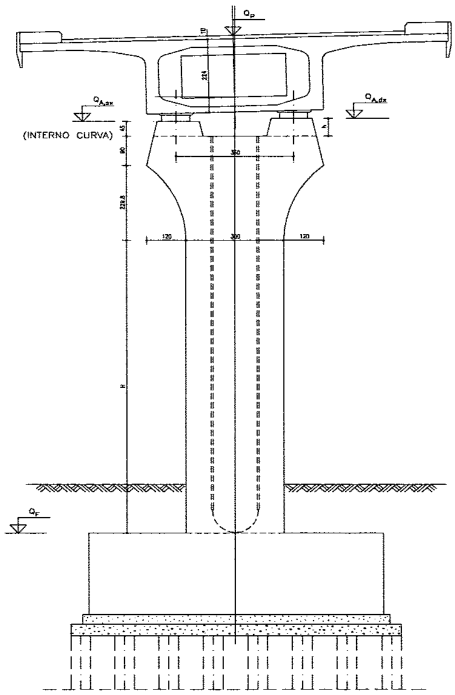
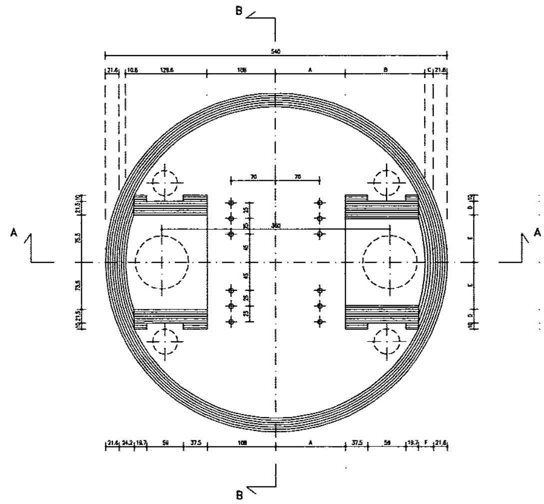
PIER: TYPICAL TRANSVERSE SECTION



PIER CAP: HORIZONTAL SECTION



PIER : VARIABLE DIMENSIONS

PILA	Q_P (m)	$Q_{A,sx}$ (m)	$Q_{A,dx}$ (m)	Q_F (m)	H (cm)	h (cm)
B1,B2	* 27.948	25.138	25.138	14.88	661	45
1	28.153	25.443	25.443	10.445	1135	45
2	28.300	25.662	25.752	11.214	1080	54
3	28.448	25.810	25.900	8.762	1340	54
4	28.587	25.949	26.039	12.331	997	54
5	28.727	26.089	26.179	12.77	967	54
6	28.866	26.228	26.318	11.86	1072	54
7	29.006	26.368	26.458	12.73	999	54
8	29.153	26.515	26.605	11.117	1175	54
9	29.300	26.662	26.752	11.214	1180	54
10	29.447	26.809	26.899	14.961	820	54
11	29.594	26.907	27.077	15.459	780	62

PILA	h (cm)	A (cm)	B (cm)	C (cm)	D (cm)	E (cm)	F (cm)
1,B1,B2	45	108	129.6	10.8	21.5	75.5	24.2
2,3,4,5,6 7,8,9,10	54	110.1	125.3	13	27.8	69.2	22.1
11	62	112	121.4	15	33.4	63.6	20.2

* PER LE PILE B1 E B2, Q_p E' RIFERITO ALL'ASSE DELL'INTERO IMPALCATO

PILA	DISASSAMENTO RADIALE (cm)
1,B1,B2	0
2–10	5.8
11	11

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ANALYSIS OF FIRST CONSTRUCTION PHASES

IN THE FOLLOWING PAGES WILL BE ANALYZED ONLY THE PROBLEMS CONCERNING HAMMERS CONSTRUCTION BEFORE CONCRETING OF THE KEY SEGMENTS AND INTRODUCTION OF THE BOTTOM SLAB PRESTRESSING. EACH SINGLE STRUCTURE IS THEN STATICALLY DETERMINED AND NO EFFECTS OF DELAYED RESTRAINT CAN BE APPRECIATED.

ASSEMBLING OF A SINGLE HAMMER

CANTILEVER CONSTRUCTION → TOP PRESTRESSING

CONTROL THE STRESSES IN JOINTS DURING EACH PHASE

POSSIBLE CONSTRUCTION PROCEDURE:

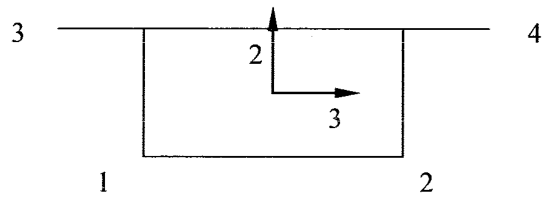
- POSITIONING OF THE PIER SEGMENT ON TWO TEMPORARY BEARINGS
- CONNECTION OF THE PIER SEGMENT WITH TEMPORARY PRESTRESSING TENDONS TO THE PIER FOR DISEQUILIBRIUM CONDITIONS DURING ASSEMBLING
- POSITIONING OF SEGMENT 1 (I.E. ON RIGHT SIDE) AND CONNECTION TO PIER SEGMENT WITH TEMPORARY PRESTRESSING BARS
- POSITIONING OF SEGMENT 1(ON LEFT SIDE) AND CONNECTION TO PIER SEGMENT WITH TEMPORARY PRESTRESSING BARS
- INSERT TOP TENDONS IN SEGMENT 1 AND PRESTRESSING (ATTENTION: SOME TENDONS CAN BE STRESSED IN DIFFERENT CONSTRUCTION STAGES)
- CONTROLLED RELEASE OF TEMPORARY BARS
- REPETITION OF THE PREVIOUS STEPS FOR ALL THE OTHER SEGMENTS UNTIL COMPLETION OF THE HAMMER

NO TENSILE STRESSES IN JOINTS ARE ALLOWED
THE JOINT SHOULD BE FULLY COMPRESSED

IN PARTICULAR:

- BEFORE STRESSING THE TENDONS OF SEGMENT K CONTROL THE STRESSES BETWEEN SEGMENT (I-1) AND SEGMENT I WITH $I \leq K-1$

actions: <u>weight of the segment</u>
$A = 6.495\ m^2$



Section conventions and key points

FIGURE 2

PHASE 2. ASSEMBLING OF SEGMENT 2

		FASE 4 - tempo: 33 [gg] - sollecitazioni totali											
		Permanenti						Precompressione					
Elemento	P.to	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
1	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2	0.0	448.7	0.0	0.0	0.0	-155.2	-28965.6	1889.2	-2.4	0.4	1.2	-6111.0
2	1	0.0	-3541.8	0.0	970.7	0.0	-176.5	-29441.6	-672.8	0.3	-593.7	1.2	-6857.9
2	2	0.0	-1594.8	0.0	970.7	0.0	7533.1	-30342.8	-665.7	0.3	-593.7	0.2	-5252.8
3	1	0.0	-1594.8	0.0	970.7	0.0	7533.1	-30361.8	-1366.0	0.3	-593.7	0.2	-5270.5
3	2	0.0	352.2	0.0	970.7	0.0	9398.2	-30677.2	-1384.7	0.3	-593.7	-0.7	-1072.2
4	1	0.0	352.2	0.0	970.4	0.0	9398.3	-30694.6	-1128.9	-0.7	-593.6	-0.7	-1061.2
4	2	0.0	2299.2	0.0	970.4	0.0	5418.8	-30444.9	-1107.3	-0.7	-593.6	1.3	1948.5
5	1	0.0	2299.2	0.0	970.6	0.0	5418.8	-30306.1	-2158.1	0.3	-593.6	1.3	2145.4
5	2	0.0	4246.1	0.0	970.6	0.0	-4405.1	-29451.1	-2033.1	0.3	-593.6	0.4	8865.3
6	1	0.0	-1588.1	0.0	31.6	0.0	-4406.8	-17297.9	1.1	0.0	-0.9	0.4	14164.2
6	2	0.0	-1134.1	0.0	31.6	0.0	-3454.1	-17250.8	1.1	0.0	-0.9	0.4	14126.4
7	1	0.0	-1134.1	0.0	25.3	0.0	-3454.1	-17250.1	127.9	5.6	3.9	0.4	14125.9
7	2	0.0	-1080.0	0.0	25.3	0.0	-3288.1	-17223.9	127.7	5.6	3.9	-0.5	14085.3
8	1	-133.9	-1071.7	0.0	8.5	1.1	-3288.2	-16573.9	4392.2	-0.2	-0.3	-0.5	13590.0
8	2	-66.7	-533.9	0.0	8.5	1.1	-820.5	-7577.1	1262.0	-0.1	-0.3	0.1	2803.5
9	1	0.0	-536.1	0.0	0.0	0.0	-820.6	-7471.7	1767.8	0.0	0.0	0.1	2768.3
9	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

		Fase 4 - tempo: 33 [gg]							
		Concio 2		Cavi concio 2		Prima della prec.		Dopo la prec.	
Elemento	P.to	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]
1	1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	2	0.004	0.004	-0.206	-0.446	-1.553	-0.046	-1.759	-0.492
2	1	0.005	0.004	-0.189	-0.463	-1.667	0.029	-1.856	-0.434
2	2	-0.071	0.080	0.037	-0.699	-0.973	-0.697	-0.936	-1.397
3	1	-0.071	0.080	0.037	-0.699	-0.976	-0.696	-0.938	-1.396
3	2	-0.148	0.155	0.273	-0.938	-0.601	-1.074	-0.329	-2.012
4	1	-0.148	0.155	0.273	-0.939	-0.601	-1.074	-0.328	-2.014
4	2	-0.225	0.229	0.447	-1.091	-0.865	-0.817	-0.418	-1.907
5	1	-0.225	0.229	0.448	-1.093	-0.840	-0.828	-0.393	-1.922
5	2	-0.301	0.301	0.446	-1.087	-1.124	-0.492	-0.678	-1.580
6	1	-0.303	0.301	0.446	-1.087	-0.113	-0.552	0.334	-1.639
6	2	-0.265	0.266	0.445	-1.085	-0.016	-0.643	0.429	-1.728
7	1	-0.480	0.377	0.809	-1.661	-0.026	-1.038	0.783	-2.699
7	2	-0.460	0.355	0.806	-1.656	0.002	-1.059	0.809	-2.715
8	1	-0.913	0.430	1.413	-2.593	-0.231	-1.768	1.182	-4.361
8	2	-0.309	0.132	-0.145	-1.653	-0.309	0.132	-0.454	-1.521
9	1	-0.299	0.143	-0.142	-1.631	-0.299	0.143	-0.441	-1.489
9	2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

