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A P P U N T I

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FOUNDATIONS

DESCRIPTION

Bridge foundations are characterised by some specific aspects.

- Very high loads with respect to buildings, depending on piers dimensions.
- Due to road or railway alignment – bridge is forced to respect a planimetry and an altimetry –, we are forced to realize foundations in uneasy positions and bad ground with consequent problems of foundation: river, sea, very compressible soils, slopes, landslides, etc.
  This implies the use of special foundations, often deep.
- We can realize a footing directly on ground in some simple cases, e.g. short span, flat region and good soil.
  Footing is realized over natural soil or on strengthened, i.e. soil with injected concrete which locally increases soil density. We may also adopt jet grouting in order to make a crown excavation and realize a foundation at bigger depth.

CRITERIA FOR PRELIMINARY DIMENSIONING

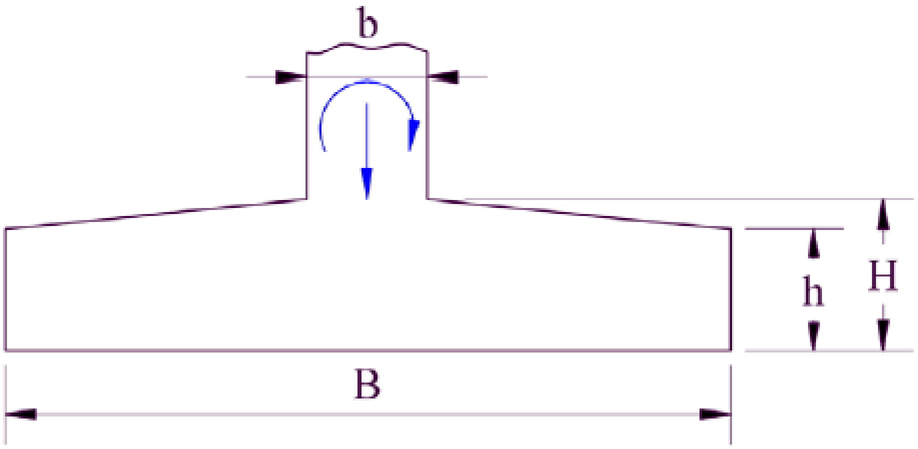
Given the pier dimension b, the larger dimension of the plinth B should be not too much bigger – in this way, the cantilever part is not important.

$$H \leq \frac{B-b}{2} \leq 2H$$

Moreover, the variation of its depth from the value h to H should respect a certain ratio, in order to avoid problems related to punching and also for construction reasons.

$$\frac{h}{H} \cong 0{,}6 \div 0{,}7$$

The H value is governed by the necessity for bending and punching and it is determined by B value. Then, we derive the h value.



Thus, having designed the B value from geotechnical criteria, we know the shape of the foundation.

The static scheme is the one of a slab loaded from the bottom, where the restraint is represented by the pier and the ground reaction on the foundation is the acting load.

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Footing reinforcement 2

Footing reinforcement and mould

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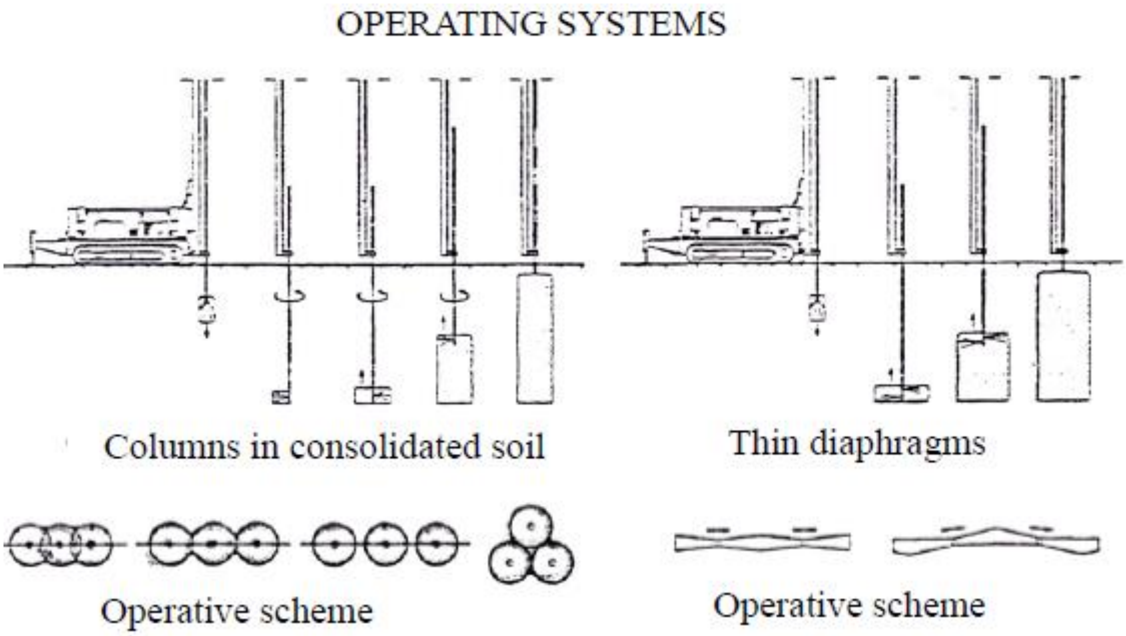
Concreting of the footing

Foundation area aerial view

This is typically a massive casting, as a foundation can have a radius of 60 m and thickness of 3 m. This implies logistic problems due to the use of a huge number of beton cars and also a structural one, related to the necessity of reducing the heat produced during the hardening, in order to limit thermal stresses inside the concrete.



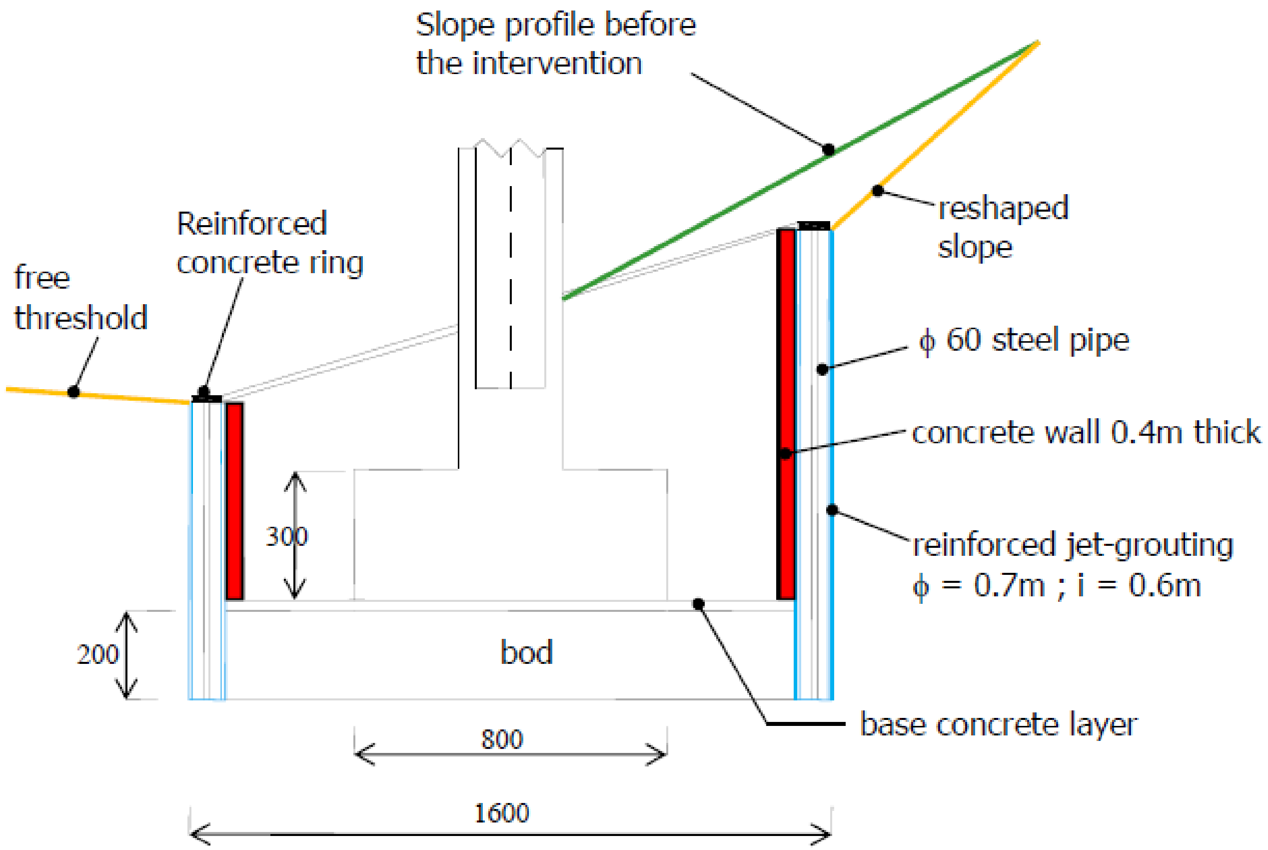
The field of application of jet grouting is represented by non-cohesive and cohesive soils with a shear strength that allows them to be disaggregated by the grouting.
In case of non-cohesive soil, we obtain a sort of concrete, which is not the classical one as any selection of geometry of aggregates has been performed.
In case of cohesive soil, the only risk is not to obtain big columns or columns with the same diameter along the depth due to cohesion, which has to be destroyed by injection.
The water bed in hydrostatic conditions does not affect the results, except it is moving, as concrete might be carried by water flow. By consequence, in this case, we should introduce an accelerating add-mixture.

Operating parameters are grouped in a table, which gives the kind of fluid and the range of pressure, nozzle, extraction speed of the tube – it is slower in multi-component systems, since we want to realize a bigger diameter –, rotation speed, ratio between concrete amount and water amount – it is a very fluid mixture, with a similar ratio in all situations – and flow as function of the system.

TABLE I – **OPERATING PARAMETERS** – Typical values

System	Fluid	Pressure *[bar]*	Nozzle (n° and ϕ) *[mm]*	Extraction speed *[cm/min]*	Rotation speed *[rpm]*	Concrete/Water	Flow *[l/min]*
MONO-FLUID	mortar	400-550	1-2 x 2-5	15-100	5-15	1.0-1.5	70-600
TWO-FLUIDS	mortar	400-500	1-2 x 2-5	10-30	4-8	1.0-1.5	70-600
	air	10-12	=	10-30	=	=	4000-10000
THREE-FLUIDS	mortar	50-100	1-2 x 4-5	6-15	4-8	1.2-1.5	80-200
	air	10-12	=	6-15	=	=	4000-10000
	water	400-500	1-2 x 2-3	6-15	=	=	40-100

At the end, the available strength of the column will depend mainly on soil characteristics.



Looking at the transverse section, we can notice that the slope has been reshaped and, due to the slope, the crown has two levels. Moreover, jet grouting is reinforced with a steel pipe inside, inserted in a second perforation drilled after the first one. Proceeding with excavation, a concrete wall has been built in order to grant protection from groundwater.
When excavation reached the wanted depth, they started to realize the plinth and then the column.

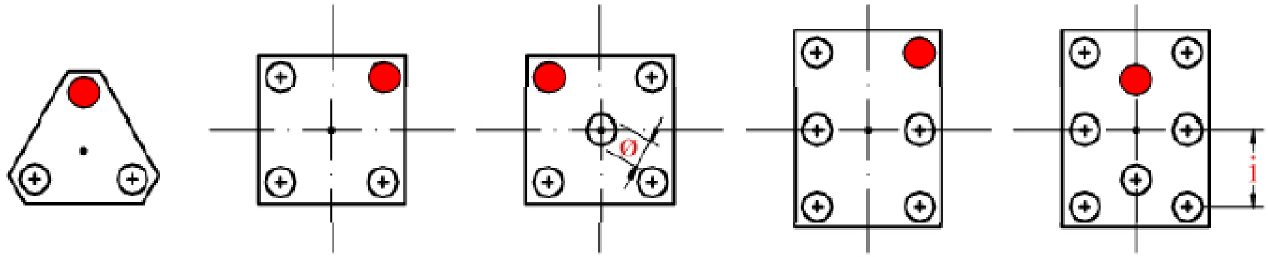
<u>Example of application</u>

A circular pier footing is placed inside a circular jet grouting crown, made with two layers of columns. One of them is reinforced with steel pipes.

FOOTING ON PILES

It is a very common solution.

Piles are realized with relatively high diameter (variable between 800 mm and 2000 mm) and the they should be arranged so that the centroid of the pile arrangement is coincident with the position of the resultant of permanent actions to limit long term settlements. The following figure shows some common disposition of piles under a footing respecting that condition.



Actually, in presence of closed spaced piles, there are interaction effects between them that are not negligible and cause a reduction in bearing capacity. According to this aspect, we should increase the distance between the piles but, doing this, we would obtain very large foundations. By consequence, we have to find a compromise between these issues.

In case of piles in clay, with an interaxis of (2 ÷ 3)ϕ – usual case –, the foundation behaves as a compact block and collapse realizes for the overall foundation, with load bearing capacity depending on its external perimeter.
In the opposite situation, i.e. interaxis bigger than 3ϕ, we can have collapse of a single pile.
We can define an efficiency factor γ for the piles, which is the ratio between the actual pile system collapse load and the sum of the single pile collapse load.

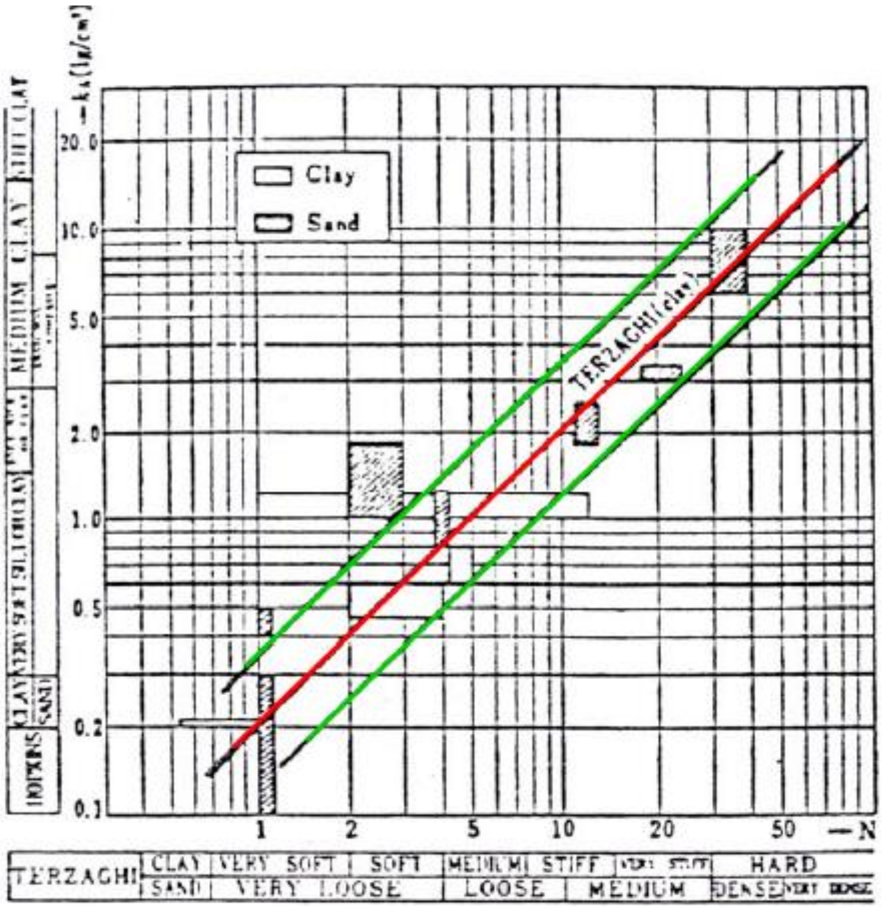
$$\gamma = \frac{pile\ system\ collapse\ load}{\sum single\ pile\ collapse\ load} = \begin{cases} 0{,}6 \div 0{,}8\ for\ i = (2 \div 4)\emptyset \\ 1\ for\ i = 8\emptyset\ (not\ common\ situation) \end{cases}$$

Indeed, as there is a reduction of the bearing capacity of the single pile due to interaction, the sum of the single pile collapse load will be smaller than the actual one of the pile system and this phenomenon is represented by the ratio γ.
The typical situation corresponds to an interaxis i equal to 3ϕ and we can assume the following value.

$$\gamma \cong \frac{2}{3}$$

It means that only 2/3 of the bearing capacity of the pile is available inside the foundation.
If there is an interaction between the plinth and the ground, as there is an additional bearing capacity of the plinth in contact with the ground, the global bearing capacity of the block made with the plinth, ground and piles may be evaluated by using a factor N_c, depending on the geometry of the foundation – plinth dimensions B and L – and the piles length D.

$$N_c = 5{,}14\left(1 + 0{,}2\frac{B}{L}\right)\left(1 + 0{,}2\frac{D}{12B}\right), \quad \left(1 + 0{,}2\frac{D}{12B}\right) \leq 1{,}5$$



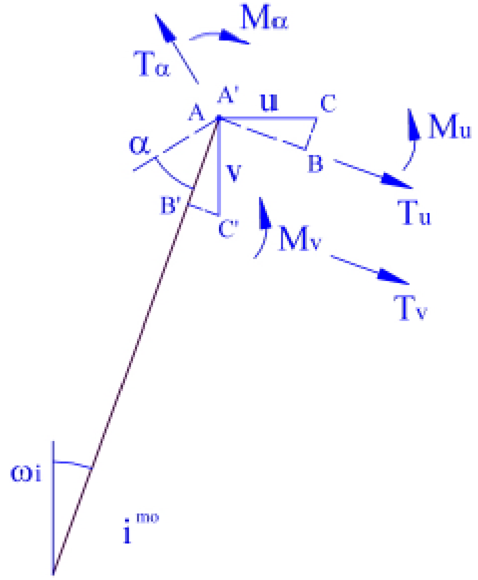
By using this diagram, we can estimate the Winkler modulus and then solve the problem of beam on an elastic soil, getting the internal actions inside the pile combined with the vertical load.

As regards the pile width, in the model of beam on an elastic soil B was the width of the foundation but this is a different situation because the pile is completely immersed inside the ground and there is a trust effect, i.e. when the pile moves, it mobilizes not only the soil in direct contact with the pile but also the adjacent portion. By consequence, we can increase the width of the equivalent beam, which varies in a range.

$$\emptyset \leq B \leq 1{,}5\emptyset$$

Moreover, first soil layers from the ground level are very disturbed by construction –soil is excavated at the top in order to discover of reinforcement and then recover – so they can not bear horizontal actions. By consequence, we should reduce the Winkler modulus or disregard the first pile diameter of depth as resisting for the horizontal actions and we start at a bigger depth.

In the case of footing on piles the distributions of the actions coming from the foundation on the single piles should be evaluated according to the relative stiffness of the single piles and their inclination.



From the geometry, we can derive the displacement of each pile in the pile direction and in the normal direction from the displacement of the foundation.

$$\overline{AB} = u\cos\omega_i$$

$$\overline{A'B'} = v\cos\omega_i$$

$$\overline{BC} = u\sin\omega_i$$

$$\overline{B'C'} = v\sin\omega_i$$

By applying the stiffness parameters, we derive the forces at pile head.

- Vertical force at pile head, depending on the vertical displacement of the pile, which includes the effect of rotation at the top of the pile – if possible – and
$$P_i = k_{V,i}(u\sin\omega_i + v\cos\omega_i + \alpha x_i\cos\omega_i)$$
- Horizontal force at pile head.
$$H_i = k_{H,u,i}(u\cos\omega_i + v\sin\omega_i + \alpha x_i\sin\omega_i) - k_{H,\alpha,i}\alpha$$
- Bending moment at pile head.
$$M_i = k_{H,\alpha,i}(u\cos\omega_i + v\sin\omega_i + \alpha x_i\sin\omega_i) - k_{M,\alpha,i}\alpha$$

These forces are projected with respect to the general reference system xz of the foundation.

$$Z_i = P_i\cos\omega_i + H_i\sin\omega_i$$

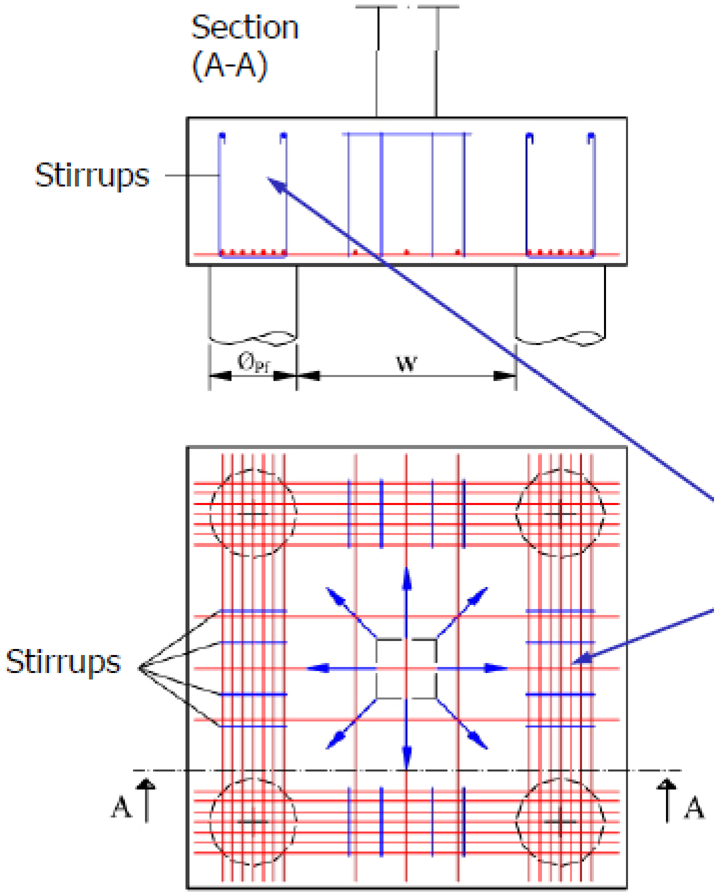
$$X_i = -P_i\sin\omega_i + H_i\cos\omega_i$$

The forces are written for all piles and we consider then the global equilibrium conditions for all the foundation.

$$H + \sum X_i = 0$$

$$V + \sum Z_i = 0$$

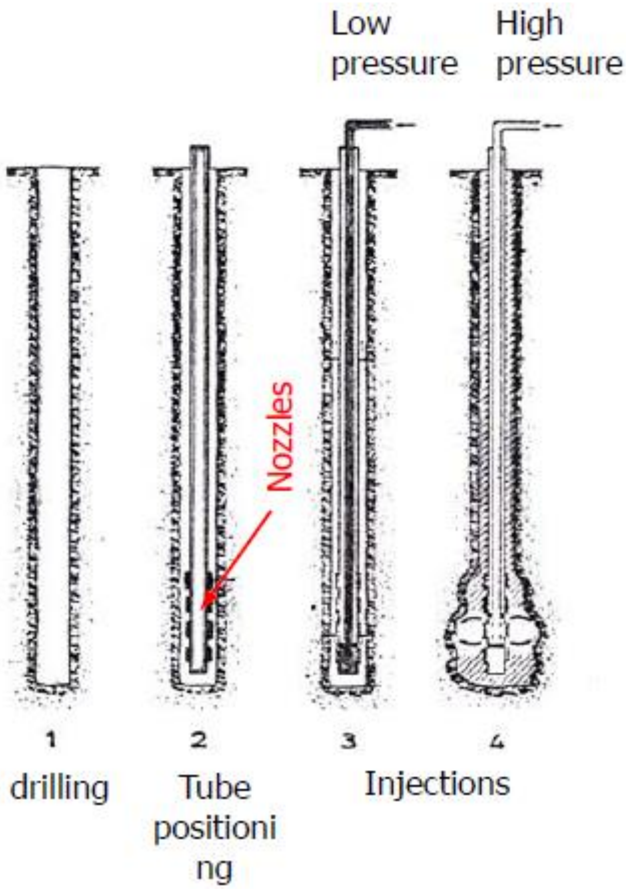
When the load goes directly to the piles, it is carried by the inclined struts mechanism. In the other case, the load arrives at the deep beam between two adjacent piles, but the vertical component should be carried by further reinforcement oriented from one pile to the other. Moreover, as struts arrive at the bottom, we have to introduce suspension reinforcement with stirrups.

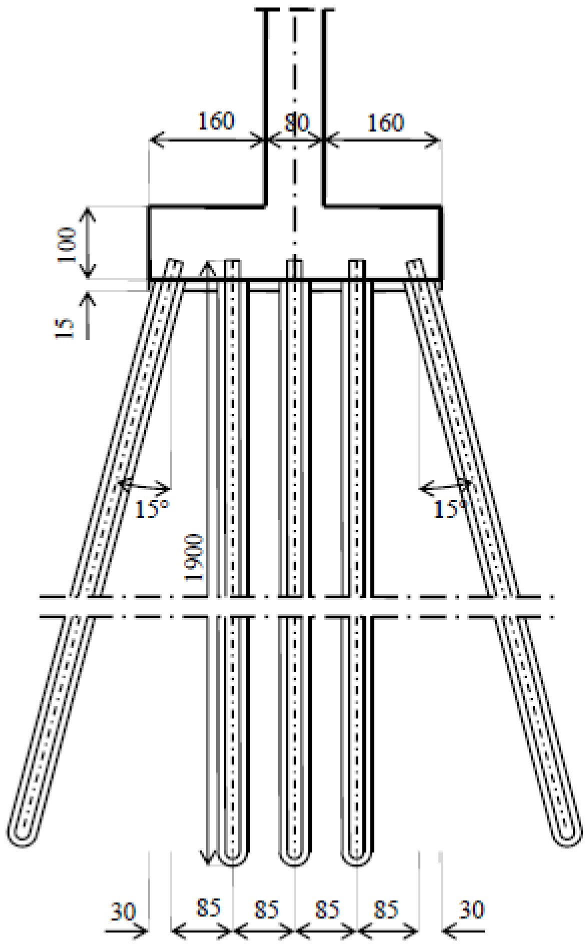


FOUNDATION ON MICROPILES

This technique uses small diameter piles ($100 \leq \phi \leq 250$ mm), which can be drilled with various inclinations, in every ground. Ground is perforated by rotation or roto-percussion continuous with a circular hollow head hammer and this technique allows the execution of piles also in rocks.
No normal piles are used, since their realization requires big machines, space and huge quantity of bentonite, in case of bored piles – it maintains the geometry of the hole during excavation. In other words, their realization would be complicated.
Moreover, micropiles are used in case of improvement of existing structures because their realization does not requires huge machinery and it interacts in a small way with surrounding elements.

During the excavation, the hole may be lined with a small tube, depending on the nature of the ground.
Then, reinforcement is sunk. Reinforcement is a steel profile or metallic tube with holes.
The holes are used to realize the injection, firstly at low pressure – 5 bars – with a mixture of water, concrete and sand, filling the tube and the interspace between the tube and the hole up to ground level. After having finished it, we perform a second grouting under pressure – 30 ÷ 40 bars. The pressure gives rise an increase of volume of injection, which means a deformation of the surrounding ground, improving the bearing capacity of the pile. The second injection is performed through some nozzles closed in the first phase by means of elastic caps calibrated to open only in case of high pressure and nozzles are distributed along the tube – about one nozzle per 1 m of length, for a total length of 4 ÷ 5 m.





Example: deep foundation

Micropiles are also used, instead of jet grouting, to realize deep foundations because they can realize the crown working like diaphragm during excavation of the ground. In such case, during excavation, every 2 ÷ 3 m – it depends by the distance between micropiles and ground characteristics – we apply internal hoopings made with steel curved beams in addition to the upper reinforced concrete ring ("cordolo") to stiffen the foundation. By adding spritz beton, we realize a wall against the piles and we work internally to it.

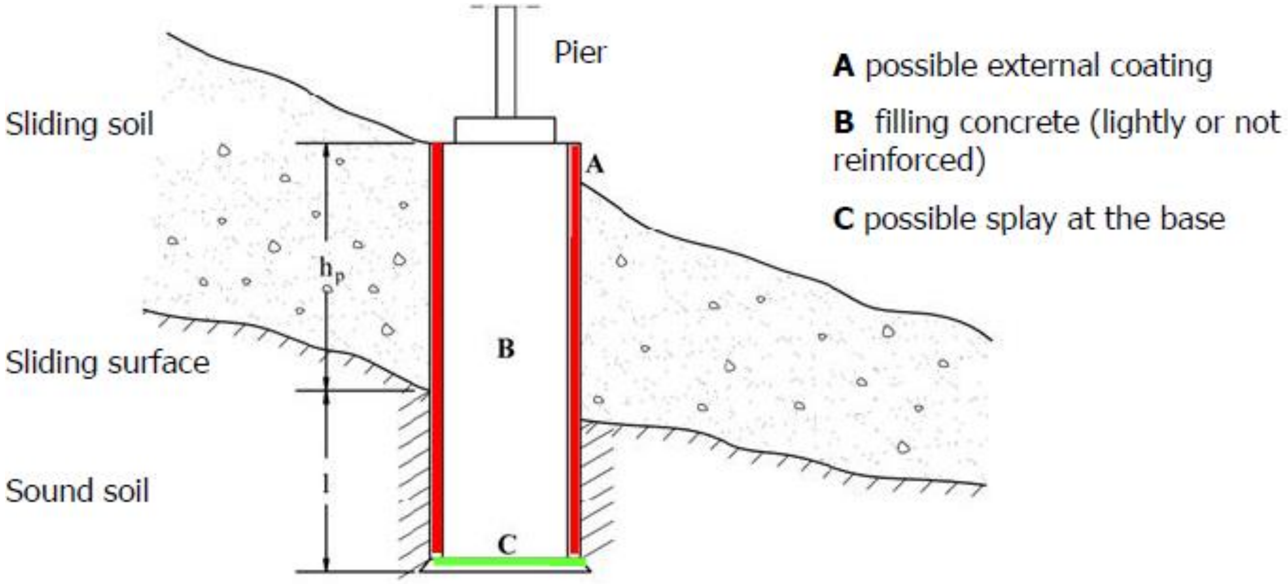
SHAFT FOUNDATIONS

Shaft foundations are an expensive solution used for bridge foundation in presence of large vertical and horizontal actions, due to different reasons.

- They allow to reach ground layers with better mechanic behaviour.
- They reduce the plan dimension of the foundation.
- They stabilize the superficial landslips – important aspect.
- They offer shelter to the piers from landslides.
- They resist to static and dynamic actions due to landslides.

A typical solution is solid body shafts, adopted in case of sliding soil.

Solid body shafts are elements with high stiffness, fully restrained in the sound soil and they work as a deep cantilever able to bear horizontal actions from landslides.
The pier is put on the top of the foundation and may be present a coating A of the excavation. The space is filled with poor concrete, so that the load is directly transferred to level C, where there is good bearing capacity for vertical actions.



They generally have circular sections, with diameter between 6 m and 20 m. the minimum diameter of 6 m is related to put the digging machine and give it some free space.
They can also have elliptic section to enhance stiffness in one direction and very strong thrust is required for the landslide.

They almost always need a pre-coating, at least in the crossing of the sliding layer, which can be realized with reinforced concrete – for high thrusts – or with steel hooping and spritz-beton.

In case of foundation going under water bed level, we always work with submerged suction pumps at excavation bottom and we realize the rings progressively from the bottom with *underpinning technique*.
In the underpinning technique, we realize first excavation of about 1,5 m and we place a coating made with reinforced concrete ring. The ring remains suspended in its position during the advancement of excavation – after the hardening of the element – because earth pressure

Shaft R.C. ring

Hooping
before
spritzbeton

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Underpinning detail

Digging at -23m

Cleaning up after emptying the flooded pit

Shaft bottom

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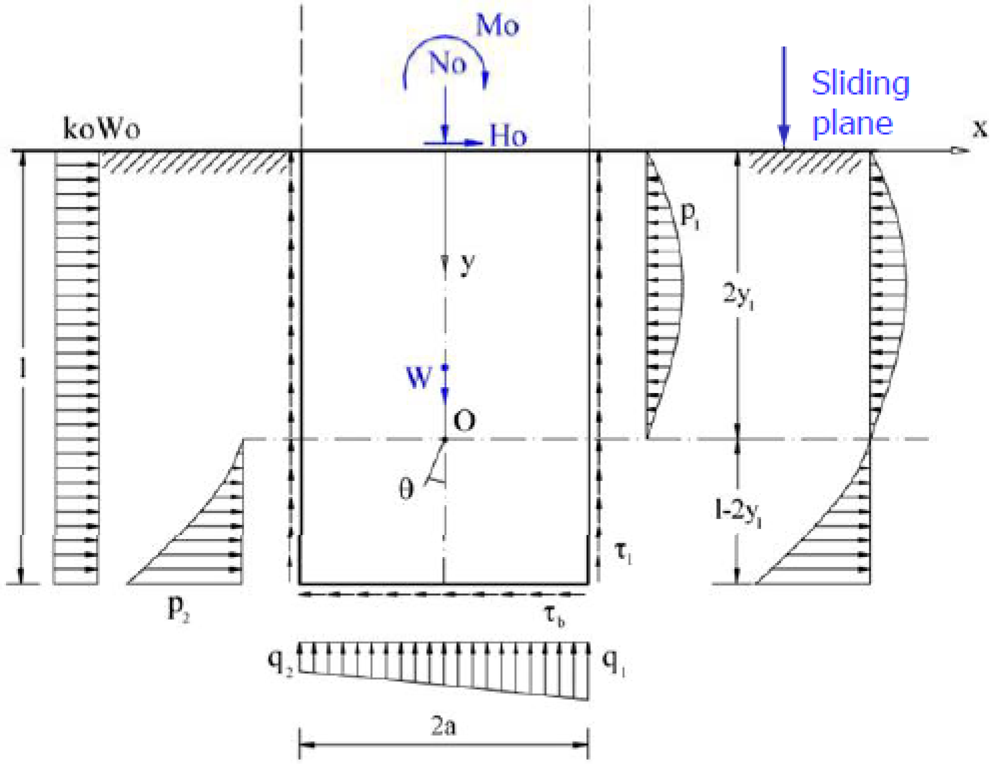


Of course, there is no resistance in the level immersed in the sliding soil, as it develops only in the sound soil.
Focusing on the part of the shaft in sound soil, it is subjected at the top to actions N_0, H_0, M_0 coming from the deck and the thrust of the landslide. In case of earthquake, there is the additional force K_0W_0, related to shaft self-weight.

Resistance is realized through mobilizing of pressures inside the soil. The cylinder in concrete in sound soil tries to develop some displacement, which mobilize reaction in the soil, in addition to classic bearing capacity of the soil for vertical load.

Assuming that the centre of rotation of the foundation is point P, i.e. the shaft rotates around it, we can identify several actions.

- Horizontal stresses p on terrain, equal to p_1 at the top right and p_2 at the bottom left – rotation is assumed to be clockwise.
- Vertical stresses q on terrain, related to bearing capacity for vertical load.
- Tangential stresses τ_l on the lateral surface of shaft.
- Tangential stresses τ_b on the base of shaft.

Generally, the evaluation of pressures p, q, τ_l and τ_b is very complex and, for the sake of simplicity, they are disregarded – even if they are important.

In case of non-cohesive soils, we can refer to the Jkeara method, which is based on the following hypotheses.

- The shaft is a rigid body.
- Friction between the surface of the shaft and ground is neglected.

$$\tau_l = \tau_b = 0$$

$$p_2 \leq \frac{1}{\eta} \gamma_T K_p y_2$$

In the condition, we assume that

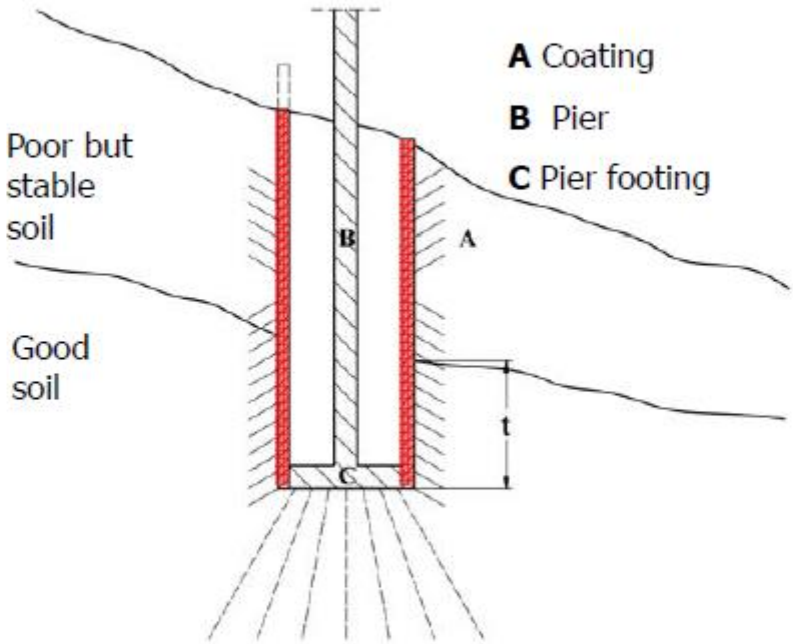
$$\eta = 2{,}5 \div 3$$

On the shaft side, typically shear and compressive-bending verifications must be satisfied.
Moreover, maximum stresses on the ground q_1 must be compatible with terrain compressive strength at depth l.

Practically, the design of the shaft consists to determine the geometry of the element by attempts, taking into account that we should realize full restraint static scheme in the sound soil. In order to do this, the embedment length should be at least equal to one diameter.
Thus, the unknown is the diameter and it is computed by attempts, by assuming a value, computing actions and performing bearing capacity verifications.

Another case is hollow core shafts.

In this case, instead to infill the excavation with poor concrete, the plinth is directly put inside the shafts and the free space is filled with dry soil.



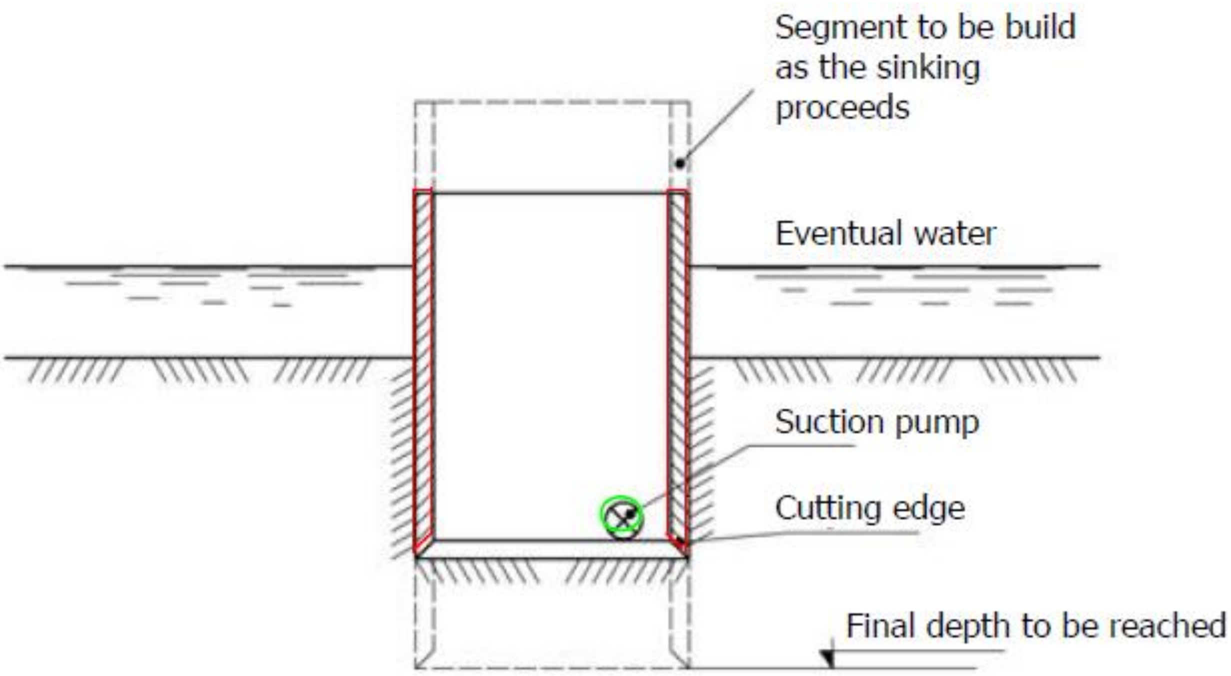
It is a good solution to pass through bad or low bearing capacity superficial soil layers, which are not interested by landslide risk and/or are difficult to pass through with piles or diaphragms. Moreover, this system provides the advantage that mass is smaller, with a better behaviour with respect to seismic actions.
The shaft becomes a mere operative device to realize the pier and its footing and to protect it from rock fall.
It can be extended over ground level to provide a protective cap

The shaft has to resist only soil against active ground thrust, so the depth t is designed only to reach a level with good bearing capacity and to allow the rigid body equilibrium of the shaft.

CAISSON FOUNDATIONS

Caisson foundations were a typical solution in the past, now are not used due to high risk for workers and are replaced by big diameter piles or walls.

They are precast deep foundations made of vertical segments assembled in situ or cast in situ structures with vertical progression, generally placed in correspondence of river crossings.
The bottom side, shaped with cutting edge, sinks thanks to self-weight, as far as the excavation inside the caisson progresses, and caisson bottom is kept dry from incoming water by means of submerged suction pumps. While it is going down, we add new elements from the top.
Irregular soil characteristics and erratic boulders cause an undesired inclination of the box section that is very difficult to be restored.



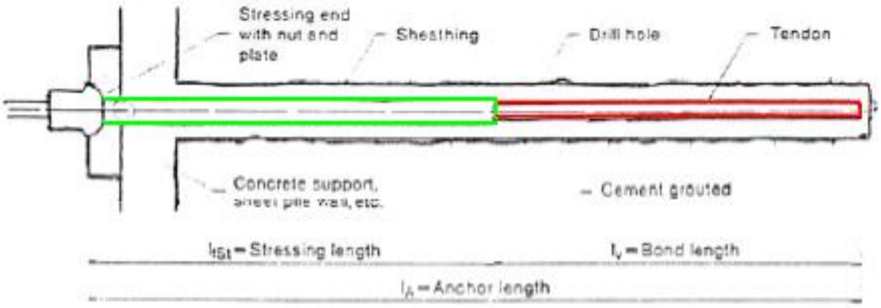
An important problem is that, in presence of water around the foundation, sometimes hydrostatic head may be too high and the ground too porous, so that bottom pumps can not keep the excavation dry or even lead to the failure of the ground due to the pressure exerted by the water – it is a terrible failure because, in an instant, soil is transformed into a mixture of water and soil and the foundation disappears.
In these cases, the unique solution is to increase pressure from inside and the best way to realize it is to pump air inside the caisson. Now, caisson structure is more complex as there are an excavation room A – with a pressure of 2 bars –, a channel B and a decompression room C. This scheme is necessary because people were affected by embolism and diseases due to rapid decompression and the difficulty to work in these conditions. So, worker are subjected to gradual decompression at shift end.

TIED FOUNDATIONS

They are not foundations, but they help in the realization of the foundation.