PHASE 4.ASSEMBLING OF SEGMENT 4

		FASE 6 - tempo: 35 [gg] - sollecitazioni totali											
		Permanenti						Precompressione					
Elemento	P.to	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
1	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2	0.0	448.7	0.0	0.0	0.0	-155.2	-45934.8	212.0	-3.9	0.3	1.8	-3711.8
2	1	0.0	-2645.4	0.0	493.3	0.0	-166.0	-46428.9	-2154.1	0.5	651.0	1.8	-4124.5
2	2	0.0	-698.4	0.0	493.3	0.0	4852.6	-47655.2	-2176.2	0.5	651.0	0.5	2073.5
3	1	0.0	-698.4	0.0	493.3	0.0	4852.6	-47674.2	-2874.9	0.5	651.0	0.5	2055.7
3	2	0.0	1248.6	0.0	493.3	0.0	4026.8	-48136.4	-2906.5	0.5	651.0	-1.0	10888.3
4	1	0.0	1248.6	0.0	493.1	0.0	4026.9	-48180.5	-2286.9	-1.1	650.6	-1.0	10917.1
4	2	0.0	3195.6	0.0	493.1	0.0	-2643.5	-47662.9	-2247.0	-1.1	650.6	2.3	17162.3
5	1	0.0	3195.6	0.0	493.0	0.0	-2643.5	-47565.6	-2161.8	0.5	651.2	2.3	17393.3
5	2	0.0	5142.6	0.0	493.0	0.0	-15158.3	-46621.1	-2037.4	0.5	651.2	0.9	24040.6
6	1	0.0	-2664.3	0.0	211.5	0.0	-15166.5	-34493.4	1.1	0.0	-2.4	0.9	29328.4
6	2	0.0	-2210.2	0.0	211.5	0.0	-13460.4	-34374.8	1.1	0.0	-2.4	0.9	29228.6
7	1	0.0	-2210.2	0.0	186.9	0.0	-13460.8	-34374.1	127.3	11.2	7.3	0.9	29228.1
7	2	0.0	-2156.2	0.0	186.9	0.0	-13133.3	-34378.7	127.4	11.2	7.3	-0.8	29213.3
8	1	-267.3	-2139.6	0.0	119.1	14.9	-13134.1	-33599.5	6497.9	-0.4	-1.7	-1.0	28604.2
8	2	-200.1	-1601.8	0.0	119.1	14.9	-7384.1	-23275.3	3207.6	-0.3	-1.7	0.3	10668.6
9	1	0.0	-1614.3	0.0	42.8	0.0	-7385.0	-23284.7	2060.9	0.2	-0.7	0.5	10690.4
9	2	0.0	-1076.2	0.0	42.8	0.0	-3282.0	-16095.9	316.0	0.1	-0.7	0.0	7094.7
10	1	0.0	-1076.2	0.0	8.5	0.0	-3282.3	-15896.5	2056.2	0.1	-0.2	0.0	7024.0
10	2	0.0	-538.1	0.0	8.5	0.0	-820.5	-7924.5	345.7	0.1	-0.2	-0.3	2922.1
11	1	0.0	-538.1	0.0	0.0	0.0	-820.6	-7717.1	1819.4	-0.1	0.0	-0.3	2850.0
11	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

		Fase 6 - tempo: 35 [gg]							
		Concio 4		Cavi concio 4		Prima della prec.		Dopo la prec.	
Elemento	P.to	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]
1	1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	2	0.004	0.006	-0.212	-0.461	-1.955	-0.924	-2.167	-1.385
2	1	0.004	0.005	-0.195	-0.479	-2.034	-0.883	-2.229	-1.362
2	2	-0.157	0.164	0.041	-0.717	-1.168	-1.810	-1.127	-2.527
3	1	-0.157	0.164	0.041	-0.717	-1.170	-1.809	-1.129	-2.526
3	2	-0.320	0.323	0.283	-0.960	-0.609	-2.381	-0.326	-3.341
4	1	-0.320	0.323	0.283	-0.962	-0.608	-2.384	-0.325	-3.346
4	2	-0.481	0.478	0.466	-1.137	-0.815	-2.147	-0.349	-3.284
5	1	-0.481	0.478	0.467	-1.140	-0.788	-2.163	-0.321	-3.303
5	2	-0.640	0.632	0.465	-1.133	-1.352	-1.549	-0.886	-2.682
6	1	-0.642	0.632	0.465	-1.133	-0.343	-1.607	0.122	-2.740
6	2	-0.603	0.595	0.465	-1.132	-0.173	-1.764	0.292	-2.896
7	1	-1.092	0.843	0.845	-1.733	-0.304	-2.872	0.541	-4.605
7	2	-1.073	0.823	0.844	-1.732	-0.246	-2.917	0.598	-4.649
8	1	-2.116	1.012	1.488	-2.728	-1.066	-5.172	0.422	-7.900
8	2	-1.506	0.713	0.254	-2.127	-2.672	-2.056	-2.418	-4.183
9	1	-1.496	0.723	0.256	-2.144	-2.636	-2.015	-2.381	-4.159
9	2	-0.897	0.432	0.253	-2.127	-1.342	-1.013	-1.089	-3.140
10	1	-0.897	0.432	0.253	-2.125	-1.337	-0.972	-1.084	-3.097
10	2	-0.299	0.143	-0.155	-1.728	-0.299	0.143	-0.454	-1.585
11	1	-0.299	0.143	-0.150	-1.683	-0.299	0.143	-0.449	-1.541
11	2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

PHASE 6. ASSEMBLING OF SEGMENT 6

		FASE 8 - tempo: 37 [gg] - sollecitazioni totali											
		Permanenti						Precompressione					
Elemento	P.to	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
1	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2	0.0	448.7	0.0	0.0	0.0	-155.2	-63124.5	-1570.4	-5.3	0.2	2.5	-1261.8
2	1	0.0	-1202.8	0.0	-310.1	0.0	-146.5	-63639.5	-3729.3	0.6	2001.0	2.5	-1320.4
2	2	0.0	744.2	0.0	-310.1	0.0	539.9	-65104.5	-3774.2	0.6	2001.0	0.7	9718.3
3	1	0.0	744.2	0.0	-310.1	0.0	539.9	-65123.5	-4471.7	0.6	2001.0	0.7	9700.6
3	2	0.0	2691.1	0.0	-310.1	0.0	-4616.2	-65829.1	-4526.0	0.6	2001.0	-1.1	23505.1
4	1	0.0	2691.1	0.0	-309.9	0.0	-4616.2	-65907.3	-3480.9	-1.5	2000.3	-1.1	23557.6
4	2	0.0	4638.1	0.0	-309.9	0.0	-15616.9	-65212.5	-3428.0	-1.5	2000.3	3.5	33215.3
5	1	0.0	4638.1	0.0	-310.4	0.0	-15616.9	-65159.2	-2166.3	0.6	2001.4	3.5	33485.0
5	2	0.0	6585.1	0.0	-310.4	0.0	-32462.0	-64113.3	-2042.3	0.6	2001.4	1.7	40049.3
6	1	0.0	-3740.4	0.0	676.5	0.0	-32484.4	-52004.5	1.1	0.0	-4.4	1.7	45328.8
6	2	0.0	-3286.4	0.0	676.5	0.0	-30025.0	-51835.6	1.1	0.0	-4.4	1.7	45183.6
7	1	0.0	-3286.4	0.0	621.6	0.0	-30026.2	-51834.9	127.1	16.9	10.7	1.7	45183.0
7	2	0.0	-3232.4	0.0	621.6	0.0	-29537.3	-51896.2	127.2	16.9	10.7	-0.9	45217.1
8	1	-400.7	-3207.5	0.0	467.6	58.4	-29540.1	-50984.6	8649.6	-0.7	-3.6	-1.3	44487.3
8	2	-333.5	-2669.7	0.0	467.6	58.4	-20507.7	-40265.4	5312.9	-0.5	-3.6	0.6	19716.0
9	1	0.0	-2690.5	0.0	256.8	0.0	-20511.5	-40405.1	2056.4	0.3	-2.0	1.1	19806.9
9	2	0.0	-2152.4	0.0	256.8	0.0	-13126.3	-32823.7	315.2	0.2	-2.0	0.3	16000.3
10	1	0.0	-2152.4	0.0	119.8	0.0	-13128.2	-32624.8	2052.4	0.2	-1.1	0.3	15929.8
10	2	0.0	-1614.3	0.0	119.8	0.0	-7384.1	-24271.5	347.0	0.2	-1.1	-0.3	11617.1
11	1	0.0	-1614.3	0.0	42.9	0.0	-7385.0	-24057.6	2171.7	-0.3	-0.7	-0.3	11541.6
11	2	0.0	-1076.2	0.0	42.9	0.0	-3282.0	-15290.0	325.3	-0.2	-0.7	0.6	7156.8
12	1	0.0	-1076.2	0.0	8.5	0.0	-3282.3	-15084.4	2070.8	0.2	-0.1	0.6	7083.4
12	2	0.0	-538.1	0.0	8.5	0.0	-820.5	-7269.9	319.5	0.1	-0.1	-0.1	3124.4
13	1	0.0	-538.1	0.0	0.0	0.0	-820.6	-7038.3	1613.2	0.0	0.0	-0.1	3025.7
13	2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