Ties are prestressed elastic restraints used for walls or foundations.

Fig. 35 Scheme of a grouted anchor

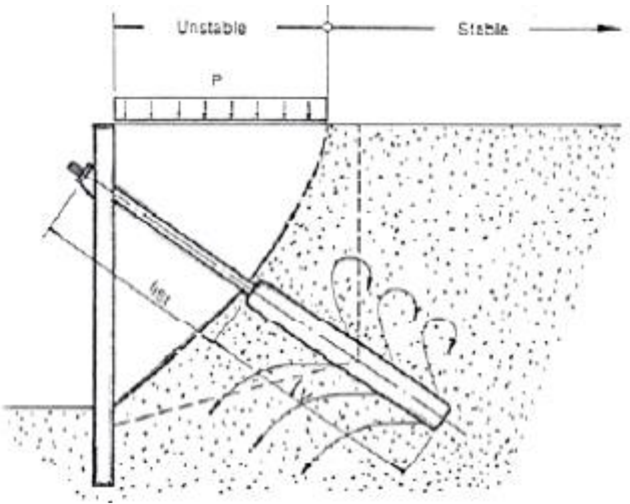


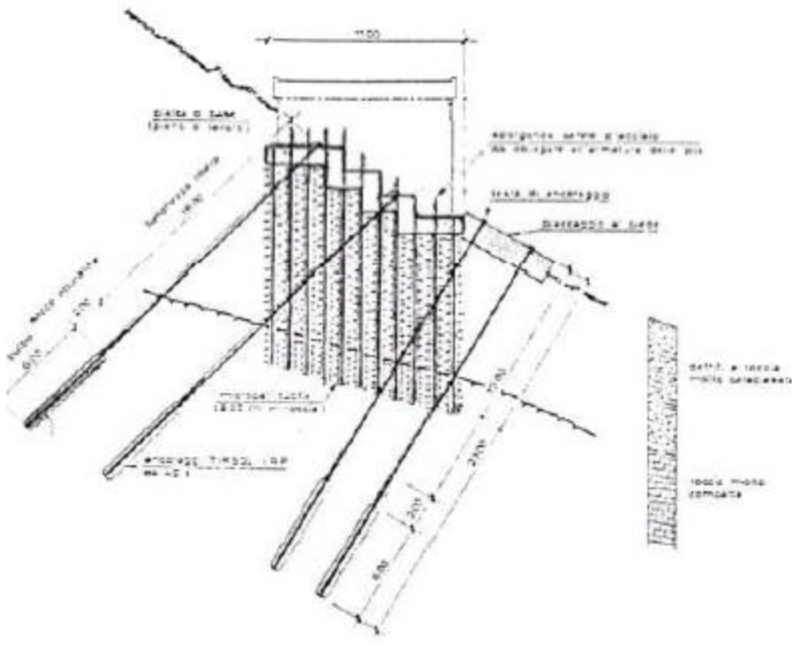
We follow a certain execution procedure for their realization.

- Perforation with roto-percussion having diameter between 8 cm and 15 cm.
- Insertion inside the perforation of prestressing tendon or bar, which is partially sheathed, i.e. partially protected by sheath.
- Realization of a sealing cap at the beginning of the bulb, through introduction of a mixture of cement and water in pressure. This gives solidarization of the tendon with the surrounding soil in the non sheathed part.
- Realization of mortar grouting between the soil and the bars around the sheathing and the anchoring zone.
- Stressing of the tendon, as it is linked to the soil through the anchoring zone and there is a free zone. By applying the force, we realize an elastic restraint to the wall.
- Second protection grouting with low pressure inside the sheath, in order to protect tendon from corrosion.

How should be realized the anchorage?

Anchorage should fall outside the region subjected to movement. Indeed, given a wall, we can estimate the sliding surface and the anchoring zone should fall outside the mass subjected to movement, otherwise there would not be equilibrium.





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- At U.L.S. conditions, we have to reach a stable dissipative mechanism. Indeed, we need dissipation, which means plasticisation and to reach a final stable situation, with dissipation only in the pier.
  The unique internal action able to give dissipation is bending, as concrete structures can show large rotation in the plastic field. By consequence, we adopt a bending mechanism, in which we have to exclude that shear failure will precede bending failure. Thus, design is performed in order to cover shear failure and to avoid its intervene and the interruption of the dissipative mechanism.
- As dissipation is occurring only in the piers, deck, bearings, abutments, foundations and ground show an elastic behaviour and no dissipation is allowed there.
  In fact, we are adopting the concept of Capacity Design – hierarchy of resistances -, where some elements plasticize and guarantee dissipation, whereas other elements remain in the elastic field.
- During earthquake, some cinematism is necessary but we should avoid hammering and fall from bearings.

These criteria have to be transferred to design, through the introduction of the so-called importance factor γ_I, corresponding to a variation of the reference period T_0 and, by consequence, a variation of the seismic action.

$$\gamma_I = \begin{cases} 1 & Ordinary\ bridge \\ 1{,}3 & Strategic\ bridge \end{cases}$$

DEFINITION OF SEISMIC ACTION

Seismic design is based on different ground types.

Each ground type is characterised by the average velocity of propagation of shear waves – important parameter for bridges – in the first 30 m of depth $V_{s,30}$. In case of ground made of several layers, the velocity is computed from the velocity $V_{s,i}$ in each layer and its depth h_i.

$$V_{s,30} = \frac{30}{\sum_{i=1}^{N} \frac{h_i}{V_{s,i}}}$$

Some type are also characterised by the number of standard impulses per one foot N_{SPT} or the undrained shear strength c_U.

- Ground type A
  It is rock or rock-like geological formation, including at most 5 m of weaker material at the surface. In this case, we do not need specific tests.
  $$V_{s,30} > 800\ m/s$$
- Ground type B
  It includes deposits of very dense sand, gravel or very stiff clay, at least several tens of meters in thickness, characterised by a gradual increase of mechanical properties with depth.
  $$V_{s,30} = 360 \div 800\ m/s$$
  $$N_{SPT} > 50$$
  $$c_U > 250\ kPa$$

REPRESENTATION OF SEISMIC ACTION

When we evaluate the seismic action, how can we represent it?
We can follow two different approaches.

RESPONSE SPECTRUM

A typical case is the spectrum of elastic response in terms of acceleration.

This is the response in terms of maximum acceleration of a SDOF – Single Degree Of Freedom – element mass, having an elastic behaviour.
A SDOF element mass is a mass, which is connected to ground by means of an column – elastic column, in this case.

The horizontal component of seismic action is defined by the value

$$a_g S$$

It represents the PGA of the system and, depending on the period of vibration T of the SDOF mass, the spectrum of elastic response is represented by four different laws.

$0 \le T < T_B$	$S_e(T) = a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot (\eta \cdot 2{,}5 - 1)\right)$
$T_B \le T < T_C$	$S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5$
$T_C \le T < T_D$	$S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \left(\frac{T_C}{T} \cdot\right)$
$T_D \le T$	$S_e(T) = a_g \cdot S \cdot \eta \cdot 2{,}5 \cdot \left(\frac{T_C T_D}{T^2}\right)$

We can notice that, in the second range of periods, the maximum acceleration of the system is constant.

The spectrum of elastic response depends on the ground type, since it affects the scale of the graph and it translates the curve, for the same value of PGA.

$0 \le T < T_B$ $\quad$ $S_{ve}(T) = 0{,}9\, a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot (\eta \cdot 3{,}0 - 1)\right)$

$T_B \le T < T_C$ $\quad$ $S_{ve}(T) = 0{,}9\, a_g \cdot S \cdot \eta \cdot 3{,}0$

$T_C \le T < T_D$ $\quad$ $S_{ve}(T) = 0{,}9\, a_g \cdot S \cdot \eta \cdot 3{,}0 \left(\frac{T_C}{T} \cdot\right)$

$T_D \le T$ $\quad$ $S_{ve}(T) = 0{,}9\, a_g \cdot S \cdot \eta \cdot 3{,}0 \cdot \left(\frac{T_C T_D}{T^2}\right)$

Ground category	S	T_B	T_C	T_D
A, B, C, D, E	1,0	0,05	0,15	1,0

If we wanted to describe the spectrum of elastic response in terms of displacements, the equations would be the same, multiplied by the value

$$\frac{T}{2\pi}$$

Where T is the generic period of vibration.
This is not a common use.

Spectrum of elastic response in displacement

$S_{De}(T) = S_e(T)\left(\frac{T}{2\pi}\right)^2$ $\quad$ $T < T_E$

Ground category	T_E	T_F
A	4,5	10,0
B	5,0	10,0
C,D,E	6,0	10,0

$S_{De}(T) = 0{,}025\, a_g\, S\, T_C\, T_D\, (2{,}5\,\eta + (1 - 2{,}5\eta)\,(T - T_E)/(T_F - T_E))$ $\quad$ $T_E < T < T_F$

$S_{De}(T) = d_g$ $\quad$ $T > T_F$

d_g = maximum displacement of ground

In conclusion, we can not forget that the seismic action will arrive in the structure in different time.

What is the effect?

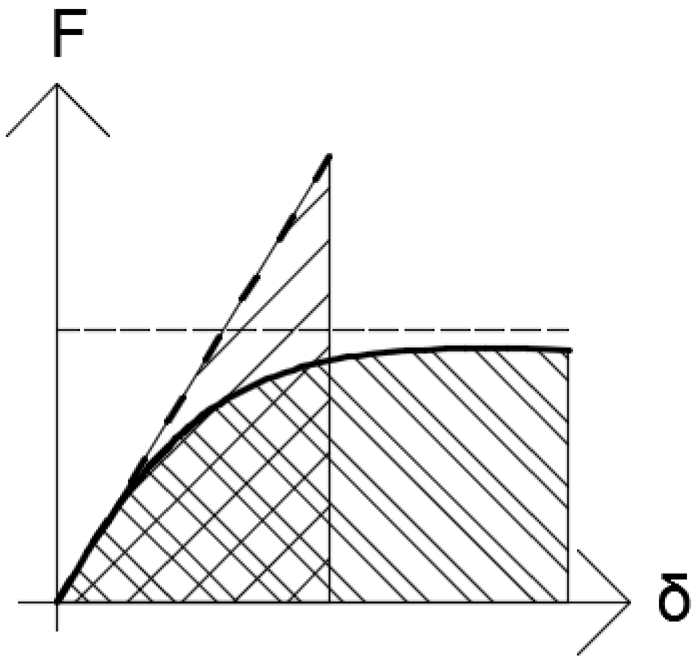
The effect of the delay is an alteration in the input of the seismic action for the structure and an increase of displacement in the analysis.
On the other side, with a system of low-cost sensors, we may realize a grid of 500 m × 500 m. indeed, if the velocity of propagation is 300 m/s, the wave will take 1 minute to reach a distance of about 18 km.
One minute is a significant time and this interval of time can be used for the warning because, in real time, we may identify the epicentre, the direction of propagation and the attenuation of the waves and, from this, we can make a prediction of the entity and of the risk for an

The direct application of the spectrum of elastic response in the computation of internal actions in the structures brings huge forces, since we are asking a structure to assume elastic behaviour during the seismic action. Of course, this kind of request is very expensive or even impossible to realize.

For this reason, we introduce the <u>design spectrum for U.L.S.</u>, which works on the following strategy: *we do not design for the elastic behaviour of the structures, but we design for the limit damage*.

Assuming that the structure is able to dissipate inelastically the energy transmitted by the seismic action, at certain point the elastic forces will stop to increase because the structure enters in he plastic field.
Practically, if we evaluate the force and the relative displacement in a point of the structure

- With the spectrum of elastic response, the behaviour is described by a linear law.
- With the design spectrum for U.L.S., we assume that, at certain level, plasticization occurs in some chosen points of the structure and the curve tends to a constant value. In this way, we put a limit in forces and we reduce the force acting on the structure.



$$q = 1$$

In this way, we accept to use the elastic response spectrum in the design in vertical direction.

Generally, the vertical component is less dangerous for the structures, since they have been designed in order to resist to vertical actions. Moreover, during the application of the seismic action, not all the vertical loads are present.
In case of bridges, unless long span bridges – bigger than 60 ÷ 70 m –, there are no problems with the vertical component of the seismic action because, in the combination of the seismic action with the other actions, generally traffic action is assumed to be null – except bridges next to a toll station.

DIRECT USE OF ACCELEROGRAMS

This was considered the most advanced technique in past, as it required a lot of calculation but nowadays it is easy to use, as computers have been improved.

Accelerograms may be of two types.

- Natural accelerograms, i.e. registrations of earthquakes.
- Artificial accelerograms, which are numerically-generated.

It is not so much ensured to use only natural accelerograms, since they depend on local conditions, energy locally cumulated and other factors which make these representations to be similar but not scaled in time.

Accelerograms are generally applied in three directions and they have to respect some conditions.

- Accelerograms are described for a period of time, including the duration of the seismic action. Due to the transitory at the beginning and the end of the seismic action – transitory, peak, transitory –, we need to have a pseudo-stationary part, not shorter than 10 seconds.
- Minimum number of groups is 3, in order to describe the different seismic actions.
- It is required coherence with elastic spectrum, i.e. the average coordinate of the elastic response evaluated with a damping ratio of 5% should be bigger than 90% of the corresponding elastic spectrum in an interval of periods of vibrations.
$$\bar{A} > 0{,}9\overline{A_{ref}}, \qquad 0{,}2T_1 \leq T \leq 2T_1$$
The term T_1 is the fundamental period in elastic field.
In this way, we fix the maximum acceleration in the horizontal plane, so that the behaviour of the element is at most 10% different from the expected elastic response spectrum.

Once we have established the accelerograms, an important issue is the <u>spatial variability of displacements</u>.
We may have variability of displacement along the bridge due to different situations.

- Inhomogeneity of the soil under the bridge, which is a more recurrent problem than in buildings.
- Geometric discontinuities of the ground, e.g. slope, river bed, variation of type of soil, etc.
- Differences in the local response of the soil.

Finally, we have to apply a superposition of dynamic effects of spectrum and pseudostatic effects, related to relative displacement between two points in the structure.

In this analysis, we apply simultaneously the three components of seismic action in order to have the global behaviour – we can not apply the superposition of effects – and we consider the maximum effects as average value of the worst effects due to each triplet of accelerograms, i.e. we consider the worst internal action in each section, with the mean value due to oscillation in time.
This should not be used as a design tool as there is no codification and we have calibration uncertainties.

The seismic action has then to be combined with the other actions in ULS.

- In resistance analysis, seismic action is multiplied by an importance factor γ_I and combined with the characteristic value of permanent loads and prestressing.
$$\gamma_I E + G_k + P_k$$
- An other important verification is compatibility of displacements, since we want to avoid the failure of bearings and joints or the impact between components.
The relative combination has also the contribution of thermal effect, with 40% of maximum elongation due to the difference of temperature with respect to the day of construction. Indeed, temperature plays an important role.
$$\gamma_I E + G_k + P_k + \psi_{0,\Delta T}\Delta T, \qquad \psi_{0,\Delta T} = 0{,}4$$

STRUCTURAL FACTOR

The value of the structural factor q is available in a table described in the codes, with reference to the concept of limited ductility – no presence of specific reinforcement to provide ductility – or normal ductility.

- Reinforced concrete piers
Struts in bending may be vertical or inclined and, in case of ordinary ductility, the structural factor depends on the shear span ratio α_s, i.e. the distance L_s between the zero moment point and the maximum moment point with respect to the dimension h of the pier in the direction of flexure.
$$\alpha_s = \frac{L_s}{h}$$
If this ratio is smaller than 3, the pier degenerates into a square pier.
With reference to the ratio, we evaluate the corrective parameter λ, to be applied to the structural factor.
$$\lambda = \begin{cases} 1{,}0 & if\ \alpha_s \geq 3 \\ \sqrt{\dfrac{\alpha_s}{3}} & if\ 1{,}0 \leq \alpha_s < 3 \end{cases}$$
- Steel piers: the maximum value of the structural factor is 3,5.
- Abutments rigidly connected to the deck.
In this case, there is continuity between the deck and the abutments and the structural factor is at most 1,5, i.e. ductility is limited and this solution is not convenient.
- Arches.
Maximum structural factor is 2,0, since they work in compression and compression reduces ductility.

In irregular bridges, we can not account for full ductility, as it is not reached due to the over-reinforcement, and the structural coefficient q is switched to a value q_r.

$$q_r = \frac{q}{\tilde{r}}$$

By consequence, *in seismic regions, it is advised to put only the minimum reinforcement necessary for the acting bending moment*.

There also bridges having intrinsically very low dissipation capacity and less ductility.

- Arch bridges, since they work in compression.
- Trestle bridges, due to high compression in the trestle.
- Cable stayed bridges, due to compression in the deck.
- Very skew bridges, having skewness bigger than 15 ÷ 20°.
- Curved bridges.

For all these bridges, the structural coefficient has to be assumed as

$$q = 1$$

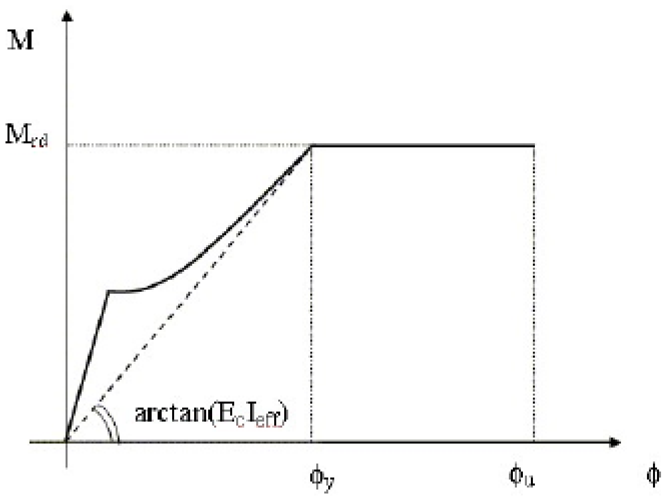
The reason is that these bridges do not have the condition of formation of plastic hinges and energy dissipation.

MODELLING FOR LINEAR ANALYSIS

How can we model the bridge for the linear analysis?

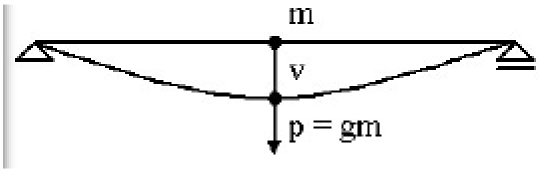
As regards <u>rigidity modelling</u>, generally, deck will be described as non cracked, as we design for not having cracks and dissipation in the deck.

On the other side, piers are described as cracked. For them, considering the bending moment-rotation diagram, we can assume that the flexural stiffness is not the one of the uncracked stage – rerpesented by the continuous line – but it corresponds to the secant stiffness. By consequence, stiffness is smaller and the advantages is that the period of vibration will be bigger and we will move in the queue of the design spectrum.

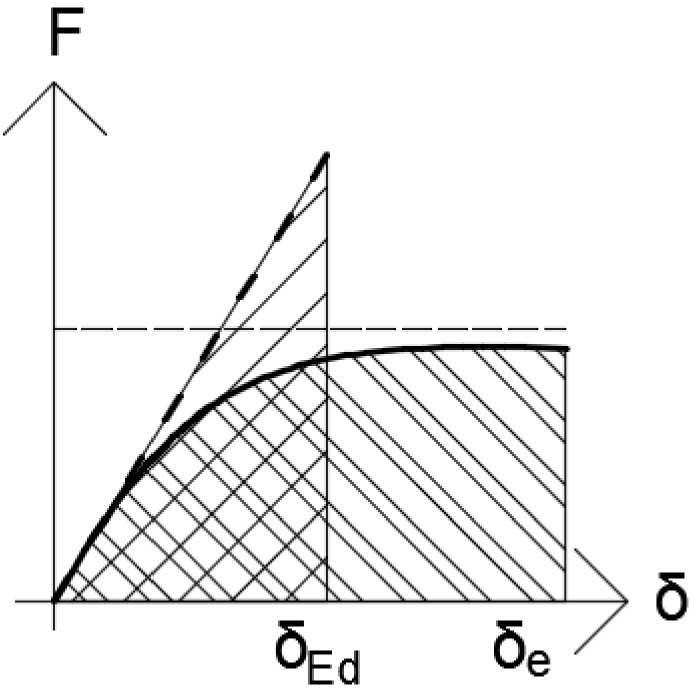


- Analysis in the longitudinal direction of straight bridges with continuous beam deck and effective mass of the piers smaller than 20% of deck's mass, i.e. there is a relevant mass at the top (rigid deck model).
- Analysis in the transverse direction of bridges respecting the first condition and which are longitudinally symmetric, i.e.
$$e_{max} < 0{,}05 l_{bridge}$$
In the equation, e_{max} is the distance between centroids of masses and stiffnesses of the piers in transverse direction.
In this situation, depth of piers is similar to depth of deck (flexible deck model).
- Girder bridges simply supported in longitudinal and transversal direction with effective mass of each pier smaller than 20% of mass carried by the deck, i.e. at least 80% of the mass is at the top (individual pier model).

In case of *longitudinal continuous bridge or girder bridge simply supported*, the simplified procedure applies a force in the piers equal to
$$F = MS_d(T_1)$$
M is total mass, given by the deck mass and mass of the upper half of all the piers, in the continuous bridge, or the deck mass on pier i and upper half mass of pier i, in the girder bridge.
$$M = \begin{cases} M_{deck} + \sum M_{i,upper\ half} \\ M_{deck} + M_{i,upper\ half} \end{cases}$$
S_d is the value of response spectrum in correspondence of the reference period T_1.
$$T_1 = 2\pi\sqrt{\frac{M}{K}}$$
K is the flexural rigidity of the system.
In case of *analysis in transverse direction of bridges which are longitudinally symmetric and continuous*, we apply Rayleigh's method. This requires the knowledge of the fundamental period, which may be derived by the principle of energy conservation.
If we consider a beam having a concentrated mass and, by consequence, subjected to a concentrated force in the point where the mass is, there will be equivalence between kinetic energy and potential energy during oscillation, in each point.