		Fase 8 - tempo: 37 [gg]							
		Concio 6		Cavi concio 6		Prima della prec.		Dopo la prec.	
Elemento	P.to	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]
1	1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	2	0.004	0.007	-0.206	-0.456	-2.372	-1.836	-2.578	-2.292
2	1	0.005	0.007	-0.187	-0.475	-2.416	-1.831	-2.604	-2.307
2	2	-0.241	0.248	0.069	-0.738	-1.527	-2.793	-1.458	-3.531
3	1	-0.241	0.248	0.069	-0.738	-1.529	-2.792	-1.460	-3.530
3	2	-0.489	0.489	0.335	-1.009	-0.936	-3.410	-0.601	-4.419
4	1	-0.489	0.489	0.336	-1.012	-0.934	-3.416	-0.599	-4.428
4	2	-0.734	0.726	0.513	-1.168	-1.223	-3.099	-0.710	-4.267
5	1	-0.734	0.726	0.514	-1.171	-1.194	-3.120	-0.680	-4.292
5	2	-0.978	0.962	0.512	-1.164	-2.206	-2.063	-1.693	-3.227
6	1	-0.979	0.963	0.512	-1.164	-1.200	-2.120	-0.688	-3.284
6	2	-0.941	0.926	0.511	-1.162	-0.955	-2.347	-0.444	-3.506
7	1	-1.704	1.310	0.928	-1.772	-1.713	-3.959	-0.785	-5.731
7	2	-1.685	1.292	0.927	-1.770	-1.619	-4.039	-0.692	-5.809
8	1	-3.319	1.595	1.654	-2.760	-4.121	-7.742	-2.467	-10.502
8	2	-2.704	1.300	0.437	-2.125	-6.979	-3.982	-6.542	-6.107
9	1	-2.693	1.308	0.440	-2.141	-6.918	-3.957	-6.478	-6.098
9	2	-2.095	1.019	0.428	-2.081	-4.434	-3.472	-4.006	-5.553
10	1	-2.095	1.019	0.428	-2.081	-4.430	-3.429	-4.002	-5.510
10	2	-1.496	0.727	0.416	-2.021	-2.610	-2.451	-2.194	-4.472
11	1	-1.496	0.727	0.416	-2.021	-2.605	-2.405	-2.189	-4.426
11	2	-0.897	0.433	0.400	-1.958	-1.342	-1.069	-0.942	-3.027
12	1	-0.897	0.433	0.400	-1.956	-1.337	-1.026	-0.937	-2.983
12	2	-0.299	0.143	0.019	-1.662	-0.299	0.143	-0.280	-1.519
13	1	-0.299	0.143	0.019	-1.609	-0.299	0.143	-0.280	-1.467
13	2	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

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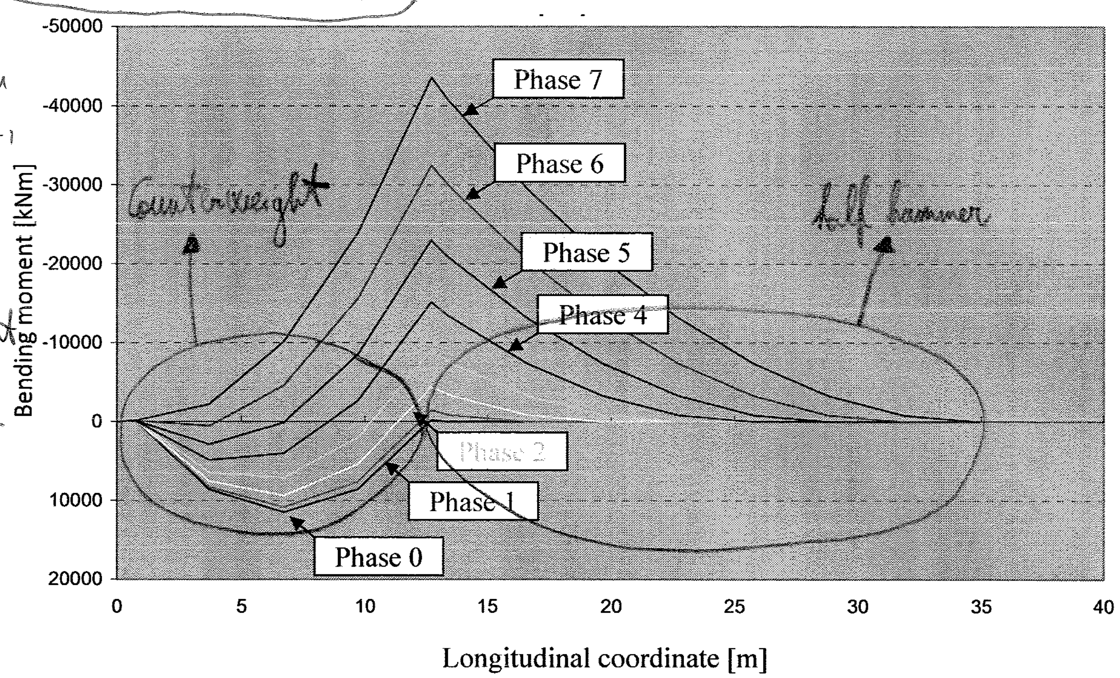
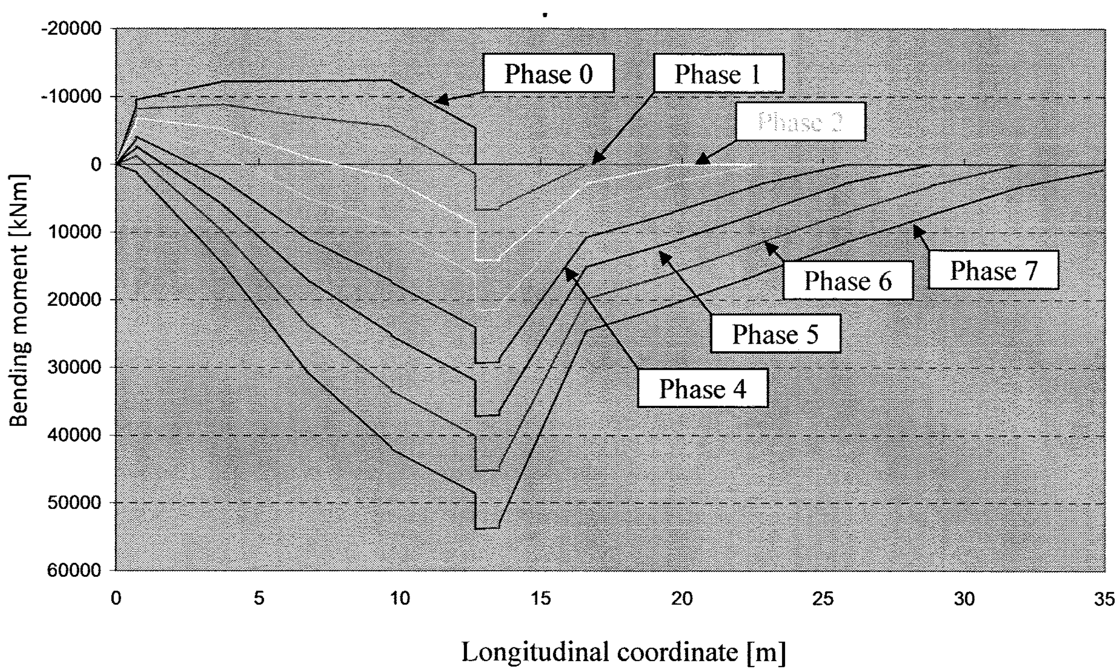
BENDING MOMENT IN COUNTERWEIGHT HALF HAMMER DUE TO SELF WEIGHT

In this figure is in the left side, while in the FEM figure seen before, the counterweight is on the right side.

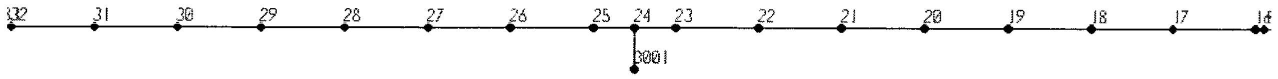


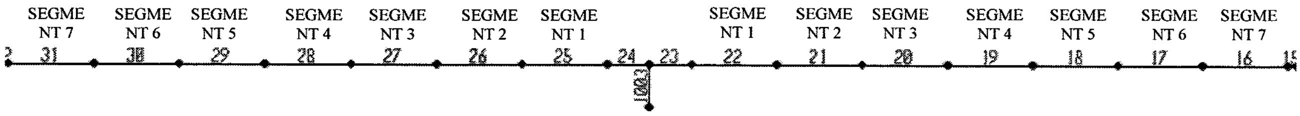
BENDING MOMENT IN COUNTERWEIGHT HALF HAMMER DUE TO PRESTRESSING



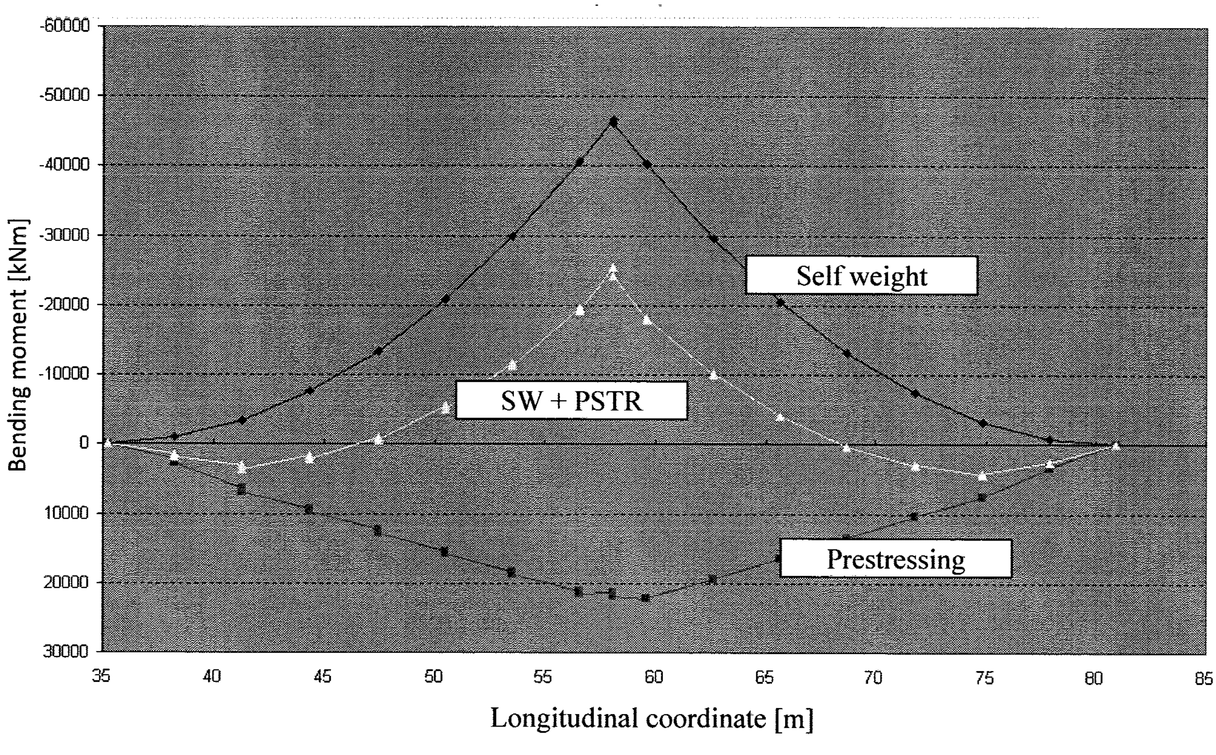
MESH OF THE FIRST HAMMER AFTER THE COUNTERWEIGHT HALF HAMMER

THIS HAMMER IS A TYPICAL ONE AND IS REPEATED 6 TIMES ALONG THE BRIDGE.