The kinetic energy depends on the velocity, which is the first derivative of displacement with respect to time.
$$E_k = \frac{1}{2}m\dot{v}^2(t)$$
Assuming sinusoidal oscillation, the kinetic energy will be
$$E_k = \frac{1}{2}mv_0^2(t)\omega^2 cos^2(\omega t)$$
The potential energy depends on the position, i.e. the displacement.
$$E_k = \frac{1}{2}pv(t) = \frac{1}{2}pv_0 sin(\omega t), \qquad p = mg$$



Moreover, we have to add the contribution of displacements due to spatial variability of motion.

- <u>Nonlinear dynamic analysis</u>
  In performing nonlinear dynamic analysis, we assume a certain structural factor q and we have to verify coherence in the choice.
  The verification of coherence is satisfied when the actual sum of actions on piers bottoms and abutments is bigger than 80% the one coming from linear analysis.
  $$\sum\left(E_{piers,bottom,nonlin} + E_{abut,nonlin}\right) > 0{,}8 \sum\left(E_{piers,bottom,lin} + E_{abut,lin}\right)$$

- <u>Nonlinear static analysis (Push-over)</u>
  The Push-over analysis consists in assigning horizontal forces to the piers and increase them until a pre-defined displacement in a referring node, e.g. the pier cap, is reached.
  For instance, we assign a horizontal force at the top of the pier until a certain displacement at the top.
  In presence of several piers mutually connected
    - We evaluate the plastic hinges formation sequence up to collapse.
    - We perform an analysis of the redistributions due to the formation of plastic hinges, i.e. the variation of static scheme.
    - We evaluate the rotation in plastic hinges under the pre-defined displacement.

  In this analysis, we have to control if it is enough: for the displacement evaluated with complete modal analysis and elastic spectrum (q = 1), the ductility requests in plastic hinges should be compatible with those available and actions in other elements should be smaller than the resistance, with the capacity design criteria.

CAPACITY DESIGN CRITERION

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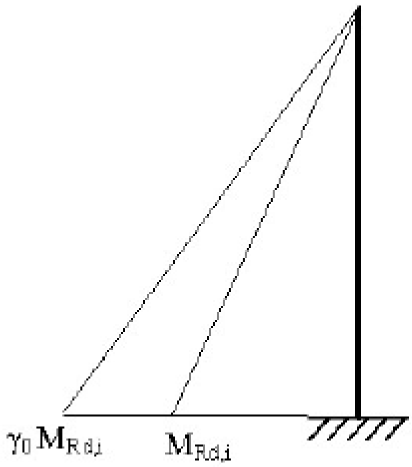


How can we ensure the formation of plastic hinges?

In plastic hinges, bending moment resistance is not higher with respect to the corresponding section without specific devices, as it depends on longitudinal reinforcement.
The difference is the capacity of deformation of concrete in compression, since the limit deformation of 3,5‰ can be increased 10 ÷ 15 times bigger.

To obtain this result, we need <u>confinement of concrete by reinforcement</u>.
Confinement is not necessary when the non-dimensional axial force is smaller than 0,08 – very small value.

$$\eta_k \leq 0{,}08$$

Another situation in which confinement is not required is the one of box sections or double T sections able to reach curvature in plastic field which is 12 times the curvature corresponding to the yielding of reinforcement.

$$\mu_c = 12\mu_y$$

In this situation, maximum deformation of concrete is 3,5‰, since there is no confinement.

Generally, we need confinement and it may be applied in different section, following some minimal rules in the design.

- Rectangular section
  The non-dimensional area of transverse reinforcement for confinement should respect two conditions.
  $$\omega_{wd,r} = \begin{cases} 0{,}33\dfrac{A_c}{A_{cc}}\eta_k - 0{,}07 \geq \begin{cases} 0{,}18 & ductile\ behaviour \\ 0{,}12 & limited\ ductile\ behaviour \end{cases} \\ \dfrac{A_{sw}}{sb}\dfrac{f_{yd}}{f_{cd}}, \qquad s \leq \begin{cases} 6\phi_{long} \\ \dfrac{1}{5} \times (minimum\ confinement\ dimension) \end{cases} \end{cases}$$
  A_c is the area of gross section of concrete, whereas A_{cc} is the area of concrete included inside the confinement.
  The distance s between confinement stirrups should not be bigger than 6 times the diameter of longitudinal bars or 1/5 of the minimum confinement dimension.
- Circular section

In this case, the design action is

$$1{,}5\alpha Q, \qquad \alpha = \frac{a_g}{g}$$

Q is the minor weight of the connected parts.

As regards displacement, we should take into account thermal effects in order to define the stroke.

$$l = l_m + d_{e,g} + d_{Es}$$

l_m is the dimension of the support, which should be bigger than 400 mm, $d_{e,g}$ is the effective relative displacement of ground and d_{Es} is the relative total displacement, which takes into account thermal effects.

$$d_{Es} = d_E + 0{,}4d_T = \pm\mu_d d_{Ed} + 0{,}4d_T$$

FOUNDATIONS

The design criterion is that the foundation should remain in elastic field or with negligible residual deformation in presence of the design seismic action.
By consequence, design is performed with reference to capacity design concept, for the actions $\gamma_0 M_{Rd,x}$ and $\gamma_0 M_{Rd,y}$. We assume at most a structural coefficient q equal to 1.

In case of foundations on piles, we will see the formation of plastic hinges in the connection of piles with footings and concrete rafts, since the plinth will translate in a rigid way and the pile will be interested by the formation of plastic hinge.
Even if we do not perform specific calculation – we are assuming elastic behaviour –, we can not forget that plastic hinges arise and we have to put confining reinforcement in the last two diameters of the length of the piles.

ABUTMENTS

The design criterion is that the abutment should remain in elastic field or with negligible residual deformation in presence of the design seismic action, in order to preserve its functionality with design seismic action.
By consequence, design is performed with reference to capacity design concept, so that they remain in elastic field.

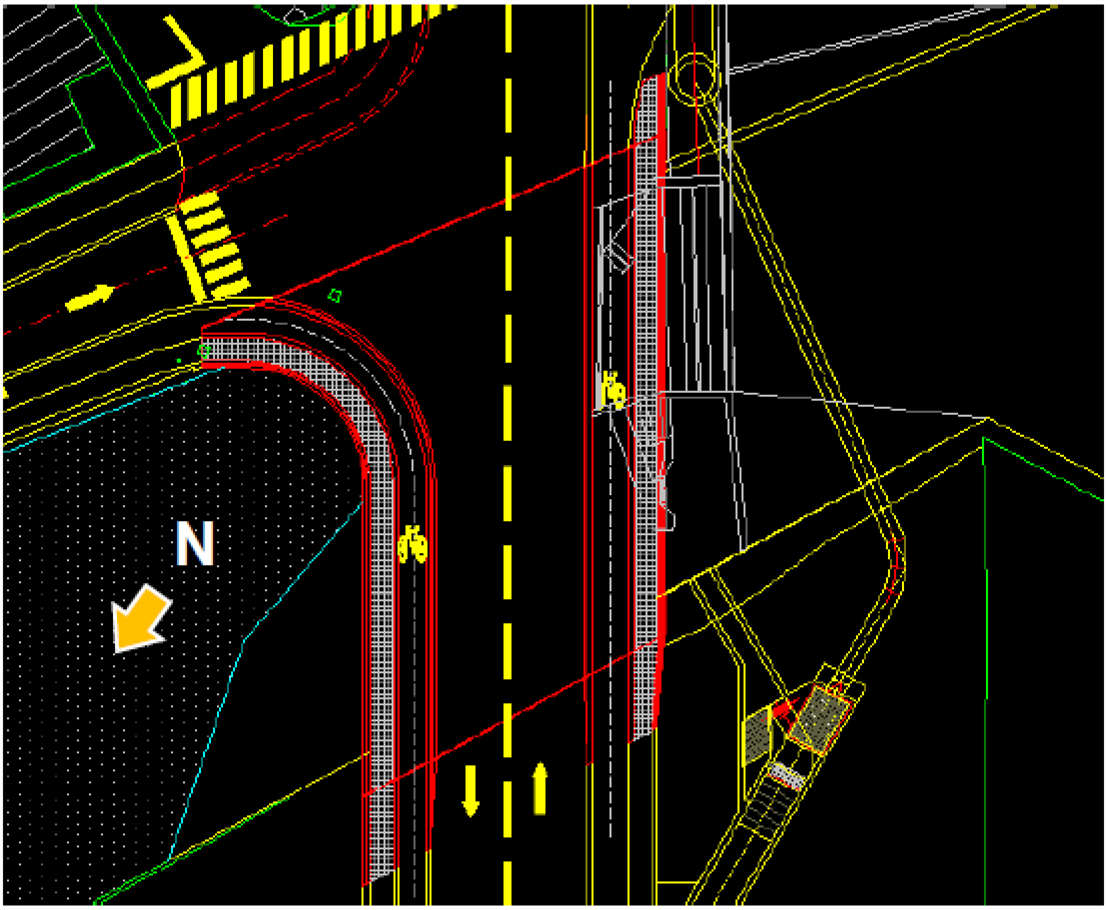
If the bearings are free in the longitudinal direction, we have to consider that the displacement may be uncoupled with respect to bridge, i.e. the bridge moves without being connected to the abutment and there is the risk of impact. By consequence, we have to calculate separately the displacement of the deck and the displacement of the abutment due to pressure of soil in dynamic conditions and to sum.
The seismic forces and friction forces of bearings should be protected by a capacity design, with a factor 1,3.

In case of fixed bearings both in transverse and longitudinal direction, there will be coupled displacement with the deck. In this situation, the structural factor q is equal to 1, since it is a very rigid element and there is no plasticization in design and seismic action is evaluated with a_g.

SLAB BRIDGE: RANTIVA BRIDGE

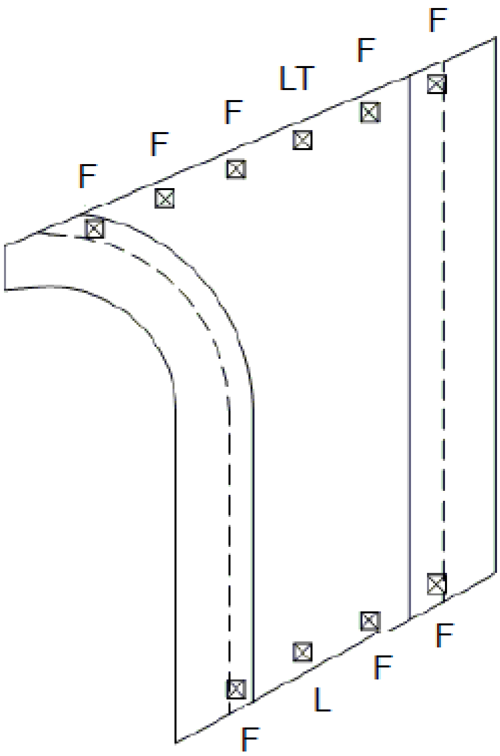
GEOMETRY

Rantiva Bridge is a small road bridge of 27,3 m of span over a channel.



It has an irregular shape because it is skew on the two sides. Moreover, there is a rounded part on one side as the road turns to an other one, which is parallel to the channel, and, since the road is opening, the width is 16,3 m on one side and 25,9 m on the other side.

- F stands for free movement in both directions – the bearing provides only vertical reaction.
- L stands for free movement in transverse direction and no movement in longitudinal direction – the bearing provides vertical reaction and horizontal reaction in the longitudinal direction.
- LT stands for no movement in the longitudinal and the transverse direction – the bearing is a 3D hinge giving two horizontal reactions and a vertical reaction.



The configuration realizes an isostatic scheme of the deck in the horizontal plane – of course, there is hyperstaticity in vertical direction. This is a good arrangement as, in slab bridge, the best solution is to place as less bound as possible. With only three degrees of freedom bonded in the horizontal plane, we grant the equilibrium of the structure and we avoid thermal forces, shrinkage and impressed deformation going to bearings and increasing internal actions.

MATERIALS

The materials used to realize the deck are the following ones.

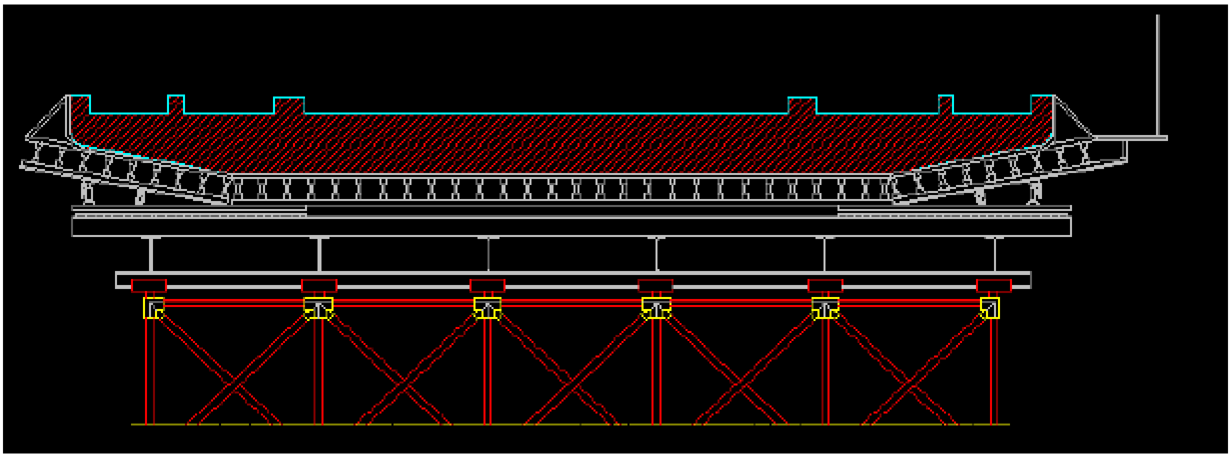
- Concrete

$$f_{ck} \geq 33\ MPa$$

- Prestressing steel with 0,6'' strands – standard strands for prestressing. Their area is 1500 mm^2, which is not the actual area but a given number, as a strand is not full section but has sparrels.

$$f_{pt,k} = 1860\ MPa$$
$$f_{pt1,k} = 1670\ MPa$$
$$E_s = 210000\ MPa$$

- Ordinary steel Fe B 44 k, described with the indications of the old codes. The main difference with respect to new codes is the yielding stress, which was assumed smaller in design.



Scaffolding

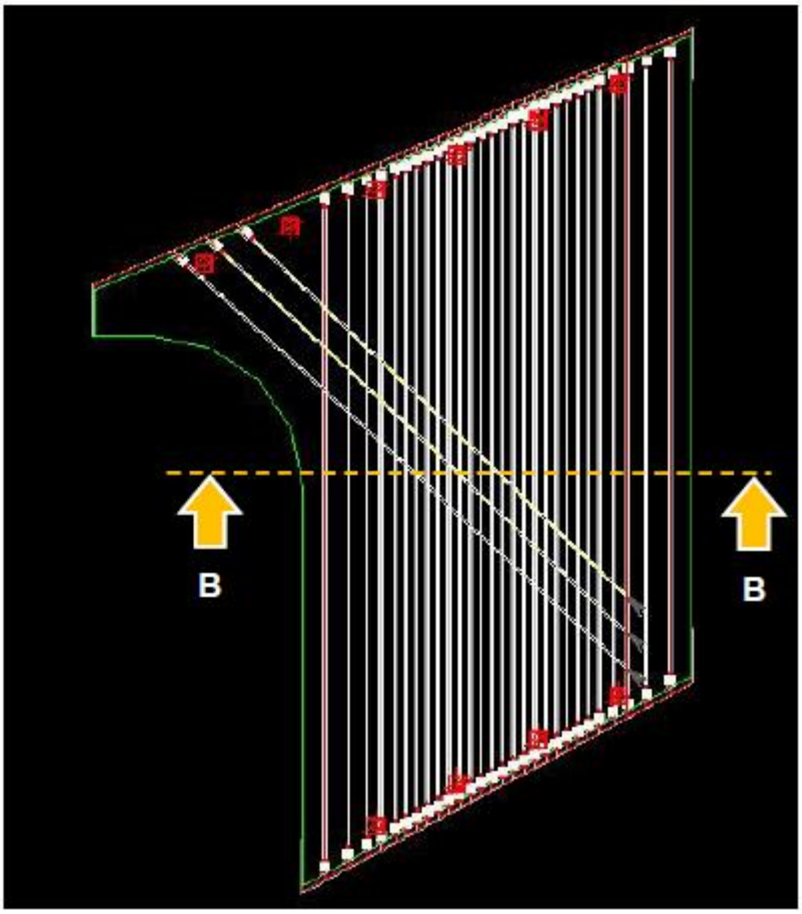
4. Once concrete has been hardened, they introduced prestressing in the deck slab, which creates a negative bending moment that lifted up the deck from the scaffolding and weight was closed on the bearings. At this point, they could dismount the scaffolding.
   After the introduction of prestressing, they build the walls of the abutments. They are quite strange abutments because they present a big foundation and a small wall because walls are used only to place anchoring jacks.

PRESTRESSING

Prestressing strands have been place inside holes and pulled by clams.

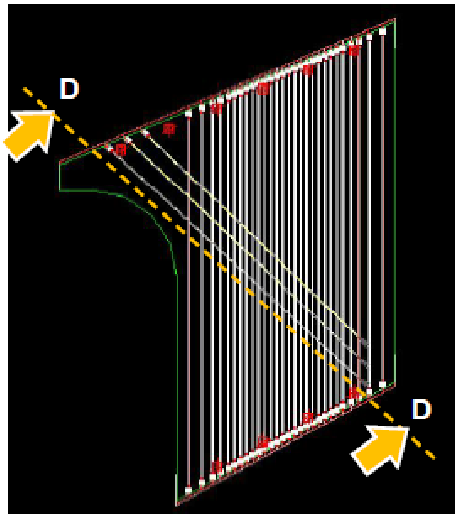
The arrangement of prestressing reinforcement presents 19 strands of 0,6'' placed inside HDPE ("polietilene") ducts, having an internal diameter of 100 mm and an external diameter of 114 mm. the pipe thickness of 14 mm provides insulation against corrosion, as HDPE is a tough material. Tensioning stress was 1420 MPa and the jack force was 3750 kN, that was a huge force.

The slab has three skew tendons. Their position has been defined through trial-and-error approach: we assume a certain layout, we run the calculation and the verification and we check if every condition is satisfied. If not, we change the number of tendons, the diameter and the position and, after a number of cycles, we obtain the correct tendon layout, which allows to verify ultimate conditions and serviceability conditions. By consequence, complete design of the bridge should be automatized because it is the only way to change prestressing layout and parameters quite easily.

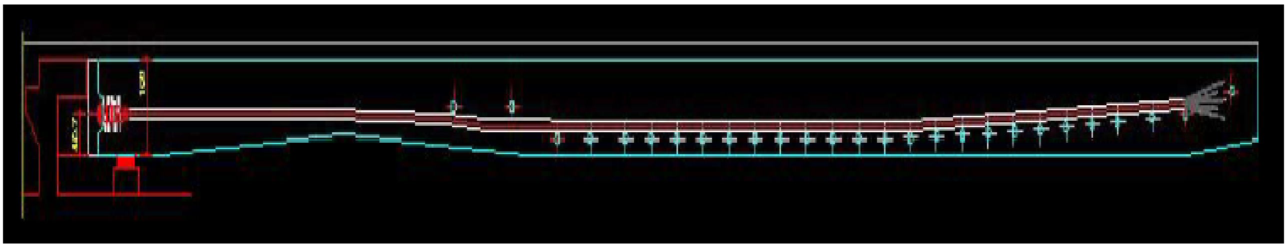


Looking at the cross section B-B, in correspondence of the mid-span in the direction orthogonal to the traffic, the tendons are placed at the bottom because positive bending moment is acting due to traffic loads and self-weight.
Skew tendons are passing above the other ones, in order to avoid interference. A general rule to avoid interference is to leave at least one diameter of spacing between parallel tendons.
In this case, a skew pipe is in contact with another one in a point – not in a line because it is a small point in the bridge – because they are not parallel in the vertical plane.

Prestressing tendons layout: skew section D-D



N.B. Vertical scale doubled



ORDINARY REINFORCEMENT

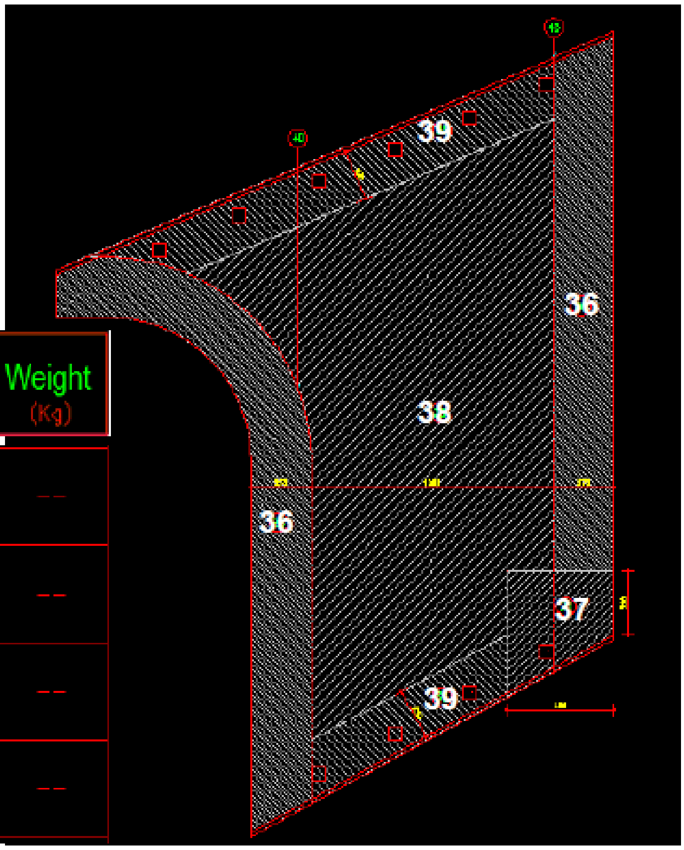
Ordinary reinforcement layout is simple because there are one top layer above and one bottom layer below and reinforcement is doubled in some sections. Then, we can notice vertical reinforcement due to shear and transverse reinforcement.