STRESSES IN THE TYPICAL HAMMER



new segment added

overturning moment

$$M_S = P \cdot \left(a + \frac{\Delta}{2} - \frac{b}{2} \right) - \mathbf{n} P \cdot \frac{b}{2}$$

$$M_R = T \cdot b$$ stress into the temporary tendon

Assuming a safety coefficient against overturning equal to 1.5:

we use 1.5 and not a lower value (always > 1) because when I put the new segment, it can fall down (for some problem). ⇓ dynamic effect.

$$\text{Mres} = 1.5 \cdot \text{Msol} \quad \Rightarrow \quad T = \frac{1.5 \cdot \text{Msol}}{b}$$

Substituting the values:

$$\text{Msol} = 538 \cdot \left(19.82 + \frac{3.05}{2} - \frac{2.00}{2} \right) - 13 \cdot 538 \cdot \frac{2.00}{2} = 3951.6 kNm$$

$$\Rightarrow \quad T = \frac{1.5 \cdot 3951.6}{2.00} = 2963.7 kN$$

$$M_S = 7 \cdot 538 \cdot \left(\frac{21.35}{2} + 0.8 \right) = 43214.9 \textbf{kNm}$$

$$M_R = 8691 \cdot \left(\frac{13.4}{2} - 0.8 \right) = 51276.9 \textbf{kNm}$$

As $\frac{M_R}{M_S} = 1.19$

is less than the required safety factor, it is necessary to use anchorages: to give the required stability.

$$Mres = P_{CP2} \cdot \left(\frac{b}{2} - c \right) + T \cdot d$$

and assuming $M_R = 1.5\ M_S$ it can be obtained the value of the reaction T.

$$T = \frac{1.5 \cdot Msol - P_{CP2} \cdot \left(\frac{b}{2} - c \right)}{d} = \frac{1.5 \cdot 43214.9 - 51276.9}{12.00} = 1128.8 kN$$

These 2 verification are done at the end of the construction (because the level arm is longer at the end of the construction → the worst situation)

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TRANSVERSE ANALYSIS

- THE PREVIOUS ANALYSIS CONSIDERS THE BRIDGE AS ONE-DIMENSIONAL BEAM
- IN GENERAL, BRIDGES CANNOT BE CONSIDERED AS ONE-DIMENSIONAL STRUCTURES
- THE 3D STRUCTURE "BRIDGE" IS ANALYZED IN A FIRST STEP IN LONGITUDINAL DIRECTION LIKE A SIMPLE BEAM AND SUBSEQUENTLY IN THE TRANSVERSE DIRECTION TO SIMPLIFY THE PROCEDURE
- IN CASE OF BOX SECTION GIRDERS THE TRANSVERSE ANALYSIS PRESENTS DIFFICULTIES BECAUSE OF THE PARTICULAR RESPONSE DUE TO THE TORSION BEHAVIOUR
- THE BREDT ANALYSIS IS GLOBALLY EQUILIBRATED BUT EACH ELEMENT OF THE BOX SECTION WHEN IS CONSIDERED ISOLATED IS NOT EQUILIBRATED IF A VARIABLE TORQUE MOMENT IS PRESENT (BREDT ANOMALY)
- IF WE DIVIDE THE ACTIONS ON THE BOX SECTION IN LONGITUDINAL AND TRANSVERSE DIRECTION IT IS POSSIBLE TO ISOLATE A SELF EQUILIBRATED SYSTEM THAT CREATES A DISTORTION OF THE SECTION
- THE DISTORTION IS PREVENTED BY BOTH TRANSVERSE BENDING STIFFNESS OF THE BEAM SEGMENT (CONSIDERED SEPARATED FROM THE BRIDGE) AND THE STRUCTURAL ACTION OF THE ADJACENT SEGMENTS THAT ARE NOT LOADED AND THEN HAVE NO REASON TO CHANGE THEIR SHAPE
- A COMBINATION OF TRANSVERSE AND LONGITUDINAL BEARING CAPACITY IS THEN ESTABLISHED BUT IT IS NOT SIMPLE TO EVALUATE THIS EFFECT FOR EACH REAL LOAD CONDITION
- THE PROPOSED ANALYSIS FOLLOWS SCHLAICH' S PROCEDURE

When a segment of a bridge is loaded by an eccentric load, this segment is warped, but the previous and following segments don't want to warp so they contrast → we have additional bearing capacity.

(Procedure for taking in account the transversal effects 2-3) → from the lessons

Look the table in the slide

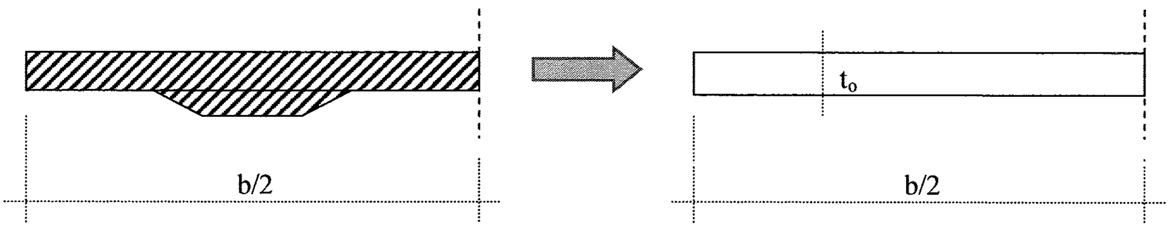
1.1 Geometry parameters

$b = 13.10$ m as in reality
$t_s = 0.40$ m as in reality

$b_0 = b_u = (1.85+0.85+0.20) \times 2 = 5.00$ m
$b_k = (13.10-5.00)/2 = 8.10/2 = 4.05$ m

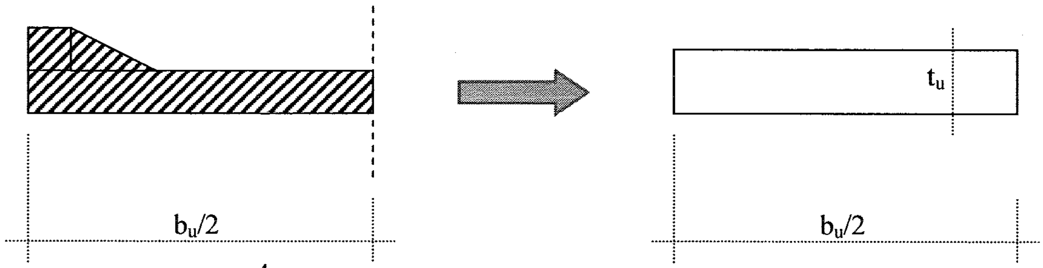
FOR THE COMPUTATION OF EQUIVALENT THICKNESSES OF THE SLABS WE ADOPT A CRITERION BASED ON THE ASSUMPTION OF EQUAL INERTIA MOMENTS

TOP SLAB



$I_o = 53.69$ m^4 $t_0 = 0.287$ m

BOTTOM SLAB



$I_o = 4.77$ m^4 $t_u = 0.458$ m

THEN THE DEPTH OF THE WEBS CAN BE CALCULATED AS FOLLOWS

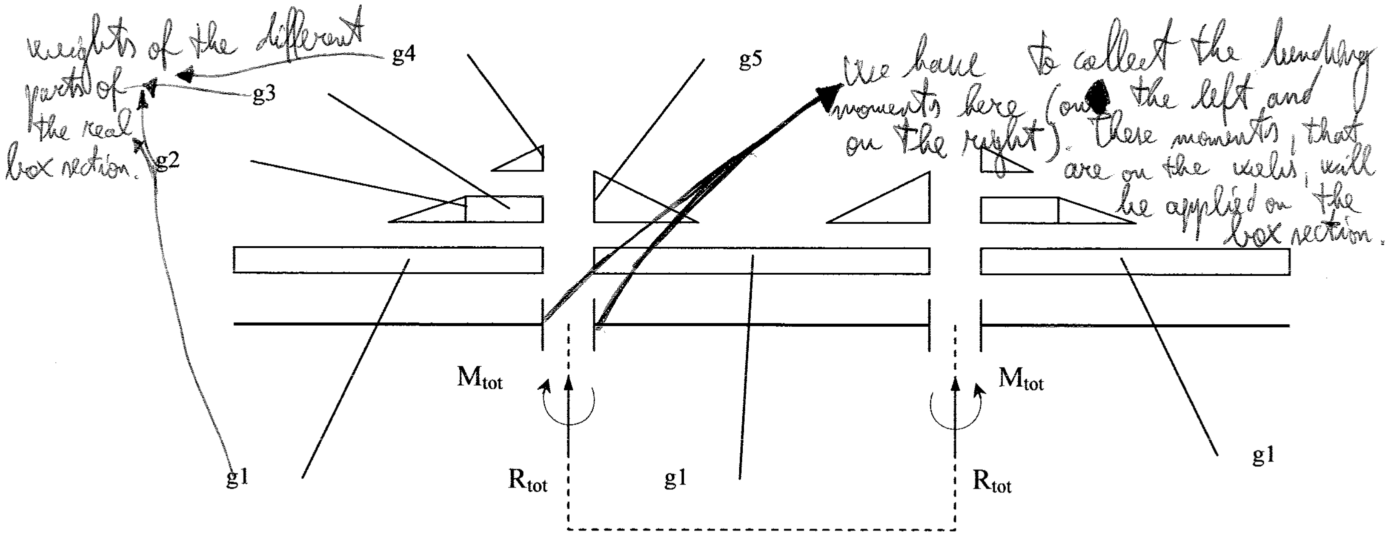
$b_s = 2.24 - 0.458/2 - 0.287/2 = 1.87$ m

AND THE INERTIA PROPERTIES OF THE SINGLE WALLS ARE:

2. STRESSES DUE TO SELF WEIGHT

THE EFFECTS OF ANY ACTION APPLIED TO THE TOP SLAB IS EVALUATED CONSIDERING IN A FIRST STEP THE TOP SLAB RIGIDLY RESTRAINED IN THE WEBS AND IN A SECOND STEP USING THE REACTION OBTAINED IN THE 1ST STEP WITH OPPOSITE SIGN TO LOAD THE BOX SECTION

2.1 RIGID RESTRAINT REACTIONS EVALUATION



$g_1 = 0.24 \cdot 25 = 6$ kN/m^2 $\rightarrow$ G_1(est)= $6 \cdot 3.85 = 23.1$ kN/m distance G1= 3.85/2 + 0.2 = 2.12 m

G1(int) = $6 \cdot 4.60 = 27.6$ kN/m

$g_2 = 0.08 \cdot 25 = 2$ kN/m^2 $\rightarrow$ $G_2 = 2 \cdot 1.075/2 = 1.075$ kN/m distance G2= 1.07/3 + 1.2 = 1.56 m

$g_3 = 0.08 \cdot 25 = 2$ kN/m^2 $\rightarrow$ $G_3 = 2 \cdot 1 = 2$ kN/m distance G3= 1/2 + 0.2 = 0.7 m

$g_4 = 0.20 \cdot 25 = 5$ kN/m^2 $\rightarrow$ $G_4 = 5 \cdot 1/2 = 2.5$ kN/m distance G4= 1/3 + 0.2 = 0.533 m

$g_5 = 0.31 \cdot 25 = 7.75$ kN/m^2 $\rightarrow$ $G_5 = 7.75 \cdot 1/2 = 3.875$ kN/m

$$m_{AB} = -\frac{1+2r_u}{K_3} \cdot \frac{m_R}{2} = -2.759 \text{ kNm/m}$$

$$m_{AD} = m_{AB} + \frac{m_R}{2} = 32.198 \text{ kNm/m}$$

$$m_D = -\frac{r_0}{K_3} \cdot \frac{m_R}{2} = -4.379 \text{ kNm/m}$$

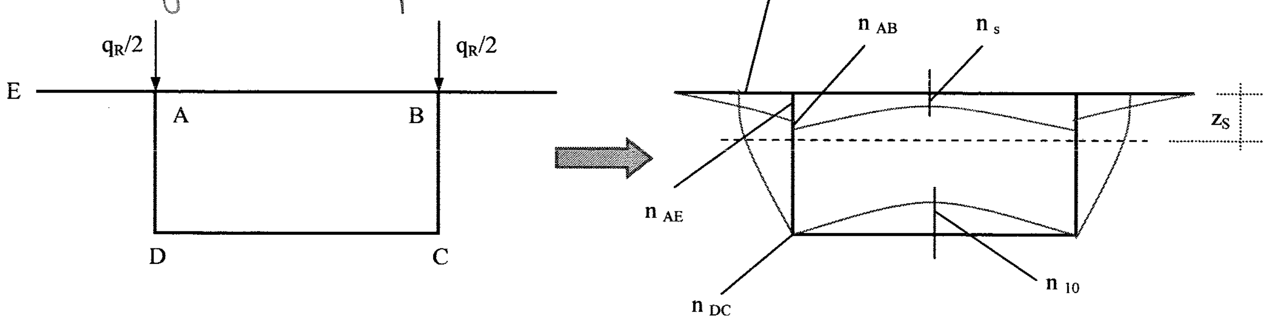
$$n_0 = \frac{3r_0(1+r_u)}{K_3} \cdot \frac{m_R}{2} \cdot \frac{1}{d} = 19.592 \text{ kN/m}$$

$$n_u = -n_0 = -19.592 \text{ kN/m}$$

$$n_S = n_u \cdot \sin\varphi = 0 \text{ kN/m}$$

2.1.2 EVALUATION OF STRESSESS DUE TO R_{TOT}

Now we manage the vertical part of the load:



From the mass geometry and with $\varphi = 0$:

$z_S = 0.722$ m e $I_X = 4.158$ m^4

$q_R = 2\ R_{tot} = 100.6$ kN/m

$$n_{AE} = \frac{q_R}{2I_X}\ t_0\ b_k^{\,2}\ z_S = 41.129 \text{ kN/m}$$

$$n_{AD} = -\frac{q_R}{2} = -50.3 \text{ kN/m}$$

Left side	q [kN/m]	dist [m]	Right side	q [kN/m]	Reactions Left side		Right side	
paviment.	8.25	1.38		3.00	R [kN/m]	M [kNm/m]	R [kN/m]	M [kNm/m]
Kerb	5.74	3.40			20.74	57.03	7.50	6.25
Barrier	2.00	3.40						
Pavement	2.75	4.10			**Total reaction**		**Rtot**	**28.24**
Parapet	2.00	4.05					**Mtot**	**50.78**

THE EVALUATION OF THE STRESSES CAN BE DONE USING THE FOLLOWING RATIOS

$$\frac{M_{\text{TOT perm. carried}}}{M_{\text{TOT sefl weight}}} = 1.452 \qquad \frac{R_{\text{TOT perm. carried}}}{R_{\text{TOT self weight}}} = 0.561$$

AND MULTIPLY BY THE STRESSES DUE TO THE SELF WEIGHT.

qR	56.48	kN/m	**m r**	101.55	kNm/m
n AE	23.09	kN/m	K3	57.87	
n AD	-28.24	kN/m	m AB	-4.01	kNm/m
n AB	23.09	kN/m	m AD	46.77	kNm/m
n DC	0.00	kN/m	m D	-6.36	kNm/m
n S	-8.99	kN/m	n o	28.46	kN/m
n 10	22.26	kN/m	n u	-28.46	kN/m
n 5	14.29	kN/m	n s	0.00	kN/m

Section	AB		AD		DA		DC		AE	
Caused	N	M	N	M	N	M	N	M	N	M
Fix res	0.00	-6.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-57.03
Actions	23.09	0.00	-28.24	0.00	0.00	0.00	0.00	0.00	23.09	0.00
Moment	28.46	-4.01	0.00	46.77	0.00	-6.36	-28.46	-6.36	0.00	0.00
Total	**51.55**	**-10.26**	**-28.24**	**46.77**	**0.00**	**-6.36**	**-28.46**	**-6.36**	**23.09**	**-57.03**

$$l_0 = 1.50 \cdot 2 + 0.74 + 2 \cdot 2.60 = 8.94 \text{ m}$$

$$l_1 = 1.5 \cdot 2 + 0.94 + 2 \cdot 0.6 = 5.14 \text{ m}$$

three loads most far from the web.

REACTION OF FIXED RESTRAINT (LEFT WEB)

the 3 loads closer to the web.

$$q_A = \frac{300}{8.94} + \frac{300}{5.14} = 91.92 \text{ kN/m}$$

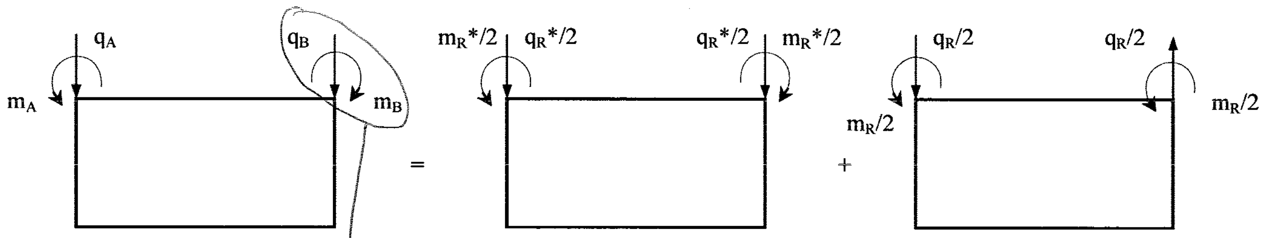
$$m_A = \frac{300 \cdot 2.6}{8.94} + \frac{300 \cdot 0.6}{5.14} = 122.27 \text{ kNm/m}$$

FOR THIS VALUES A DINAMIC COEFFICIENT THAT FOR THE TRANSVERSAL ANALYSIS IS **1.4.**

$$q_A = 128.69 \text{ kN/m}$$

$$m_A = 171.17 \text{ kNm/m}$$

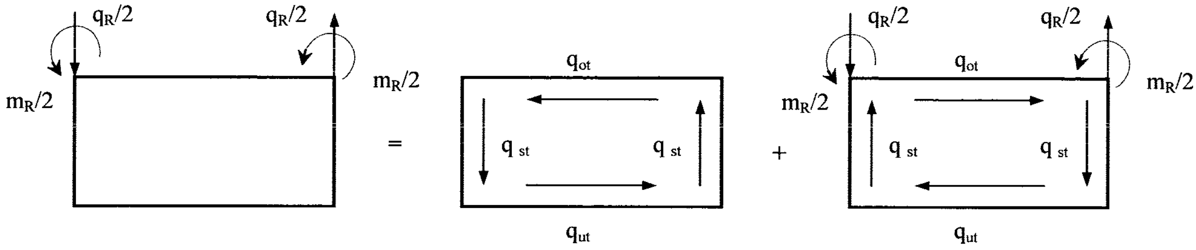
THE APPLIED LOAD IS NOT SYMMETRIC AND THEN IT CAN BE SEEN AS THE SUM OF A SYMMETRIC AND AN ASYMMETRIC LOAD



SIMM.
$$\begin{cases} q_R^* = q_A + q_B \\ m_R^* = m_A + m_B \end{cases}$$

ASYMM.
$$\begin{cases} q_R = q_A - q_B \\ m_R = m_A - m_B \end{cases}$$

are 0 because we are managing the left end.