The position of each bar is identified by means of a number. Reinforcement is described by means of a table that gives, in association to the position, the diameter in mm, the spacing in cm, the shape of the bar – there are many pieces of bars overlapping themselves because the bridge is long – , the length – the length is different as the deck is skew –, number pf bars and weight. This information is required in the drawings because it is used by the workers to place the reinforcement in the correct way.

Ordinary reinforcement:

Stirrups layout

Pos.	ϕ (mm)	Shape	Length (cm)	N°	Weight (Kg)
36	14 / 40X40	17 94÷46 17	128 + 80	400 + 510	--
37	20 / 40X40	27 94÷46 27	148 + 100	138	--
38	14 / 40X60	17 94 17	128	1140	--
39	16 / 40X40	20 94 20	134	432	--



In the disposal of reinforcement, we place plastic ducts for prestressing running in the longitudinal direction inside the reinforcement, that are empty and closed on both sides and some vertical pipes on the two sides, in order to create the hole to place the guardrail. There are also some small black tubes connected to the ducts and these are air-exausting tubes. They are required because, after the tensioning of the cable, grouting is done and there is the necessity to expulse the air inside the ducts, otherwise air bubbles may create.

Live anchorage detail

Confinement reinforcement

Each anchorage is identified with a number written along the side of the slab, in order to apply the prestressing in the correct sequence. The anchoring heads present tendons going out and the swedges (“cunei”) used to lock them in their position. The element on the left with vertical reinforcement is the top of the abutment.

Live anchorage detail – anchorage heads

Live anchorage detail

Reinforcement description

Pos.	ϕ (mm)	Shape	Length (cm)	N°	Weight (Kg)
8	20 / 20	27 36÷1146 27	89 ÷ 1200	31	--
10	20 / 20	27 1173 1200 580÷750 27 sovrapp.≥125, alternare le sovrapp.	1200x2 + 607÷777	82	--
12	20 / 20	27 605÷1173 sovrapp.≥125, alternare le sovrapp.	632 ÷ 1200	57	--
23a	20 / 20	77 R=25m Sv=35cm 250 28÷861 27	367 ÷ 1200	23	--
42	26 / 20	1200 810 sovrapp.≥180	1200 + 810	5	--
42a	26 / 20	27 275 580 275 27 R=25m Sv=35cm 883 R=25m Sv=35cm sovrapp.≥180	1200 + 917	5	--

Pos.	ϕ (mm)	Shape	Length (cm)	N°	Weight (Kg)
44	20	25 94 25	144	1+1 x 50	--
45	26	35 94 35	164	1+1 x 50	--
46	20 / 20	25 94 25	144	5+5 x 50	--
47	20 / 20	120° 60 120	180	(1+1)x10 x 2	--
47a	20 / 20	152° 148° 55 120 10	185	1+1 x 8	--
48	20 / 20	773 673 27 27 27 17 773 673 In corrispondenza dei cavi 3÷26	800x2 + 700x2	6 x 2	--

DEAD ANCHORAGE DETAIL

In this anchorage, each strand opens – candy-shaped strands – to give a better anchorage and the region presents small square stirrups in order to simulate the effect of the spiral. At the end of the duct, plastic foam is applied in order to avoid concrete going inside it during the casting.

Dead anchorage detail

Dead anchorage detail

Reinforcement description

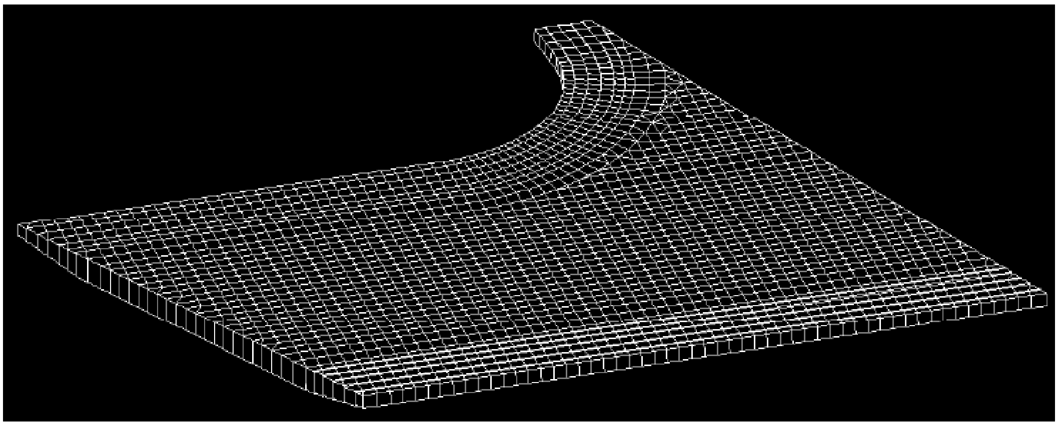
Pos	φ (mm)	Shape	Length (cm)	N°	Weight (Kg)
49	20 / 20	150, 60, 150	360	1+1 x 3	--
50	20 / 20	160, 80, 160	400	1+1 x 3	--
51	20	25, 50, 25	100	2+2 x 3	--
52	10 / 20	10, 10, 20, 15, 20, 15	90	3	--
52a	10 / 20	10, 10, 25, 20, 25, 20	110	3	--
53	20 / 20	120	120	3+3 x 3	--
54	20	25, 94, 25	144	1+1 x 3	--
55	26	35, 94, 35	164	1+1 x 3	--
56	20 / 20	25, 94, 25	144	5+5 x 3	--
58	14	7, 30, 40, 7 spire	440	3	--

Mesh

1. Elements of different thickness (see table)
2. The mesh is not plane as it is realized in the centroid surface of the slab (see picture)

Slab zone	Thickness [m]
1	0.52
2	0.61
3	0.71
4	0.80
5	0.88
6	0.96
7	1.00

There is a 3D view of the mesh, only to give an idea, as the mathematical model is a surface located on the mid plane.



The mesh layout is chosen with reference to some guidelines.

- The mesh should fit properly the geometry of the structure. By consequence, we should avoid skew irregular elements as much as possible and follow thickness changes and possibly main reinforcement distribution, i.e. the mesh should be aligned in the direction of the main reinforcement.
- The mesh should have a number of elements able to describe properly the deformation of the structure.
- The mesh should have a regular spacing between the nodes in order to be able to place live loads and prestressing equivalent loads as nodal loads.

$$pds = prd\alpha$$

As the tendon is not going to move, we impose equilibrium and close the triangle of forces.

If we neglect friction, the force exchange between the tendon and the concrete will be the following.

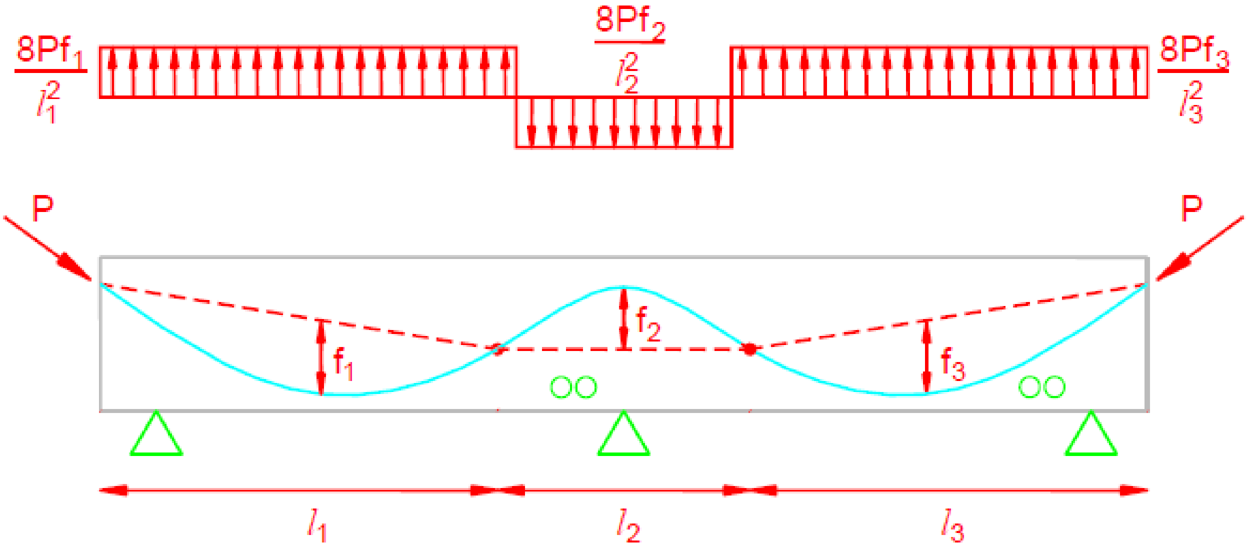
$$p = \frac{P}{r}$$

Assuming a parabolic path of the tendon, we get a closed-form solution of this force.

$$p = \frac{8fP}{l^2}$$

The force exchanged between the tendon and the concrete is function of prestressing force P and the deflection f of the tendon.

With reference to a hyperstatic beam with two spans, generally the tendon layout is curved according to a certain shape.
This shape may be realized by combining three parabolic curves arranged in a certain way. Knowing their deflection f_i, in function of it we evaluate the equivalent forces that will be placed on the structure, in absence of friction.



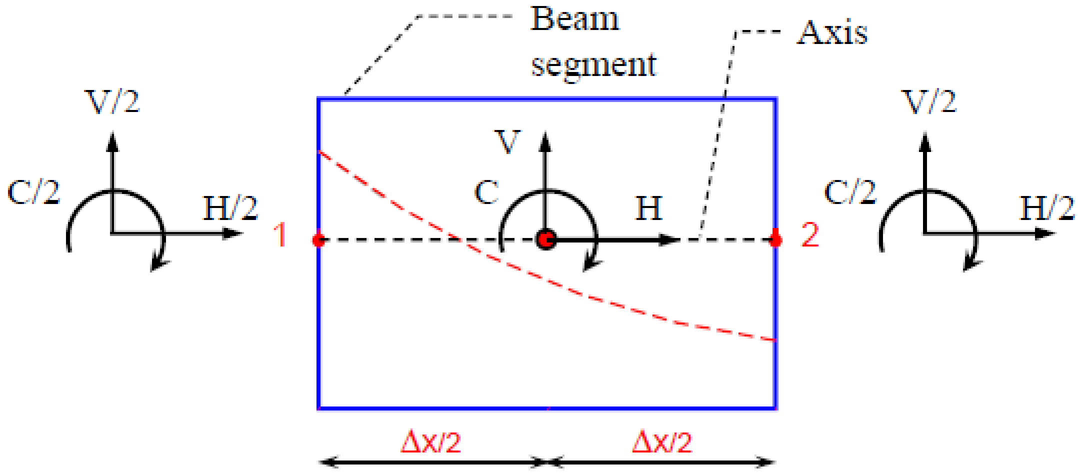
Of course, this is a simple approach, based on two important hypotheses.

- All tendons follow a parabolic path, which is not the case of bridges.
- The value of the prestressing force P is constant along each tendon but it is not, due to friction.

Actually, this model could not be adopted for the evaluation of equivalent forces.

A more general approach is based on an approximation, in which the beam is divided into a number of segments with an arbitrary length Δx – generally, the length is equal to the depth. In the centroid of each segment, we place a set of forces, representing the equivalent loads in the segment due to the presence of tendons.
In a plane system, each segment has 3 degrees of freedom and forces coming from prestressing are in a number of 3, i.e. 2 forces and 1 bending moment.

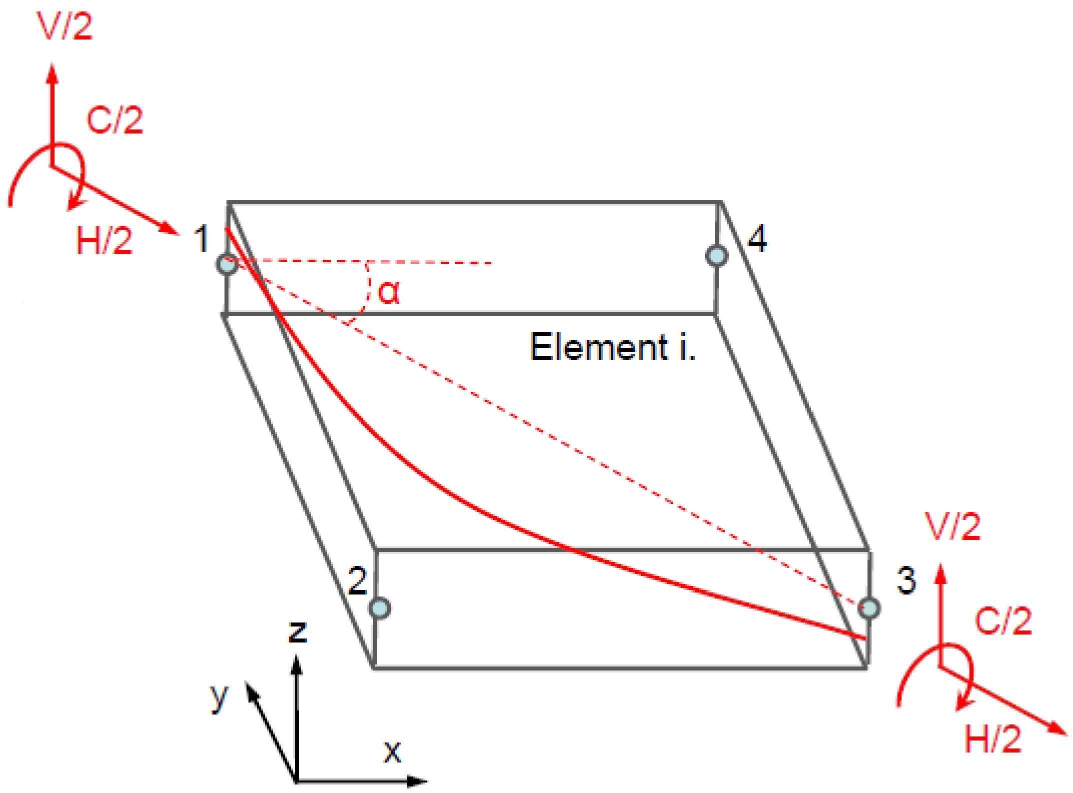
The equivalent loads can be substituted by equivalent ones placed at the extremities. This is useful in case of finite element nodes coincident with the extremities of the segment.



In case of shell elements, that are two-dimensional elements, we can identify three situations.

- Tendon is passing through two nodes of the same side, which is the situation of the longitudinal tendons in the deck.
  Given a shell element with four nodes, tendon is aligned on one local direction, in this case x direction. The equivalent actions in nodes 2 and 3 can be evaluated by using the same static scheme for the beam and the nodal forces can be approximated as one half of these actions – it is a fair enough approximation.

$$F_{2,x} = \frac{H}{2}$$
$$F_{2,y} = 0$$
$$F_{2,z} = \frac{V}{2}$$
$$M_{2,x} = 0$$
$$M_{2,y} = \frac{C}{2}$$
$$M_{2,z} = 0$$
$$F_{3,x} = \frac{H}{2}$$
$$F_{3,y} = 0$$
$$F_{3,z} = \frac{V}{2}$$
$$M_{3,x} = 0$$
$$M_{3,y} = \frac{C}{2}$$
$$M_{3,z} = 0$$

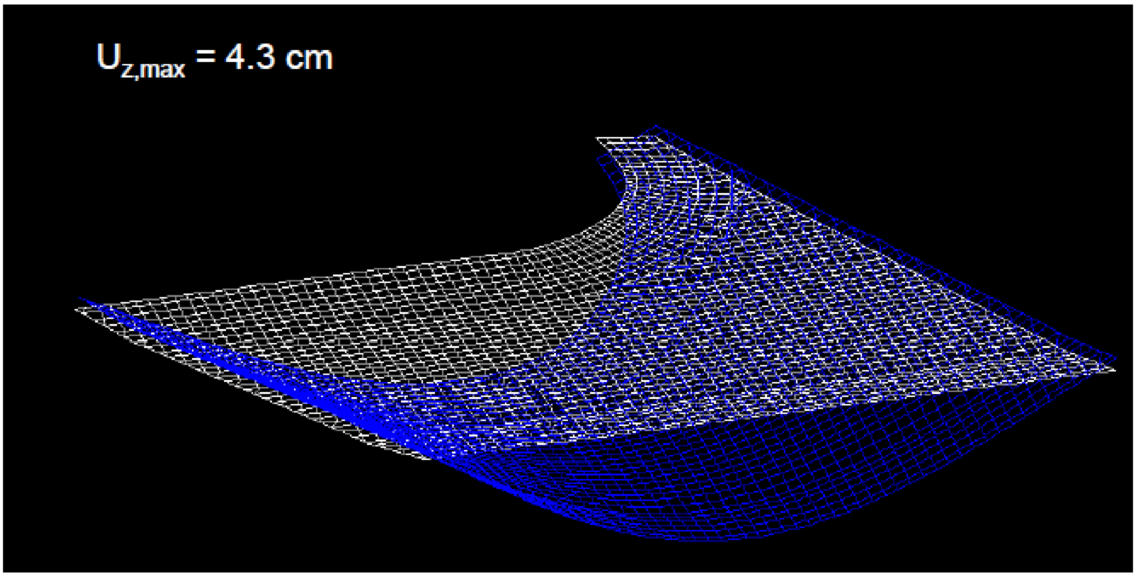


- Tendon is crossing the element in a general position.
  The computation may be complicated, but we can adopt some approximations. Considering the direction of the tendon, which is inclined of the angle α with respect to x direction, we evaluate the equivalent loads in this direction as a mono-dimensional system. These actions are applied at the centroid of the projection of the tendon in the mid plane of the element – it is not coincident with the gravity centre of the element – and the actions are then decomposed into 6 components.
  Finally, these components are transferred to nodes and, to do it, we can follow two ways.
  - Use of the shape function of the element, which is manually written by user or given by producer –, but it is quite a complicate option.
  - Adoption of the simplifying hypothesis according to which each node takes an amount of force inversely proportional to the distance between the application point of equivalent loads and nodes. If this point is close to the node, a big amount of force will go to the node; if this point is far from the node, a small amount of force will go to the node.

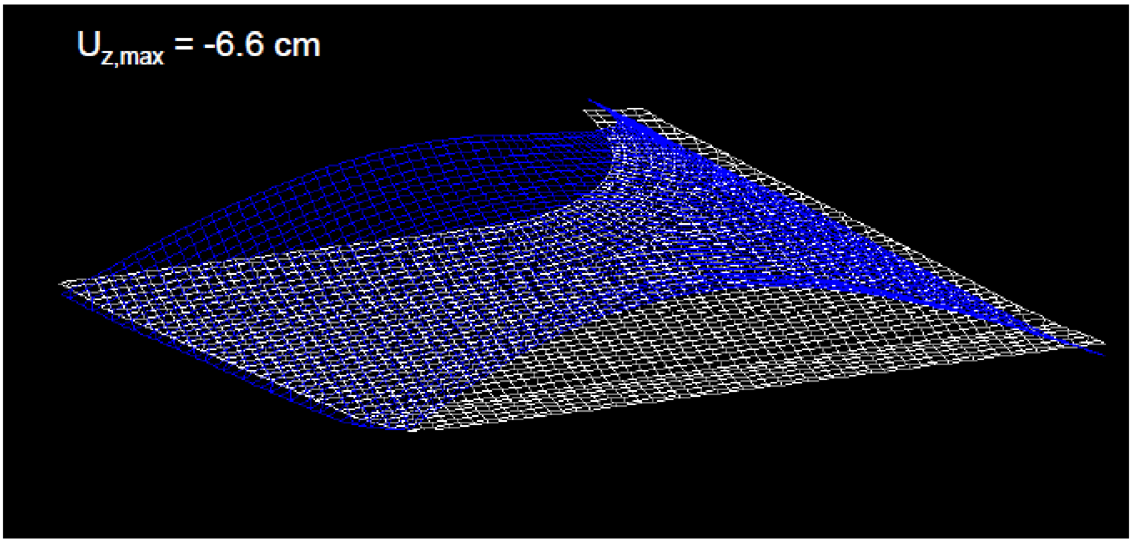
ANALYSIS RESULTS

Once the model has been set, we can see some analysis results.