TORSION PART:

$$\Delta M_t = \frac{q_R}{2} b_0 + m_R = 492.89 \text{ kNm/m}$$

$$q_{0t} = \frac{\Delta M_t}{2 \cdot A_K} b_0 = 132.00 \text{ kN/m}$$

$$q_{St} = \frac{\Delta M_t}{2 \cdot A_K} b_S = 42.29 \text{ kN/m}$$

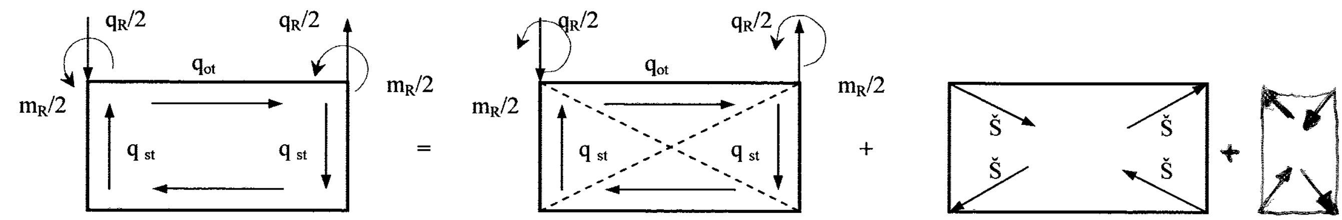
$$q_{ut} = \frac{\Delta M_t}{2 \cdot A_K} b_u = 132.00 \text{ kN/m}$$

ATTENTION
THIS STRESS FLOW CORRESPONDS TO THE TANGENTIAL STRESSES DUE TO THE TORQUE MOMENT AND THEN IS ALREADY CONSIDERED IN THE LONGITUDINAL ANALYSIS

DISTORTION PART:

FOR THIS ACTION, THE RESISTING MECHANISM IS A COMBINATION OF A LONGITUDINAL AND TRANSVERSE LOAD BEARING CAPACITY

lo



LOCAL EFFECTS

GLOBAL EFFECT (FOLDED PLATE)
effect

LOCAL EFFECTS OF THE DISTORTIONAL PART OF THE LOAD

$$K_4 = (r_0 + 2)(r_u + 2) - 1 = 34.00393$$

$$S = -\left(\frac{\beta}{1+\beta} + \frac{r_0 - 3 \cdot \beta - 2 \cdot \beta\, r_u}{K_4} \right) \cdot \frac{g\, m_R}{b_u d} = -50.89\ \text{kN/m}$$

$$m_{AB} = -\frac{3 + 2\, r_u}{K_4} \cdot \frac{m_R}{2} = -16.53\ \text{kN/m}$$

$$m_{AD} = m_{AB} + \frac{m_R}{2} = 69.05\ \text{kN/m}$$

$$m_D = -\frac{r_0}{K_4} \cdot \frac{m_R}{2} = -18.24\ \text{kN/m}$$

$$\cos\vartheta = \frac{4\, d^2 + b_0^2 - b_u^2}{4\, g\, b_S} = 0.3498$$

$$n_{AB} = \frac{m_R}{2\, d\, (1+\beta)} = 22.92\ \text{kN/m}$$

$$n_{AD} = \frac{S}{2} \cos\vartheta + \frac{2\, m_{AB}}{b_0} \cos\varphi - n_{AB}\, \text{sen}\varphi = -15.51\ \text{KN/m}$$

$$n_{DA} = n_{AD} + \frac{m_R\, b_S}{d\,(b_0 + b_u)} = 1.60\ \text{kN/m}$$

$$n_{DC} = -\beta \cdot n_{AB} = -22.92\ \text{kN/m}$$

IN THE FOLLOWING TABLE A SUMMARY OF THE LOCAL EFFECT OF THE WARPING PART OF THE LOAD IS SHOWN

Section	AB		AD		DA		DC		AE	
Action	N	M	N	M	N	M	N	M	N	M
Loads	43.08		-48.26		-16.09		-43.08			
Moments	22.92	-16.53	-15.51	69.05	1.60	-18.25	-22.92	-18.25		
Total	**66.00**	**-16.53**	**-63.77**	**69.05**	**-14.48**	**-18.25**	**-66.00**	**-18.25**		

GLOBAL EFFECTS OF THE WARPING PART OF THE LOAD

$$K = \frac{24b_S}{b_u^2 \cdot d^2} \cdot K_1 \cdot K_2 \cdot E \cdot \bar{I}_S = 9.27 \cdot 10^4$$

$$I_{Si} = \frac{2 \cdot \beta[(\alpha_0 + 2)(\alpha_u + 2) - 1]}{(1+\beta)(3 + 3\beta + \alpha_0 + \alpha_u \cdot \beta)} \cdot I_S = 0.916\,m^4$$

$$K_1 = \frac{(1+\beta)(2 + 2\beta + 2\beta^2 + \alpha_0 + \alpha_u \cdot \beta^2)}{3 + 3\beta + \alpha_0 + \alpha_u \cdot \beta} = 2$$

$$K_2 = \frac{2 + 2\beta + 2\beta^2 + r_0 + r_u \cdot \beta^2}{\beta[(r_0 + 2)(r_u + 2) - 1]} = 0.442$$

V IS THE DISPLACEMENT OF THE CONSIDERED PLATE IN ITS PLANE

X IS THE ABSCISSA OF THE BEAM

q IS THE LOAD THAT ACTS ON THE BEAM. THE q VALUE IS DIFFERENT IN THESE THREE CASES:

1. SUPERIMPOSITION ZONE OF THE EFFECTS OF THE TWO LANES OF THE q_{1A} (AS PREVIOUSLY CALCULATED)
2. ZONE WHERE THIS SUPERIMPOSITION IS MISSING

$$q_A = 1.4 \cdot \frac{300}{8.94} = 46.98\,kN/m$$

$$m_A = 1.4 \cdot \frac{300 \cdot 2.6}{8.94} = 122.15\,kNm/m$$

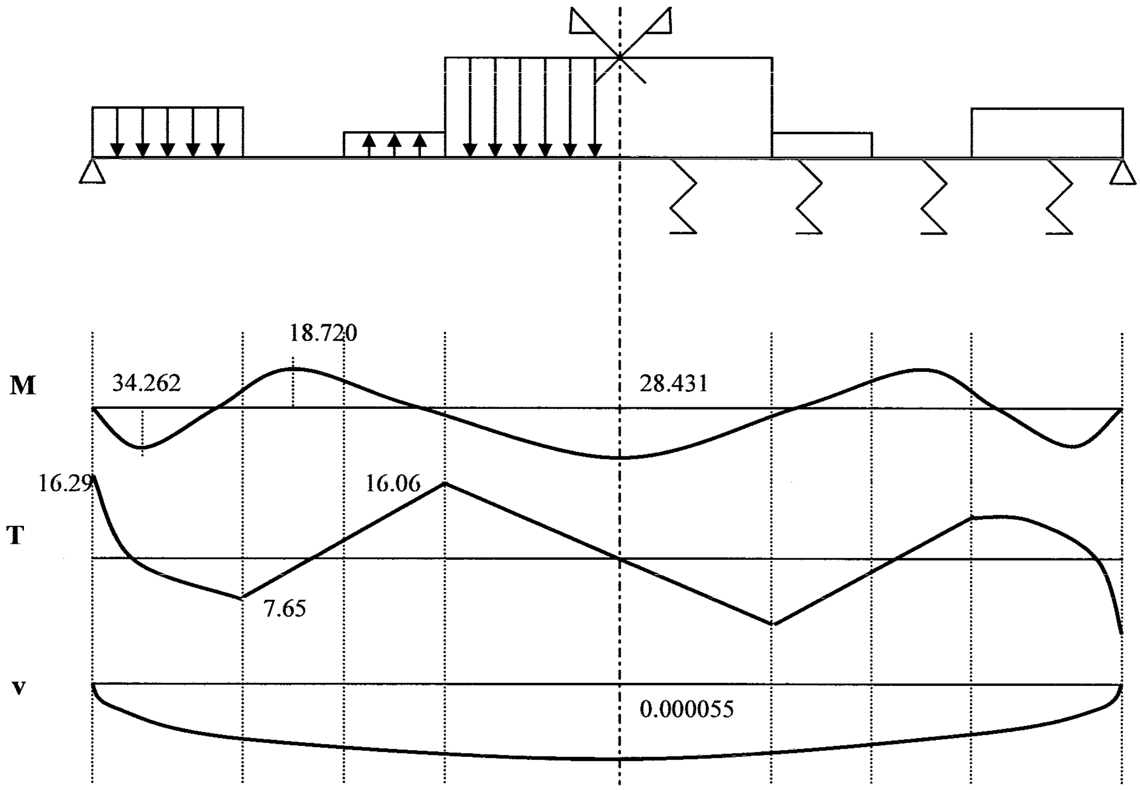
New reactions of fixed reastraint

REPEATING THE PROCEDURE WITH THE NEW REACTION OF FIXED RESTRAINT IT IS OBTAINED THE STRESSES INTO THE DIAGONALS AND CONSEQUENTLY TO THE SINGLE PLATES:

$$q_0 = -2.57\,kN/m$$
$$q_S = -0.96\,kN/m$$
$$q_u = -2.57\,kN/m$$

3. ZONE WHERE q_{1B} IS APPLIED

- STATIC SCHEME, THE LOAD OF THE BEAM ON ELASTIC FOUNDATION AND THE STRESSES AND DISPACEMENT RESULTS ARE INDICATED IN FIGURE; THE EXTREME BEARINGS REPRESENT THE IMPOSSIBILITY OF THE BOX GIRDER TO SWARP BECAUSE OF THE PRESENCE OF DIAPHRAGM INTO THE PIER SEGMENTS



THE LENGHTS OF THE LOAD ARE CALULATED AS FOLLOWS:

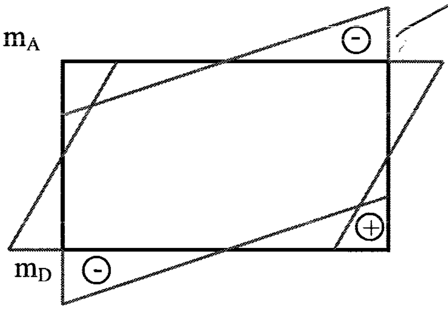
- q_S1 component along the web of the local effects derived from the warping part of the load $\mathbf{q_{1a}}$, in the zone where the external strip is superimposed to the internal one. This distributed load acts for a length of l_l= 5.74 m
- q_S2 component along the web of the global effects derived form the warping part of the load $\mathbf{q_{1a}}$, in the zone where the external strip of concentrated load is not superimposed on the internal one. This load acts externally to the first, for a length of $l_0 - l_l$ = 9.54 - 5.74 = 3.8 m. For each part, the length becomes that assumed in the picture: 3.8 / 2 = 1.90 m.
- q_S3 component along the web of the global effects derived from the warping part fo load $\mathbf{q_{1b}}$. In order to determine the active zones where is acts we refer to the standars:

- ONCE CALCULATED THE DISPLACEMENT IS ALSO POSSIBLE TO EVALUATE THE STRESS PORTION INTO THE DIAGONALS THAT INDUCE TRANSVERSAL STRESSES

$$\overline{S}(x) = K\frac{g}{b_S}v_S(x) = 14.58\ \text{kN/m}$$

the relation is linear with the displacements

obtained from the usual table



$$K_5 = 2 + 2\beta + 2\beta^2 + r_0 + r_u \cdot \beta^2 = 15.03$$

$$\cos\vartheta = \frac{4\cdot d^2 + b_0^2 - b_u^2}{4\cdot g\cdot b_S} = 0.3498$$

$$m_A = \frac{1+\beta(2+r_u)}{K_5}\cdot\frac{\overline{S}}{2}\cdot\frac{b_u\cdot d}{g} = 4.06\ \text{kNm/m}$$

$$m_D = -\frac{2+\beta+r_0}{K_5}\cdot\frac{\overline{S}}{2}\cdot\frac{b_u\cdot d}{g} = -8.69\ \text{kNm/m}$$

$$n_{AB} = n_{DC} = 0$$

$$n_{AD} = n_{DA} = \frac{2\cdot m_A}{b_0}\cos\varphi - \frac{\overline{S}}{2}\cos\vartheta = -0.93\ \text{kN/m}$$

- ALL THE STRESSES EVALUATED MUST BE ADDED IN THE FOLLOWING TABLE

	Section	AB		AD		DA		DC		AE	
		N	M	N	M	N	M	N	M	N	M
Self weight.		60.72	-18.99	-50.30	32.20	0.00	-4.38	-19.59	-4.38	41.13	-51.19
Other perm.		51.55	-10.26	-28.24	46.77	0.00	-6.36	-28.46	-6.36	23.09	-57.03
Load q1A	symmetr.	100.58	-6.76	-64.35	78.83	0.00	-10.72	-47.97	-10.72	52.61	-171.17
	asyim.	66.00	-16.53	-63.77	69.05	-14.48	-18.25	-66.00	-18.25	0.00	0.00
	action. S	0.00	4.06	-0.93	4.06	-0.93	-8.69	0.00	-8.69	0.00	0.00
	Total	**278.85**	**-48.48**	**-207.58**	**230.91**	**-15.41**	**-48.40**	**-162.02**	**-48.40**	**116.83**	**-279.38**

$$K_5 = 2 + 2\beta + 2\beta^2 + r_0 + r_u \cdot \beta^2 = 15.034$$

$$m_A = \frac{\beta(1 + 2\beta + r_u \cdot \beta)}{4(1+\beta)K_5} \cdot b_u \cdot q_R = 25.59 \text{ kNm/m}$$

$$m_D = -\frac{\beta(2 + \beta + r_0)}{4(1+\beta)K_5} \cdot b_u \cdot q_R = -54.84 \text{ kNm/m}$$

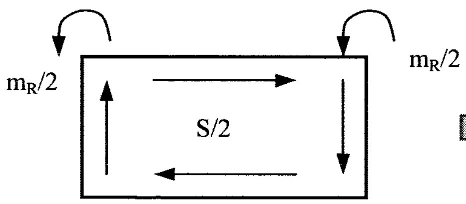
$$n_{AB} = \frac{b_0 \cdot q_R}{4(1+\beta) \cdot d} = 43.08 \text{ kN/m}$$

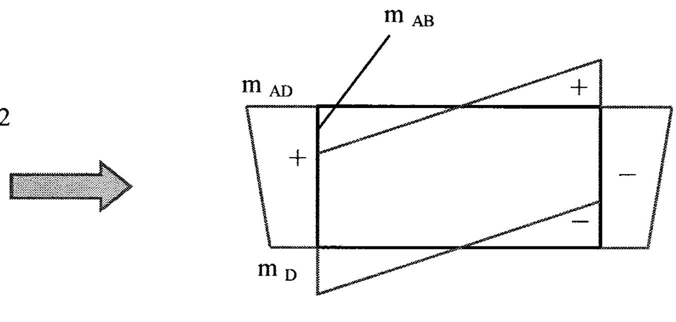
$$n_{DC} = -\beta \cdot n_{AB} = -43.08 \text{ kN/m}$$

$$n_{DA} = \frac{2m_D}{b_u} \cdot \cos\varphi + n_{DC} \cdot \operatorname{sen}\varphi = -21.94 \text{ kN/m}$$

$$n_{AD} = n_{DA} - \frac{b_S \cdot q_R}{2d(1+\beta)} = -54.11 \text{ kN/m}$$

- EFFECTS OF m_r





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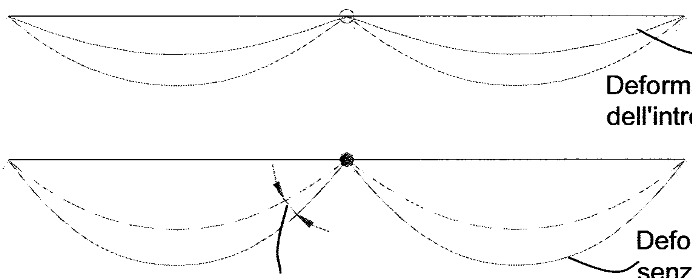
LEZ. 14-01-2014

STRUCTURAL EFFECTS OF THE CREEP

- THE CONSTRUCTION PROCEDURES ADOPTED TAKES INTO ACCOUNT 12 CHANGES OF STATIC SCHEME THAT CORRESPOND TO 12 CAST IN SITU JOINTS FOR THE HAMMERS
- THE CHANGE OF STATIC SCHEME, BECAUSE OF THE VISCOELASTIC PROPERTIES OF CONCRETE, DETERMINES STRESS CHANGES FROM THE STATIC SCHEME IN WHICH LOADS ARE INTRODUCED TO THE NEXT SCHEME
- IT CAN BE OBSERVED THAT REDISTRIBUTION OF STRESSES IS CONDITIONED BY THE EVOLUTION OF THE CREEP DEFORMATION NEGLECTED BY THE DELAYED RESTRAINT



- AN INCORRECT EVALUATION OF THE STRUCTURAL EFFECTS OF CREEP CREATES PROBLEMS IN SERVICEABILITY BEHAVIOUR WITH STRESS PATTERN AND CRACK OPENING BIGGER THAN EXPECTED (because in the SLU the opening of the joints is allowed, in the serviceability limit state no.)
- FOR THIS REASON IN ORDER TO EVALUATE THESE EFFECTS WE CONSIDER THE FOLLOWING TIME STEPS:
  - CONSTRUCTION OF THE COUNTERWEIGHT: 3 DAYS AFTER CASTING AND 28 DAYS FOR AGING, FOLLOWING INTRODUCTION OF PRESTRESSING FORCE
  - EACH SEGMENT WHEN IS LAUNCHED IS 28 DAYS OLD
  - THE CONSTRUCTION IS DONE LAUNCHING TWO SEGMENTS/DAY AND THEN THE HAMMER IS COMPLETED IN 7 DAYS
  - THE SEGMENT/JOINT IS CAST WHEN THE HAMMERS ARE COMPLETED AND AFTER 3 DAYS IT IS PRESTRESSED ALONG ITS BOTTOM SLAB
  - FINALLY, THE CONSTRUCTIVE PROCEDURE FOR PART 1 IS SHOWN IN FIGURE:

$$X_k^{(k)}(t) = X_k^{el(k)} \cdot \xi(t, t_k, t_0)$$

- Reactions for the delayed restraints introduced at time $t_j < t_k$

$$X_j^{(k)}(t) = X_j^{el(j)} \cdot \xi(t, t_j, t_0) + \sum_{m=j+1}^{k} a_{jm}^{(m-1)} \cdot X_m^{(m)}(t)$$

with $a_{jm}^{\ (m-1)}$ influence factor that is to say the stress effect that appears into the restraint j^{-mo} for the introduction of a unitary stress instead of the restraint m^{-mo}, evaluated in the static scheme $(m-1)^{-mo}$.

➢ IN THIS SITUATION WE NEED TO CALCULATE:

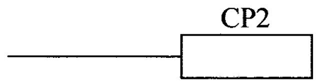
- REACTIONS IN THE PREVIOUS STATIC SCHEME FOLLOWING TO THE APPLICATION OF THE DELAYED RESTRAINT ($X_K^{el(k)}$ O $X_J^{el(j)}$).
- FUNCTION $\xi(t, t_k, t_0)$.
- COEFFICIENTS FOR THE ELASTIC REACTION a_{jm}, WHERE j IS THE DELAYED RESTRAINT PREVIOUSLY INTRODUCED AND m IS THE DELAYED RESTRAINT UNDER CONSIDERATION, EVALUATED UNDER THE PREVIOUS STATIC SCHEME m^{-mo}

EVOLUTION OF THE STATIC SCHEME

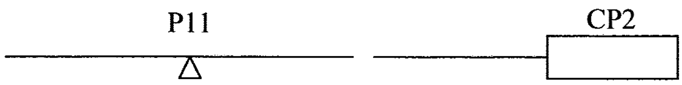
IN THE NEXT FIGURE IT IS SHOWED THE EVOLUTION OF THE STATIC SCHEME (FOR THE FIRST TWO CAST JOINTS, FOR THE FURTHERS IT IS POSSIBLE TO PROCEED IN THE SAME MANNER) ACCORDING TO THE PHASES IN THE PREVIOUS TABLE

<u>STATIC SCHEME 1</u>

Phase 9 : 0 ÷ 38 d.

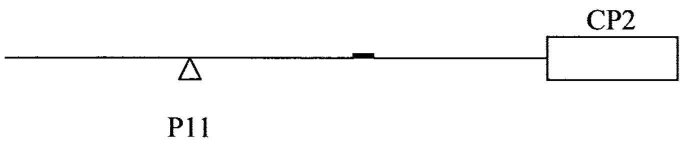


Phase 10 : 38 ÷ 45 d.