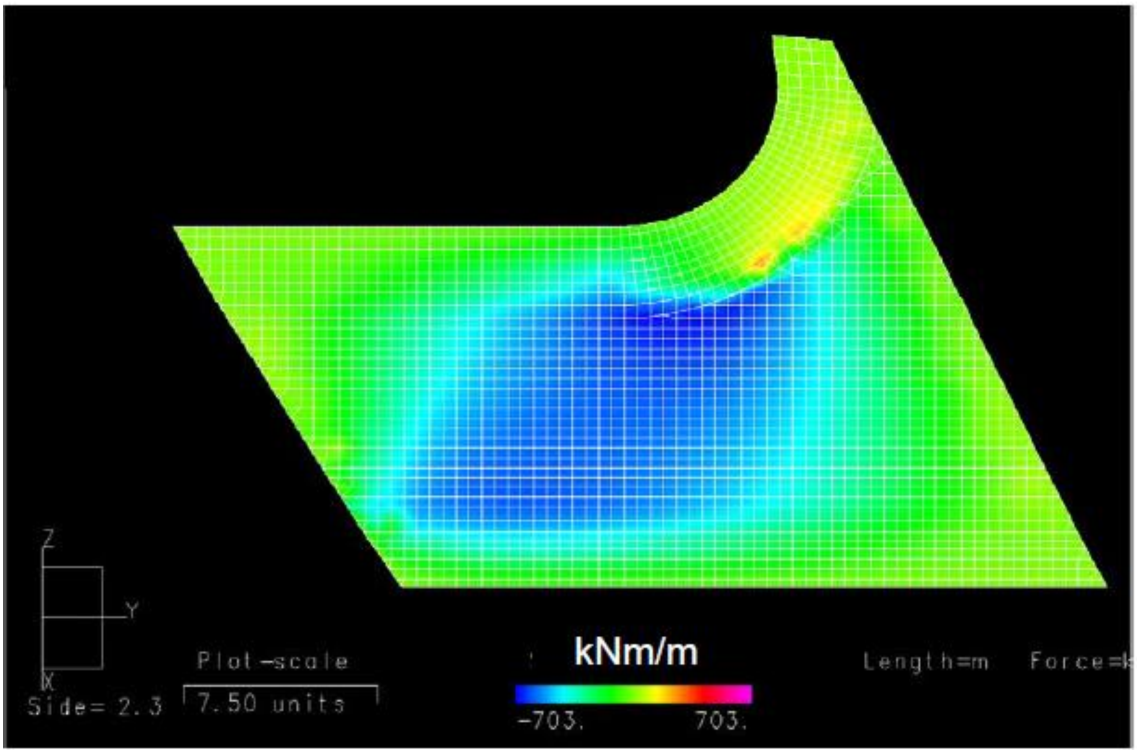
- The deformed shape of the deck due to dead weight shows a maximum vertical displacement of 4 cm with respect to 18 m of longitudinal dimension, which means a vertical displacement of 2 cm every 10 m.



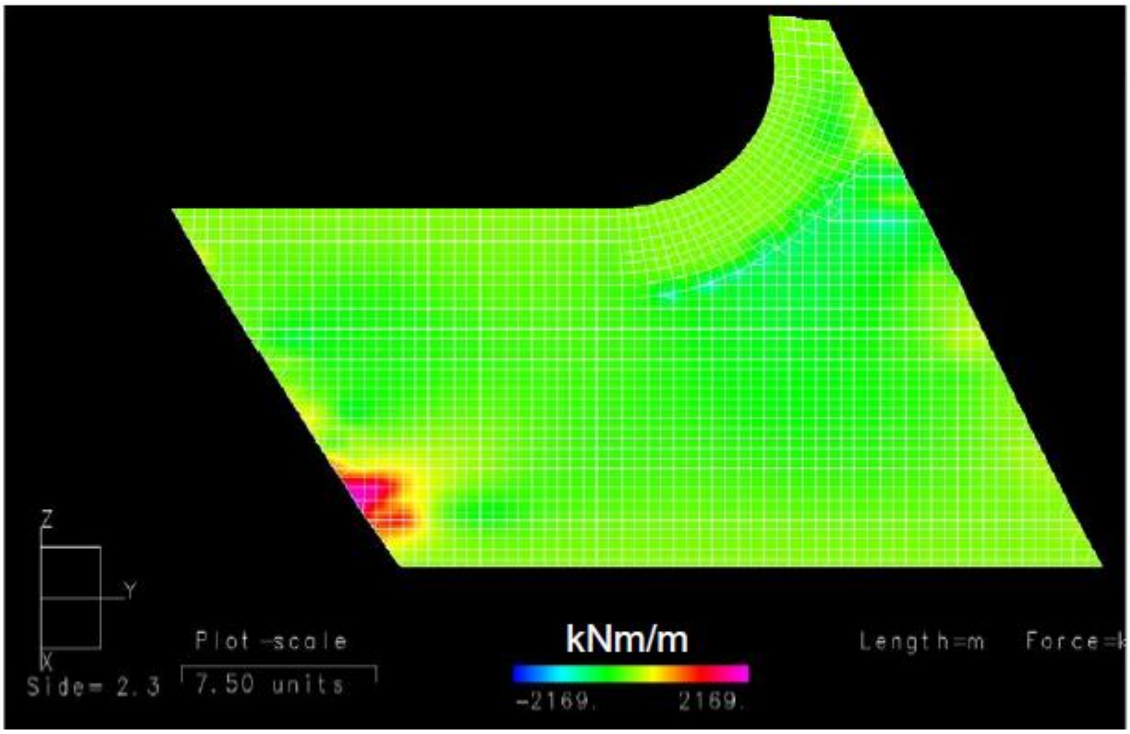
- The deformed shape of the deck due to prestressing is in the opposite direction with a maximum vertical displacement of 6,5 cm. indeed, generally prestressing – as an action – is bigger than self-weight, also 1,5 times bigger.



- Bending moment m_x in x direction due to self-weight, which generates stresses in the transverse direction – transverse bending moment.

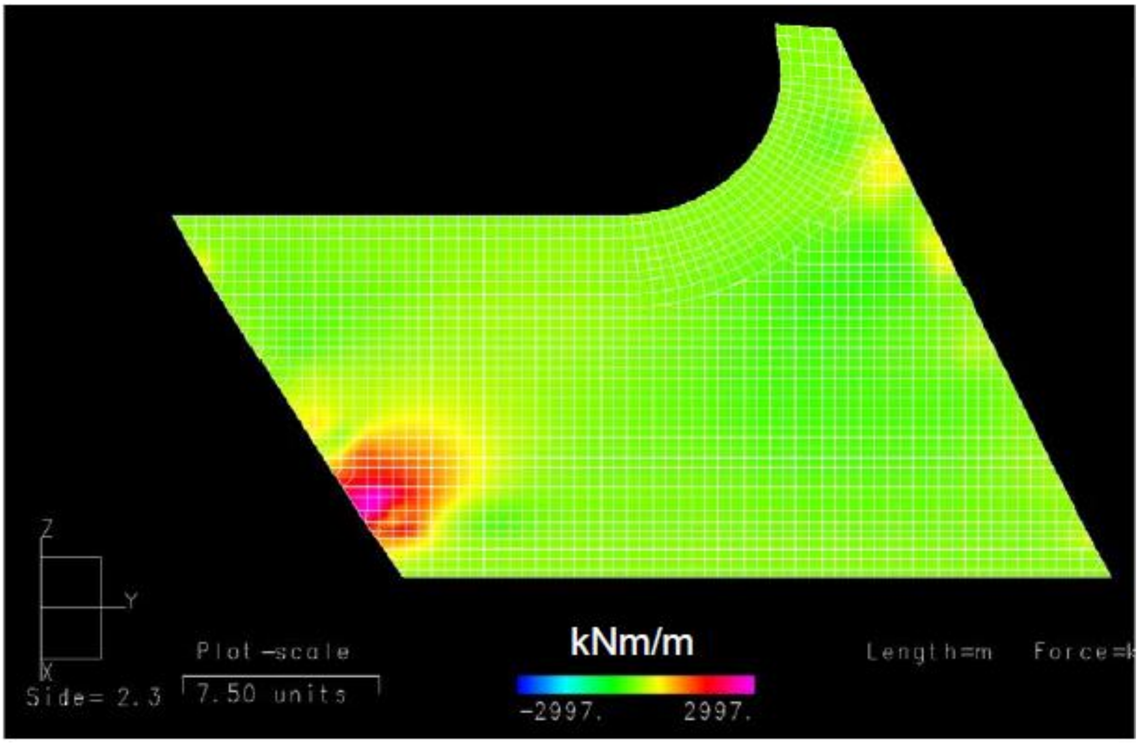


- Bending moment m_x in x direction due to prestressing, which is small as most of prestressing is aligned in the longitudinal direction and does not generate stresses in the transverse direction. There is just a concentration next to the corner due to the anchorage of skew tendons.

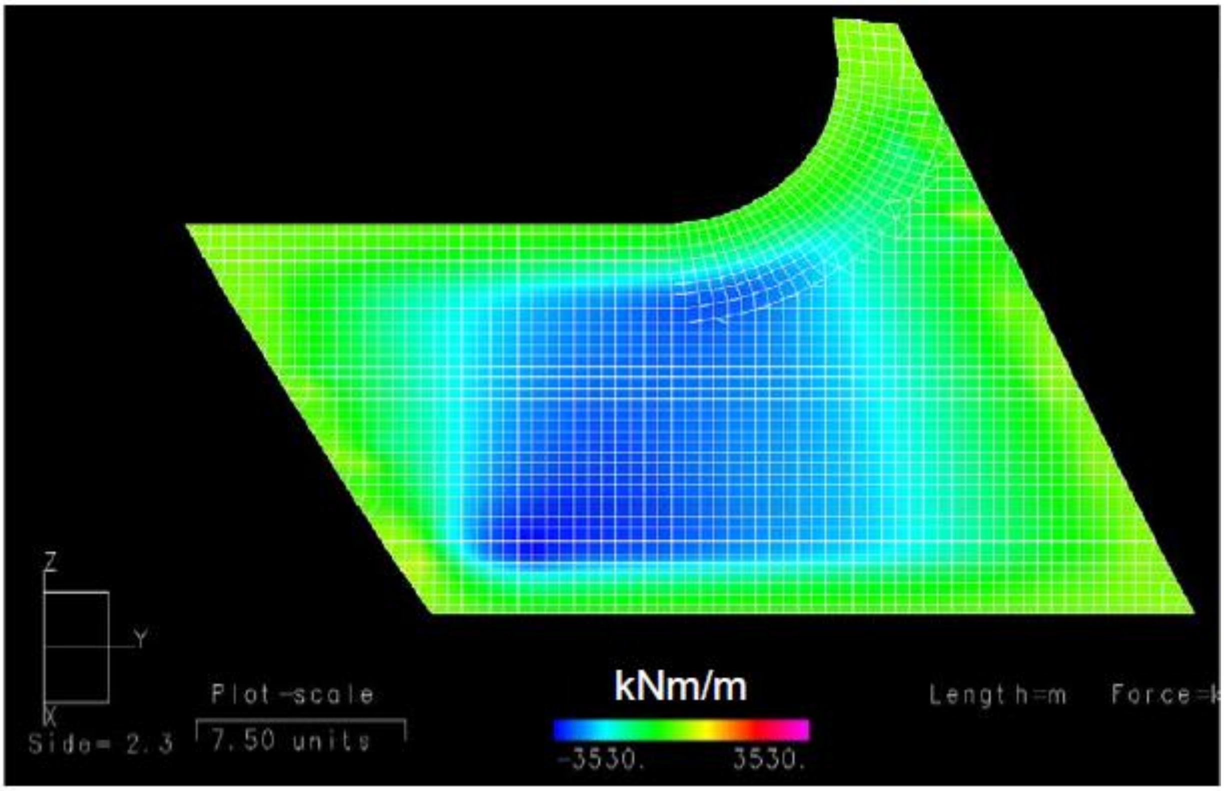


- Bending moment m_y in y direction due to prestressing, which presents a big negative value in order to counterbalance the positive bending moment, with a value of 3250 kNm/m with respect to 2000 kNm/m.

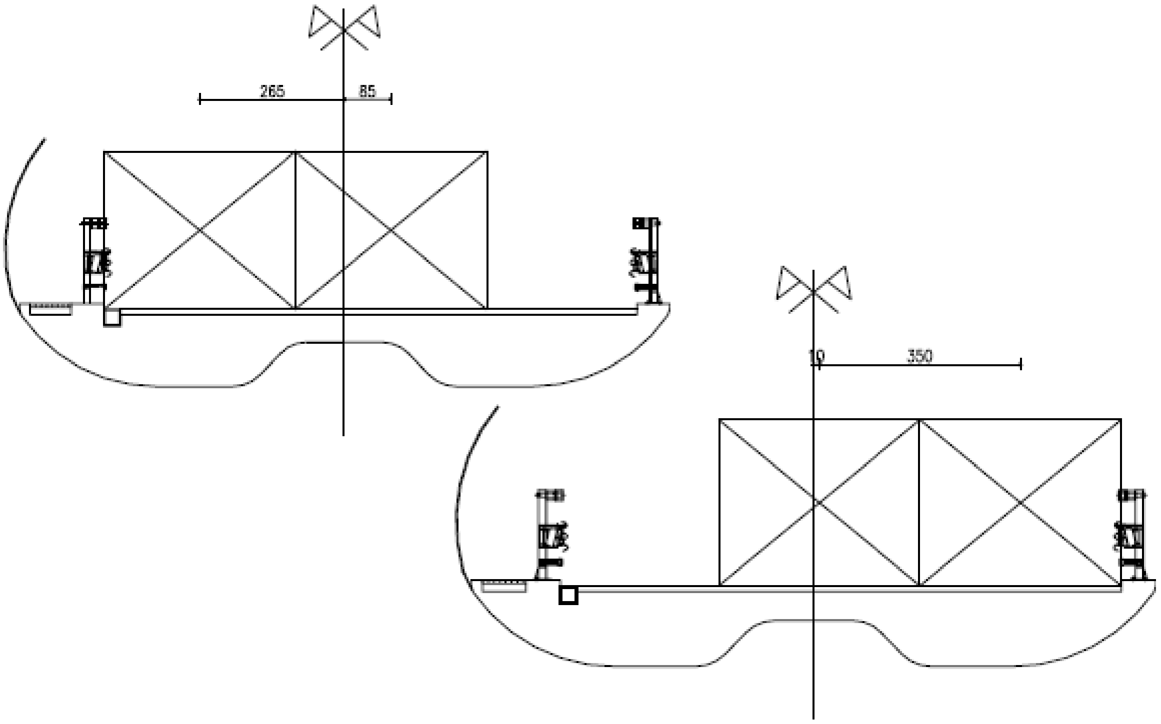
different due to prestressing losses, with a loss of 50% - it is a small value, related to the fact that the bridge has short span and the deck is a slab.



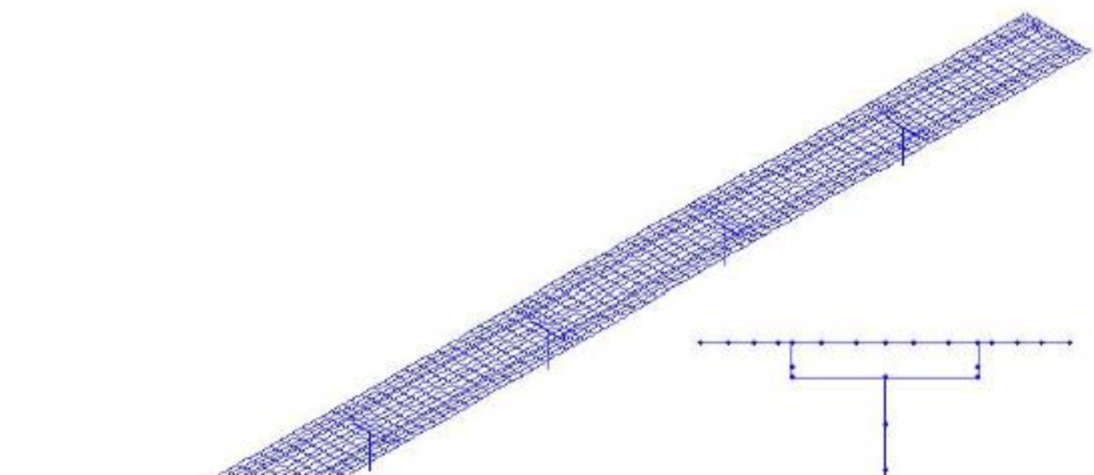
- Principal bending moment m_2 at the end of service life is negative as principal stress σ_2 is negative due to prestressing. It presents a decrease of 15% of its value due to prestressing losses.



and, in this way, actual beams are loaded. Indeed, the software is able to run a moving load but it needs a real or virtual path, along which the movable load is moved.
The position of these longitudinal elements is related to the position of loaded lanes on the deck – the boxes in the picture – and each box needs a couple of paths in order to let wheels run. Moreover, we need an additional path for pedestrian traffic.



By consequence, there is the necessity of a lot of longitudinal elements to identify the transverse position of the loads.
To do this, we realize a model which is essentially a two-girder model. Actually, seeing from the top, it resembles a slab.



In the front view, it resembles a box section beam. This is due to the variable cross section in order to have space for the bearings. Thus, at the top of the bearings, we need to model the real positions bearings are broken, the deck is kept in position without stiff bumps, granting safety conditions and no important damages to the structure.

The bridge was designed in 2003, when the seismic code was different. Indeed, in Italy there was a quick development of seismic provisions in the last 2 years and, nowadays, there is a good seismic code, written by Italy and Greece.
Due to the different code, we can notice that there is much reinforcement at the foot of the pier, as old codes do not give any provisions about seismic insulation, which was considered as a special device. By consequence, there is too much reinforcement for such short piers and also beams are short and stiff and they feel seismic actions very strongly.

Detail of reinforcement cage at the foot of the pier

Thus, the bridge tends to show problems in presence of seismic actions.

In order to improve its behaviour, more operations have been performed.

- Pier foundations were lowered more than the necessary, lengthening the piers a little bit. In this way, its slenderness, deformability and longer vibration time increase and vibration frequency decreases and the bridge feels less seismic action.
- Isolation system was placed in the bearings, in order to make the pier free to move, whereas the deck remains in the same position during the seismic action and the action does not go inside the deck. In this way, the system presents only the kinetic energy of the piers and the system is more efficient, as it requires less reinforcement.

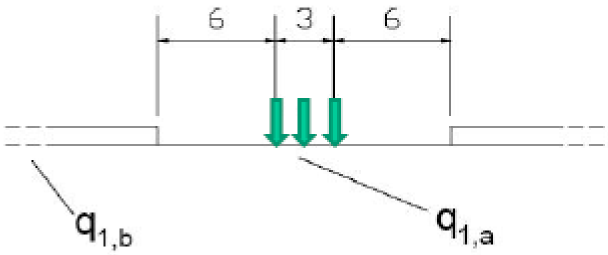
BEARINGS

As regards bearings configuration, they are associated with a code defining the degree of restraint exerted by them.

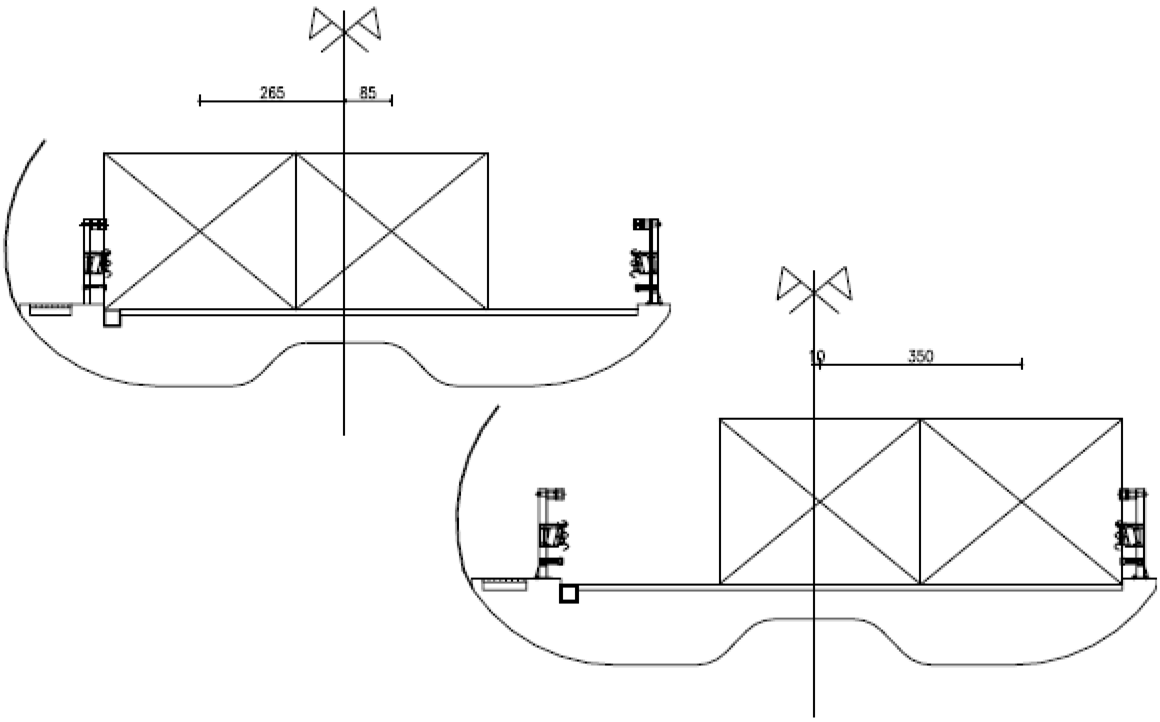
LOADS

As regards load, the bridge was designed with Load Model 1 according to the old code. This consists of three concentrated forces $q_{1,a}$ of 150 kN each – three tandem systems – with a total resultant of 600 kN and a distributed load $q_{1,b}$ of 3 kN/m^2, non-continuous along the tandem systems, with a free space of 6 m on each side. Nowadays, tandem systems are concentrated over a smaller area and distributed load is continuous, so new code is more severe from this point of view.

LM1 according to old Italian code



Live loads are applied with reference to the position of notional lanes, represented on the cross section of the deck. We can notice that there is a remaining area, which might be free for traffic but it is not loaded in the model. The systems of forces are acting with different eccentricities of notional lanes in the two decks.



As traffic is a movable load, we have to evaluate different situations.