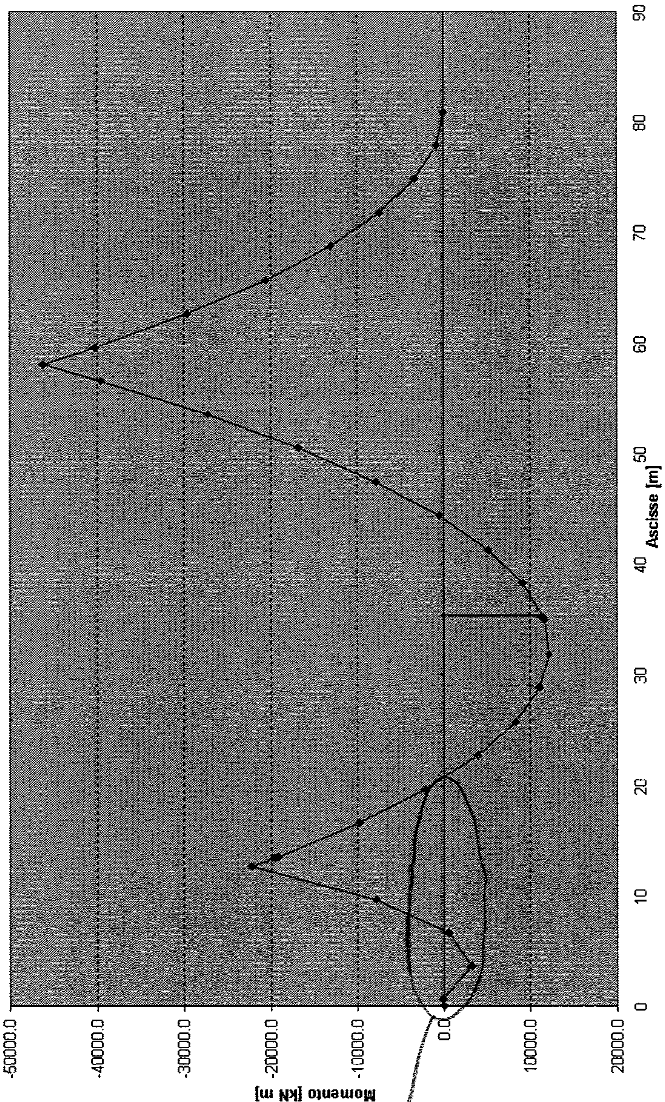


<u>STATIC SCHEME 2</u>

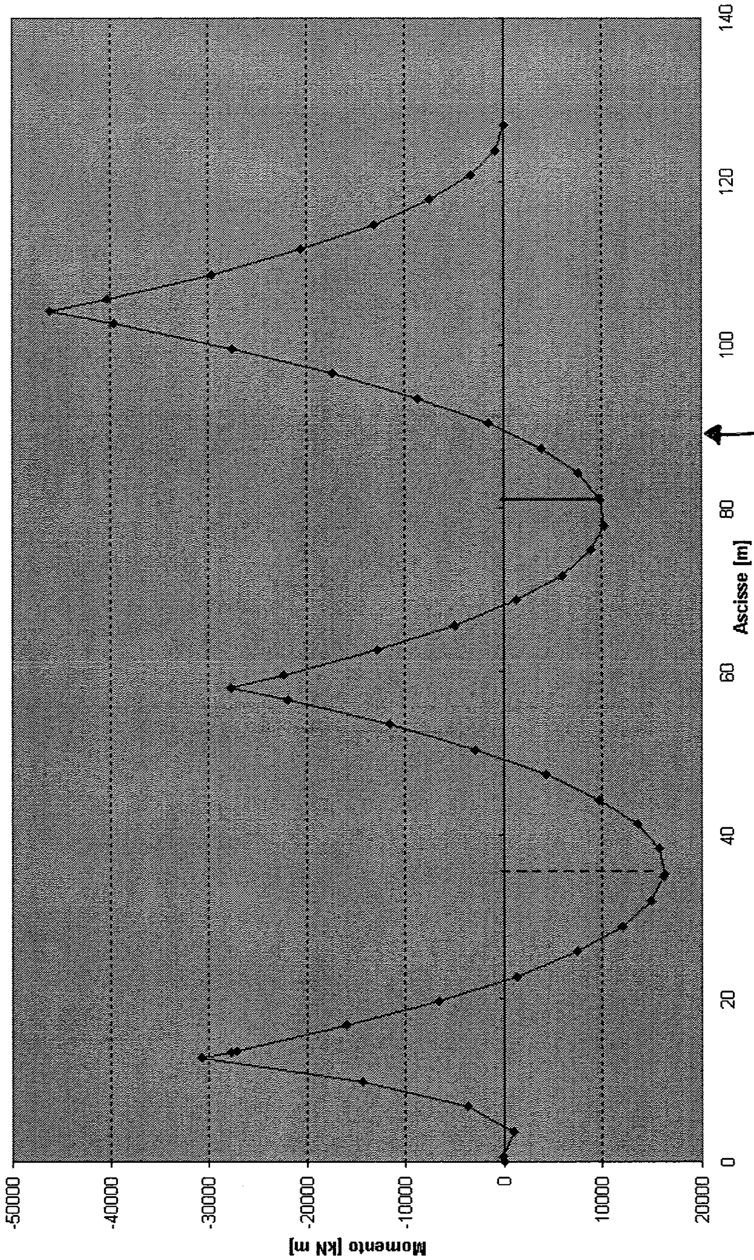
Phase 11 : 45 ÷ 48 d.



	Concio	M	T
inizio	1	0.0	2.11E-15
fine	1	-155.2	448.7456
Contrappeso	2	-157.7	-2066.6
	2	3123.7	-119.624
	3	3123.7	-119.624
	3	560.5	1827.356
	4	560.5	1827.356
	4	-7847.2	3774.35
	5	-7847.2	3774.35
	5	-22099.4	5721.33
	6	-22102.5	-3828.53
	6	-19581.5	-3374.51
	7	-19581.3	-3374.51
	7	-19079.2	-3320.49
Prima semistampella	8	-19078.3	-3294.89
	8	-9777.2	-2757.13
	9	-9774.3	-2778.54
	9	-2120.4	-2240.45
	10	-2116.9	-2240.45
	10	3895.9	-1702.36
	11	3899.3	-1702.36
	11	8270.9	-1164.27
	12	8273.7	-1164.27
	12	11004.1	-626.177
	13	11005.9	-626.177
	13	12095.1	-88.0841
	14	12095.6	-88.0841
	14	11543.7	450.0085
Concio di sutura 1	15	11543.4	450.0085
	15	11401.1	498.7214
Seconda stampella completa	16	11400.4	498.7214
	16	9058.7	1036.813
	17	9056.7	1036.813
	17	5073.8	1574.905
	18	5071.0	1574.905
	18	-553.0	2112.996
	19	-556.1	2112.996
	19	-7821.3	2651.089
	20	-7823.9	2651.089
	20	-16730.2	3189.181
	21	-16731.3	3189.181
	21	-27273.3	3727.031
	22	-27273.4	3727.031
	22	-39461.4	4265.125
	23	-39461.5	4265.125
	23	-46147.5	4649.577
	24	-46147.5	-4151.11
	24	-40209.2	-3766.66
	25	-40209.2	-3766.66
	25	-29540.5	-3228.51
	26	-29540.5	-3228.51
	26	-20513.3	-2690.36
	27	-20513.3	-2690.36
	27	-13127.6	-2152.21
	28	-13127.6	-2152.21
	28	-7383.4	-1614.07
	29	-7383.4	-1614.07
	29	-3280.7	-1075.92
	30	-3280.7	-1075.92
	30	-819.6	-537.772
	31	-819.6	-537.772
	31	0.0	2.78E-10
Concio di sutura 2	32		
	32		



here there is the counterweight



Elastic solution (the real solution will be: $\xi \cdot$ (elastic solution))

➢ THE RESULTS CAN BE SUMMARIZED INTO THE FOLLOWING TABLE, WHERE THE VALUES ON THE DIAGONAL ARE THE STRESSES OF THE RESTRAINT k^{-mo}, EVALUATED ON THE STATIC SCHEME k^{-mo} (IT IS AN AVERAGE VALUE INSIDE THE CAST JOINT SEGMENT); THE OTHER STRESSES ARE THE STRESSES IN THE STATIC SCHEME k^{-mo}, BUT INTO THE PREVIOUS CAST JOINT SEGMENTS IN WHICH WE ARE NOT INTERESTED.

FUNCTIONS $\xi(t, t_k, t_0)$

- t → TIME UNDER CONSIDERATION

  t_k → TIME FOR THE DELAYED RESTRAINT

  t_0 → LOAD APPLICATION TIME

- IT IS NECESSARY TO SIMPLIFY THE CONSTRUCTION PROCEDURE PREVIOUSLY DESCRIBED

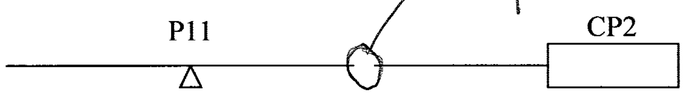
- EVALUATION OF t_0:

  FOR THE SINGLE HAMMER THE LOAD APPLICATION TIME IS DIFFERERENT FOR EACH SEGMENT . IN FACT THEY ARE LAUNCHED WHEN THEY ARE 28 DAYS OLD, BUT SEGMENT 1 IS LOADED FOR 7 DAYS (WITH THE WEIGHT OF THE OTHER SEGMENTS AND PRESTRESSING), WHILE SEGMENT 7 IS LOADED ONLY FOR ONE DAY. IN ORDER TO SIMPLIFY THE PROCEDURE WE ASSUME THAT THE ENTIRE HAMMER IS LOADED AT THE SAME TIME EQUAL TO THE AVERAGE OF THE LOADING TIME FOR THE FIRST AND THE LAST SEGMENT ($t_0 = 31$ DAYS FOR THE HAMMER WITH 6 SEGMENTS, $t_0 = 31.5 \approx 31$ DAYS FOR THE HAMMER WITH 7 SEGMENTS) It's related to the changing of the static scheme

- EVALUATION OF t_K:

  IT IS THE TIME IN WHICH IS INTRODUCED THE DELAYED RESTRAINT, REFERRING TO THE CASTING. CONSIDERING THE STATIC SCHEME 1

[when you close here the joint,.....]



THE HAMMER OF PIER P11 IS 28 + 7 + 3 = 38 DAYS

Age seg 1 — Time for construction of the hammer; — Time for cast joint

If we look to CP2: HALF HAMMER COUNTERWEIGTHL 28 + 7 + 7 + 3 = 45 DAYS

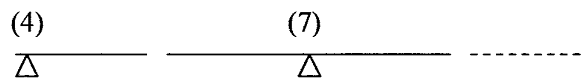
Age seg 1 — Time for construction — Time for cast joint

7 for the construction of the hammer of P11

WE CAN CONSIDER $t_k = (38 + 45) / 2$

REFERRING TO THE STATIC SCHEME 2

- CAST JOINT FOR THE LAST SPAN (HALF HAMMER OF 4 SEGMENTS AND HAMMER OF 7 SEGMENTS)



$\xi_2(t, 36, 31)$ $\xi_1(t, 45, 31)$

- EVALUATION OF ξ

ξ VALUES SHOULD BE EVALUATED AT THE INTRODUCTION TIME FOR DELAYED RESTRAINT FOLLOWING THE ONE UNDER ANALYSIS, ACCORDING TO THE SCHEME