Focusing on the tandem system's load $q_{1,a}$, tandem has been placed 64 times in different longitudinal positions on the girder model, with stepping of 3 m. Stepping is our choice and the closest it is, better the results will be, but it also requires computation times, so we should make a compromise. A stepping of 3 m is quite a fair choice as span is 30 m long and so each span is analysed

In conclusion, there are 128 different positions for concentrated loads and 128 different positions for distributed loads and, in order to obtain effects due to Load Model 1, we have to solve 256 configurations.

MATERIALS

The materials used to realize the deck are the following ones.

- Different concrete classes were used.
  - Good concrete for the deck.
    $$R_{ck} \geq 40\ MPa$$
  - Average concrete for the deck.
    $$R_{ck} \geq 35\ MPa$$
  - Normal concrete for the deck.
    $$R_{ck} \geq 30\ MPa$$

- Prestressing steel made of posttensioning tendons with 0,6'' strands – standard strands for prestressing. Their area is 1500 mm^2, which is not the actual area but a given number, as a strand is not full section but has sparrels.
  $$f_{pt,k} = 1860\ MPa$$
  $$f_{pt1,k} = 1670\ MPa$$
  $$E_s = 200000\ MPa$$
- Ordinary steel Fe B 44 k, described with the indications of the old codes. The main difference with respect to new codes is the yielding stress, which was assumed smaller in design.
  $$f_{yk} \geq 430\ MPa$$
  $$E_s = 200000\ MPa$$
  $$f_{yd} \geq 374\ MPa$$

CONSTRUCTION PHASES

The realization of this bridge was performed by using moving scaffolding technique.

This technique uses a set of wooden mould elements, which are used several times, as we do not build immediately the complete scaffolding, but we proceed piece by piece. Indeed, these elements are segmental and can be mounted, cleaned, dismounted and mounted again.

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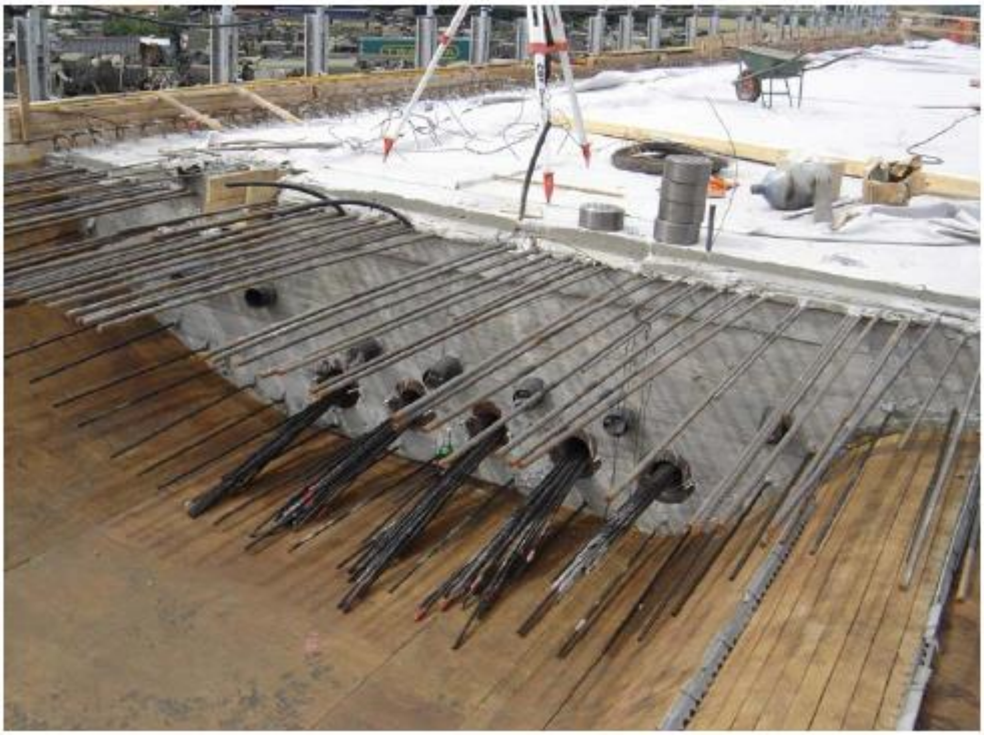
consequence, construction was performed with two working groups, one to build the scaffolding on one span and the other one to place reinforcement, cast concrete and dismount the scaffolding.

Reinforcement just before concreting

In correspondence of the joint, we can notice ordinary reinforcement coming out for overlapping and tubes used as prestressing ducts. Some strands are already placed. In the picture, there are also couples, used to couple each strand with the subsequent one and tie them together.

After concreting, the wooden element is removed and we see the overlapping reinforcement previously placed and five tendons ready to be coupled. These tendons come out from a hole having a high radius of curvature, which corresponds to a coupler. Indeed, couplers are made of two cones, one placed inside the old casting and the other one placed inside the new casting. The other seven tendons are not coupled, as they are continuous on all the bridge.



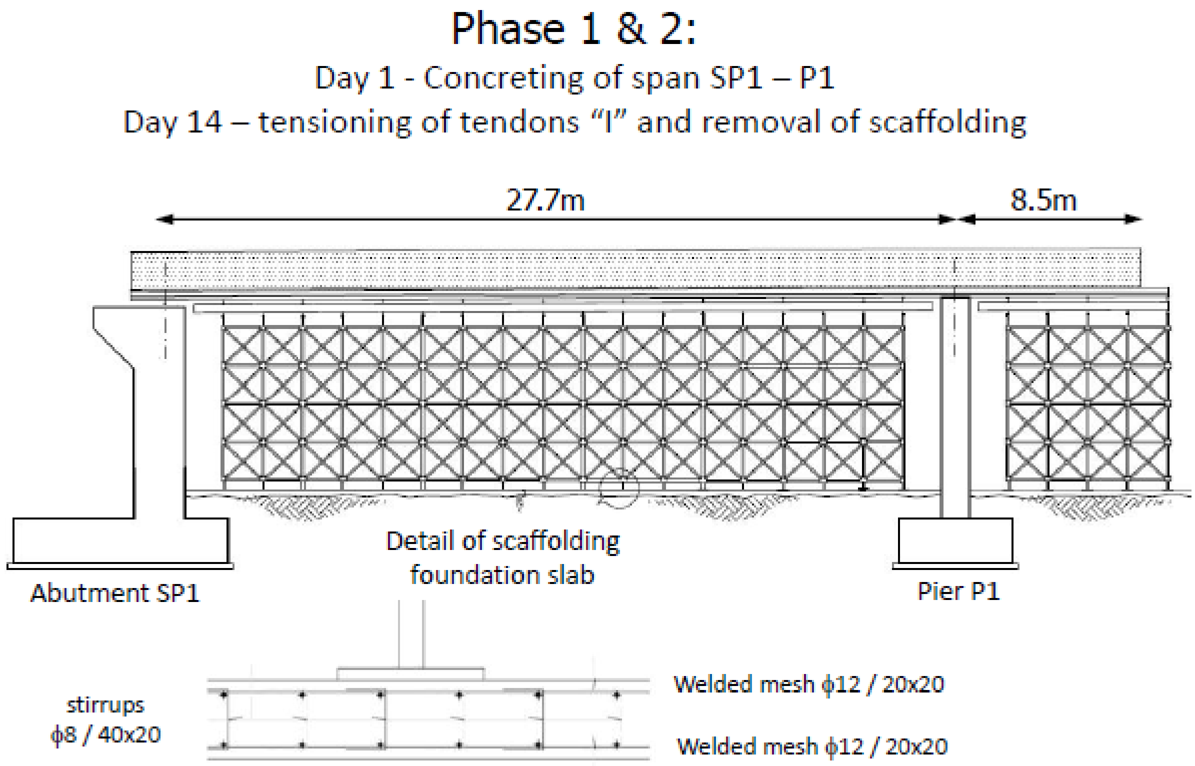
In this situation, in order to remove the scaffolding on the already built part, it is necessary to leave some prestressing in order to bear self-weight, otherwise the part would fail as ordinary reinforcement is not enough. By consequence, after concrete hardening, we apply prestressing to

2. Disposal of the reinforcement cage on the first span and realization of the scaffolding of the remaining part of the second span.
3. Concreting of the first span and disposal of the reinforcement cage on the second span.
4. When concrete is hardened, application of the first construction phase prestressing and use of the first scaffolding for the third span, and so on.

The construction technique is called moving scaffolding and this is the simplest moving scaffolding technique, as it is done all by hand – no machinery.

We see that translated in chronological terms.

1. Day 1: concreting of first span SP1-P1.
2. Day 14 (concrete is hardened): application of first stage tendons and removal of scaffolding.



3. Day 45: concreting of second span P1-P2, where scaffolding installation ad disposal of reinforcement takes 45 days from concreting.
4. Day 58: tensioning of first stage tendons and removal of scaffolding.

Phase n°	Time [d]	Description	Tendons
1	1	-	-
2	14	Scaffolding removal SP1-P1	I1
3	57.99	-	-
4	58	Scaffolding removal P1-P2	I2
5	63	-	-
6	68	-	-
7	78	-	-
8	88	-	-
9	101.99	-	-
10	102	Scaffolding removal P2-P3	I3
11	107	-	-
12	112	-	-
13	122	-	-
14	132	-	-
15	145.99	-	-
16	146	Scaffolding removal P3-P4	I4
17	151	-	-
18	156	-	-
19	166	-	-
20	176	-	-
21	189.99	-	-
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		

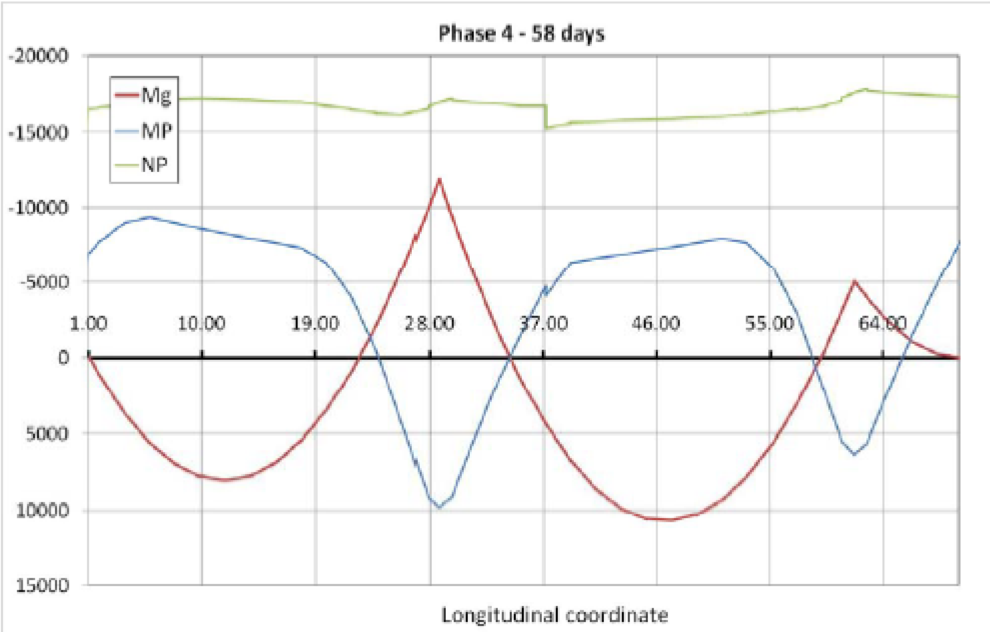
End of construction tendons are stressed at day 239 and permanent loads are introduced at day 300, i.e. the bridge was built in one year.
From this point, there are different phases in which nothing happens and time goes on, until the limit of service life, corresponding to 170 years.

This aspect is important because bridge should be designed in order to have a good behaviour both at construction time and at the end of service life, where creep, prestressing losses occur, taking also into account the redistribution of internal actions due to variation in static scheme in the different construction stages, related to creep.

Phase 4:

Day 45 - Concreting of span P1 – P2

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



At day 45, bending moment due to self-weight reduces and passes from -5000 kNm to -10000 kNm at the first pier, due to the weight of the second span – 5000 kNm was the effect of the cantilevering span.
In the region of positive bending moment, e.g. in the longitudinal coordinate of 10 m in the first span, the value comes down to 7000 kNm due to negative bending moment.

From the creep point of view, stresses in the first span are changing due to the variation of static scheme and part of creep is evolving and starting from a different value of bending moment.
By consequence, in each construction stage, internal actions change not only in the new part but also in the old part and creep will change in time, from that time we have to correct all creep effects from that time and at every stage.
This is a complicated analysis, due to the evolution in creep effects in time, at each phase.

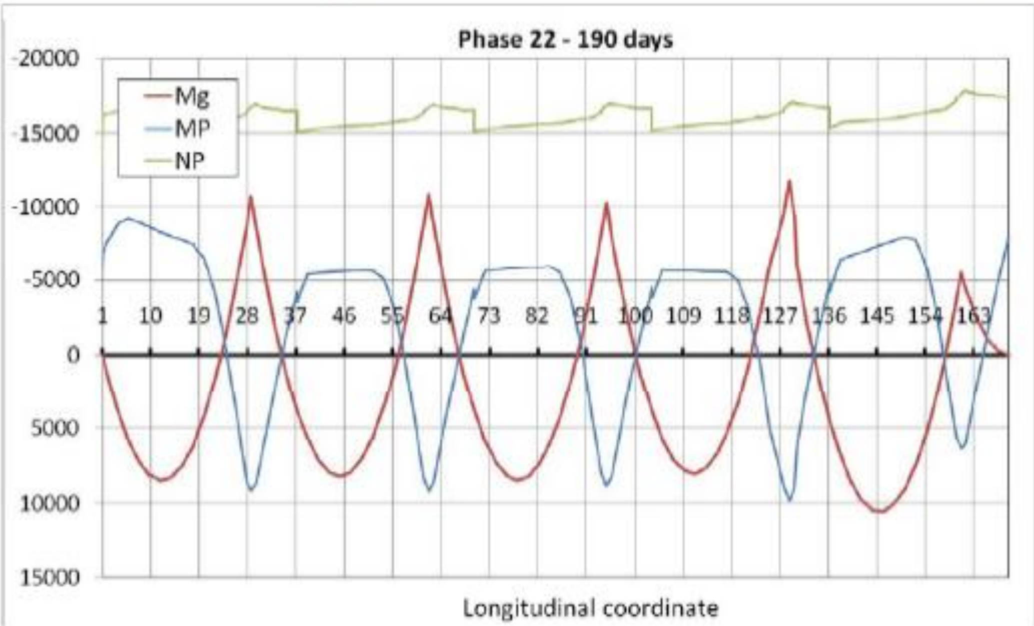
Axial force shows a jump due to couplers, as there is a minimum movement, which is free – it is not infinitely rigid and, when tendon is pulled from the end of the second portion, axial force reduces due to friction and there is a loss at the coupler, as one part is tensed and the other one is different.
Moreover, there is big loss in the axial force in correspondence of the pier due to high curvature of the tendon, which determines big friction.
Bending moment due to prestressing is positive or negative, due to the going up and down of tendons.

At the end of construction of the second span, we can see the couplers' effect and bending moment in the first span passes from 7000 kNm to 8000 kNm, i.e. it increases because the weight of the new span gives positive bending moment. Moreover, the position of the maximum changes.

Phase 22:

Day 175 - Concreting of span P4 – P5

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

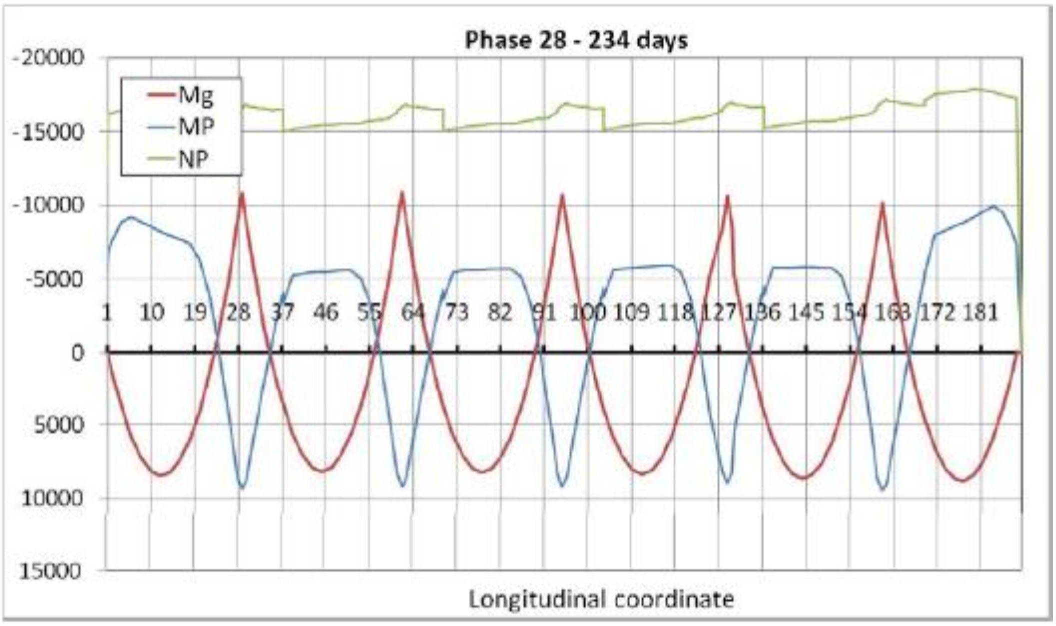


At the end of construction, all the six spans are realized and bending moment due to self-weight is very regular, with a value of 10000 kNm, very regular of bending moment due to prestressing – opposite sign and value of 10000 kNm – and regular axial force with a value close to 16000 kN and the effect of couplers.

Phase 28:

Day 220 - Concreting of span P5 – SP2

Day 234 - tensioning of tendons "I" and removal of scaffolding



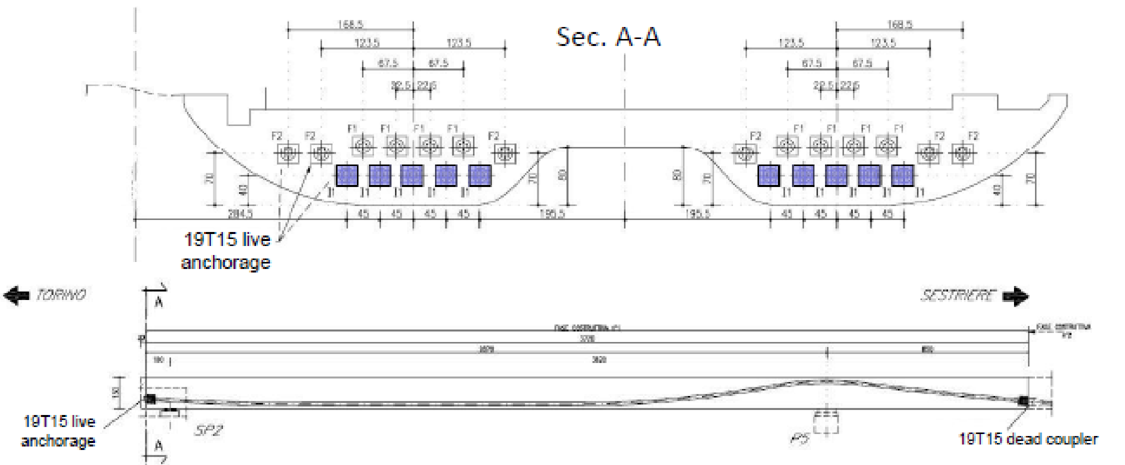
With the final tendons, axial force is jumping up to 35 kN and it goes down in the central region because it is stressed from the two sides and friction reduces its effect. New tendons increase

Construction phase tendons have different positions.

- At the first span, they start from a position at the beginning, they go down and up again to be coupled.
- At the second span, they start from the coupler and follow a similar path.

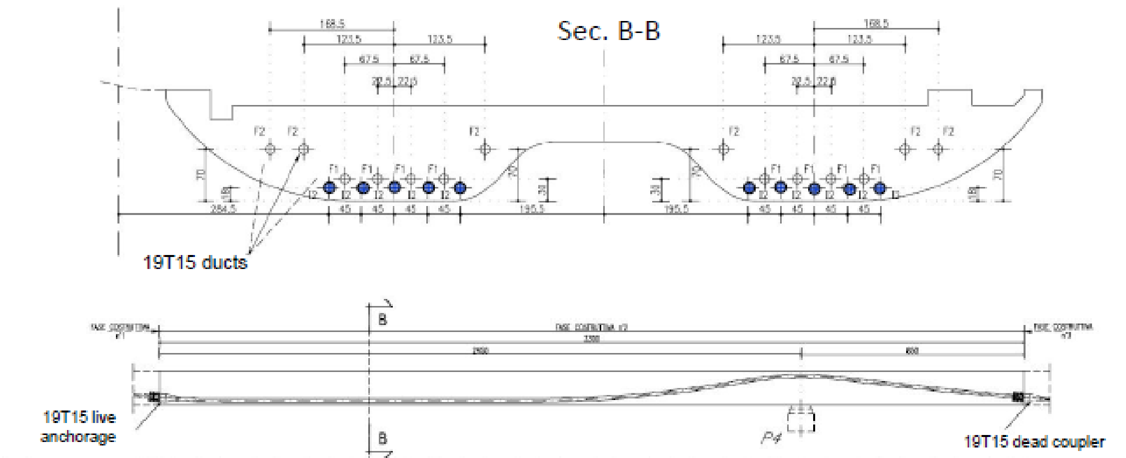
The geometry is the same in each span.

Phase 1: tendons I1



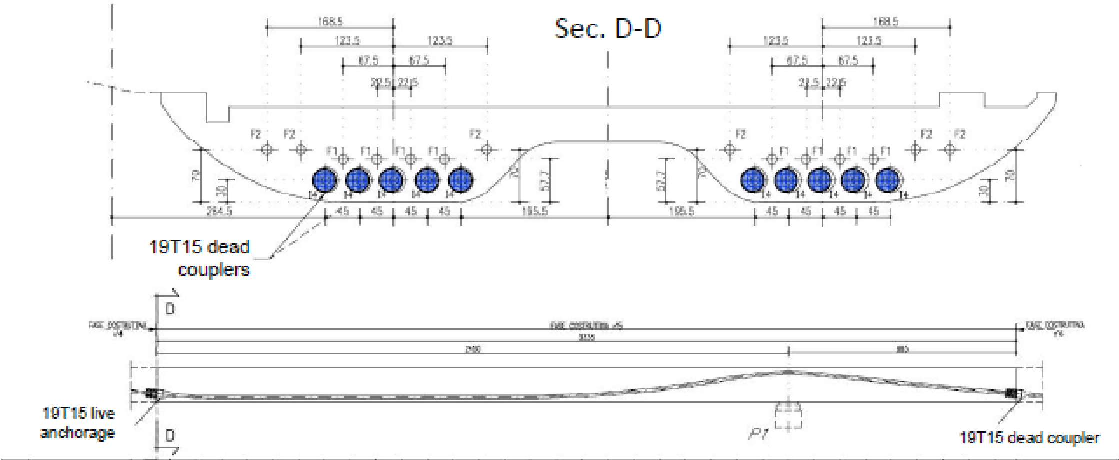
I1 X (cm) horizontal		70	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	50	The vertical Position Z of the tendon is taken from intrados
Z (cm) vertical	40.0	38.5	30.7	29.3	23.9	20.8	19.0	18.4	18.3	18.1	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.5	21.1	23.5	32.1	40.8	51.4	64.1	78.9	95.5	107.4	111.4	107.6	96.4	84.5	73.2	62.6	52.6	43.2	34.3	30.0	

Phase 2: tendons I2



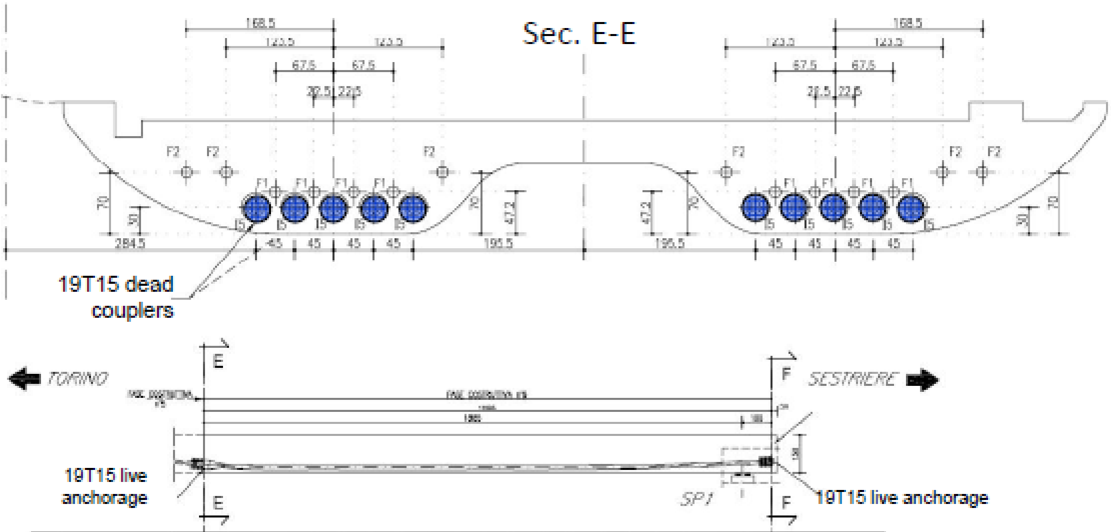
I2 X (cm) horizontal		50	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	50	The vertical Position Z of the tendon is taken from intrados
Z (cm) vertical	30.0	25.6	18.7	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	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 5: tendons I5



I5																																				
I5	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	85	The vertical Position Z of the tendon is taken from intrados
I5	Z (cm) vertical	30.0	29.6	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	86.8	75.8	66.5	56.7	46.3	37.4	30.0		

Phase 6: tendons I6

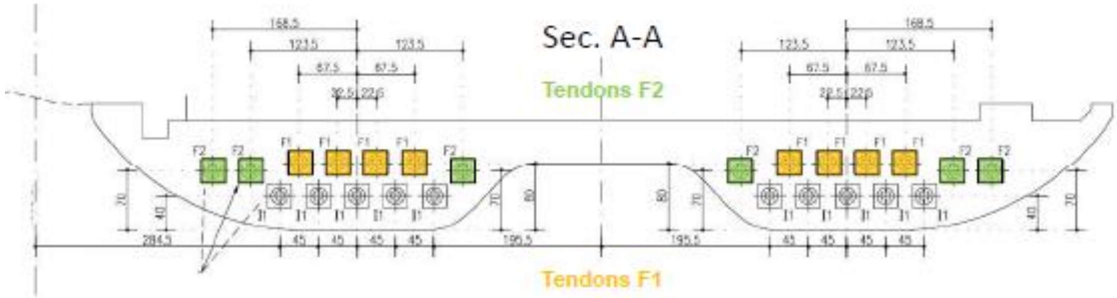


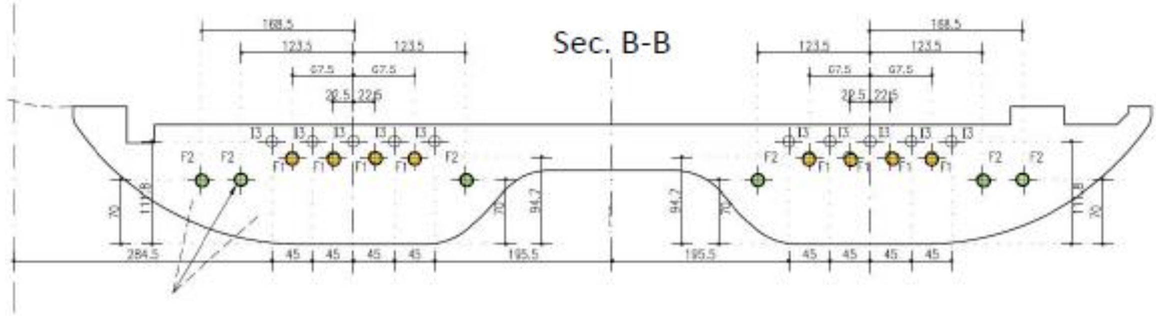
I6																								
I6	X (cm) horizontal		85	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	The vertical Position Z of the tendon is taken from intrados
I6	Z (cm) vertical	30.0	22.6	18.1	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.0	18.1	18.2	18.4	18.7	19.9	22.0	25.2	29.3	34.4	40.0	

The end of construction tendons are two different types.

- F1: they are four tendons moving up and down with the same philosophy of construction phase tendons, i.e. they go up where bending moment due to self-weight is bigger and down when bending moment due to self-weight is smaller.
- F2: they are three tendons, which are baricentric and perfectly straight. In this way, they provide only axial force, whereas bending moment does not increase so much.

End of construction: tendons F1 & F2





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ADVANTAGES

- Good speed of construction, as we use precast elements and they are ready for construction.
- Computation simplicity, related to isostatic scheme.
- Widely tested solution in term of rail traffic safety and passengers comfort.
- It is an advised solution by the railway authority.
- Practically no problem of interaction between track and structure.
- Necessity of accessibility from the bottom to the construction site (or

DISADVANTAGES

- Necessity of accessibility from the bottom to the construction site (or utilization of high dimensions and expensive launching girders).
  Indeed, we may use cranes to place in position the elements and cranes under the bridge and, to do this, we should deviate the river and build an artificial island on which the crane works. Of course, this will be a dangerous situation.

  The launching girder technique uses instead trussed steel elements to launch the girder above from the bottom, but it is quite an expensive solution.

- High number of bearings and joints, as each pier has two rows of bearings, i.e. 4 bearings for the previous beam and 4 bearings for the subsequent beam. Bearing service life is 15-20 years, whereas deck service life is 50-100 years. It means that, during service life, we have to change bearings at least 3 times and this is a cost.
- Large width piles and capitals to accommodate two rows of bearings (3.0 m, 4.1 m)
- Non optimal distribution of the stresses and low slenderness $L/h \leq 15$. This is not the beat solution, especially if we want to leave lots of clearance for floods.

HISTORY OF THE BRIDGE

Flooding in Piedmont in 1994 concerned principally Tanaro basin, with a flow measured in Alessandria of about 3800 m^3/s.

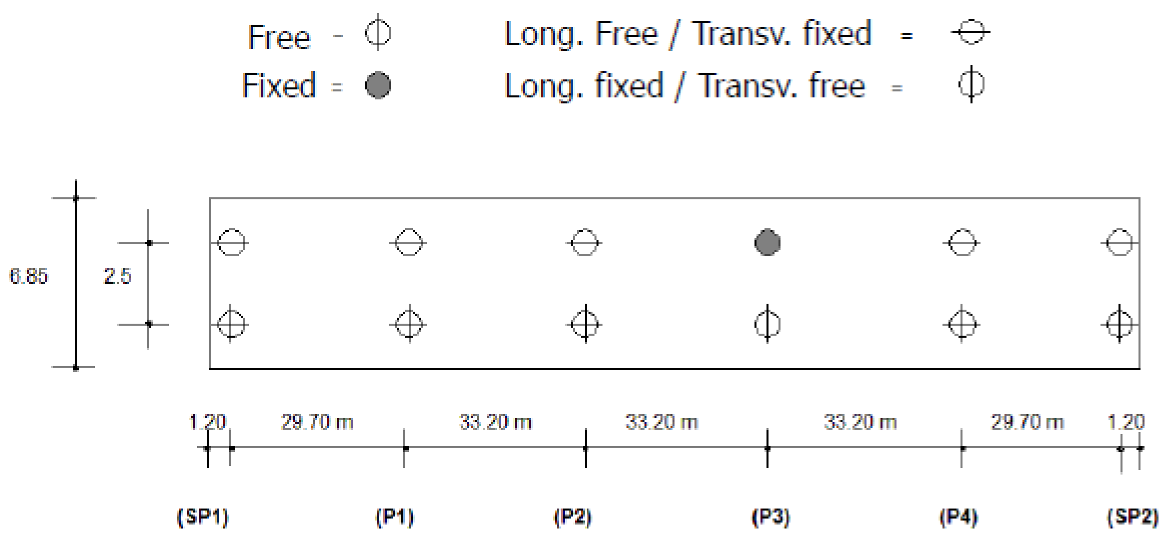
Old Corso Savona bridge in Asti was made of a upper way road deck, realized with 4 prestressed precast concrete beams with cast in situ slab, and a lower railway deck made of steel.

The span was 20 m and both the decks were supported by huge masonry piers that left very little free span between them.

By consequence, during the flooding the bridge presented different problems.

- Insufficient hydraulic clearance: water reached the intrados of the prestressed concrete deck.
- Violent impact of transported material against the upstream beam, as fluid had high density due to presence of mud and transported rocks rolling in water and moving with a velocity of 3÷5 m/s, i.e. they were really big bullets, which hit the bridge and damaged beams.
- Drifting of material against the piles with consequent dam effect.

During post-flooding repair works of river Tanaro, the river bed in correspondence of the bridge has been enlarged in order to leave more space for the flooding.



- Free bearings, providing only vertical reaction.
- Longitudinal free and transverse fixed bearings. We can notice that each pier follows the same scheme of Pinerolo bridge, with free longitudinal movement and transverse restraint.
- The fixed bearing is placed on an internal pier and not external in order to reduce movement at the beginning and at the end of the bridge. Indeed, total displacement in case of internal fixed bearing is small than the one in case of external bearing, as displacement is split on the two sides – it is not concentrated in one side – and it is reduced.
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CONSTRUCTION TECHNIQUE: INCREMENTAL LAUNCHING

As regards construction technique, incremental launching is adopted because it is a good system for spans between 20 m and 80 m and total span between 200 m and 600 m. Typical velocity is 15÷30 m/week.

The construction of one span, i.e. 33 m, takes 10 days, which is much smaller than 45 days, as incremental launching consists of realizing the beam on the ground level, with comfortable access to the scaffolding. When span is ready, it is pushed forward and leaves the space to build the other parts. Launching time is just 3 hours, i.e. beam proceed with a speed of 10 m/h.

NOSE DESIGN

Nose is used because, when deck is pushed, deck is working with a cantilever scheme, until it does not reach the next pier.

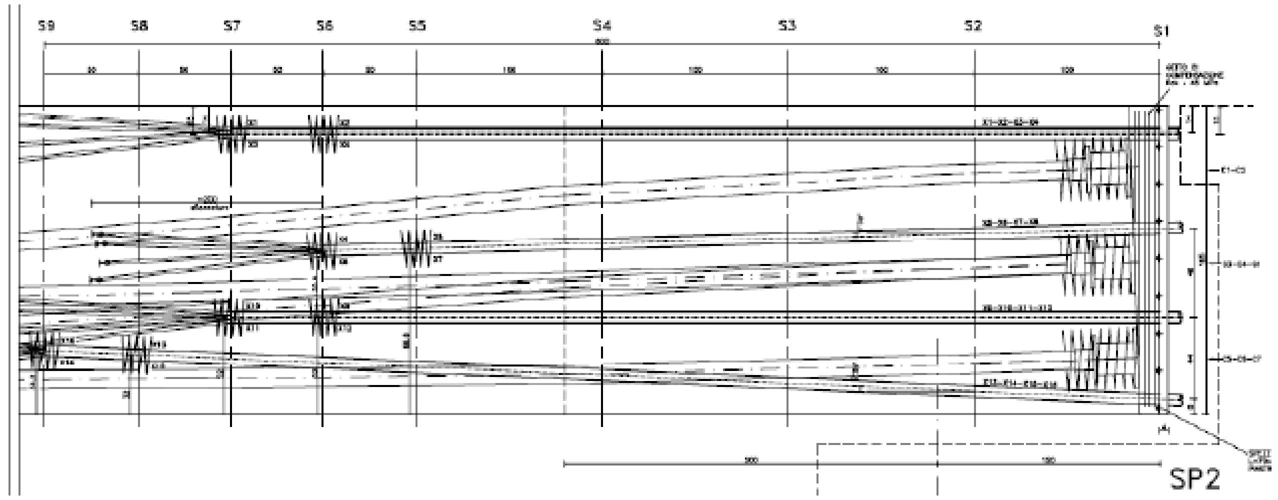
In the cantilever scheme, the available weight of the cantilever is limited and, as beam is a massive thick concrete element – heavy structure –, we use thin steel element to reduce cantilevering weight.

Nose is removed at the end of construction, to be reused on an other construction.

Form the economical point of view, nose is convenient in case of long bridges, as cost is distributed on long time, many similar bridges or by adapting an old nose on the new bridge, in order to let it work.

The launching nose is placed on the first section of the deck and connected to it through anchoring cables.

Tendons are very short and anchoring of prestressed tendons is given through live anchors inside the bridge because they connect the nose and push it against the deck, when prestressed. Cables are used only during the launching and then nose is unlocked and they are left there, without working anymore.

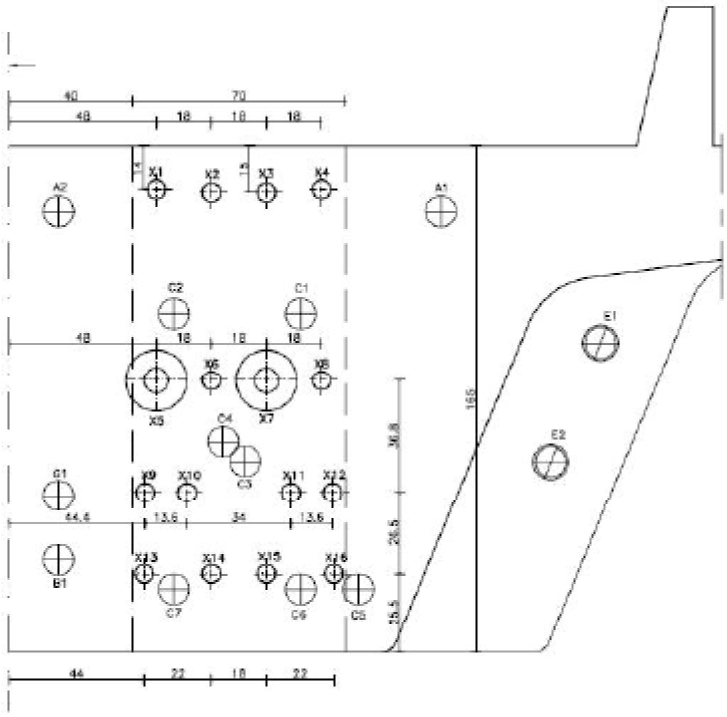


Looking at the section of the interface with the nose in the front view, there is an iron plate, which is 70 cm wide and 165 cm tall, having four rows of four holes. Each hole has one tendon inside connecting the nose to the bridge, with a total of 16 tendons.

The plate is thick and, on the surface on the concrete side, it has presents ribs ("costolature"), realized by welding ϕ20 steel bars to the plate. Ribs create an interlock between the plate and concrete, granting shear transmission between the plate and concrete.

On the other side, surface is smooth because the plate is coupled with an other plate welded to the nose and shear forces pass to the plate by means of tendons – this is the only system joining the plate on concrete and the plate on the nose. By consequence, we need prestressing tendons in order to create friction and no to avoid the nose sliding from the deck.

The big round elements in the section are the anchoring heads of tendons used in the bridge.

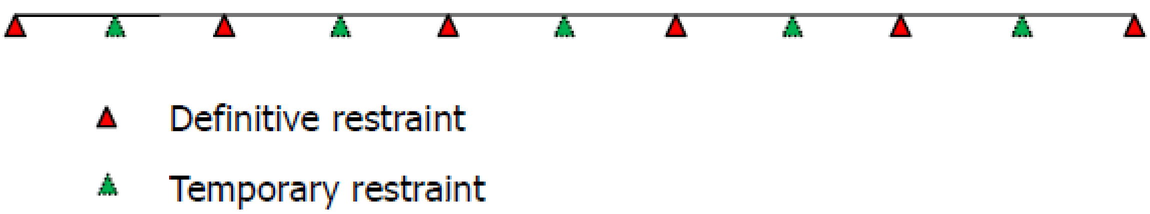


Section S5

(4m from the nose)

EVALUATION OF THE INTERNAL ACTIONS DURING LAUNCHING AND LAUNCHING PRESTRESSING

Static scheme is characterised by two types of restraint.



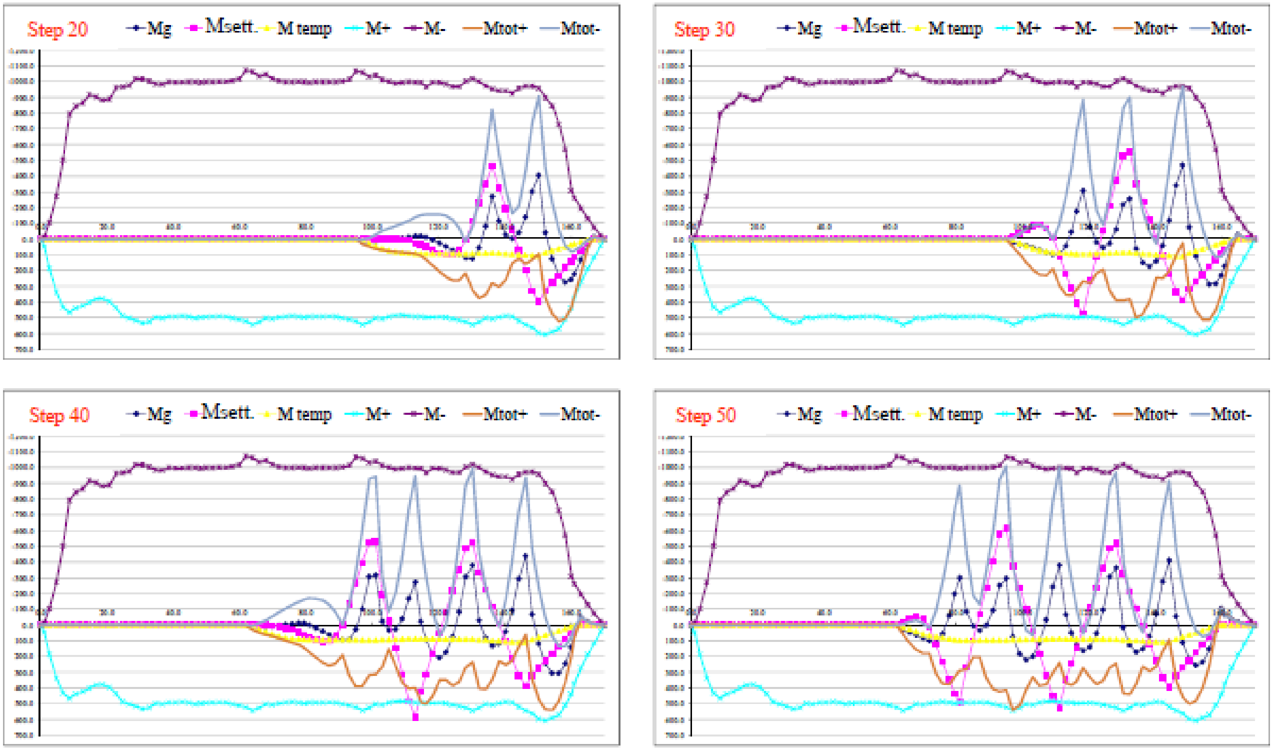
- Definitive restraint, corresponding to piers and abutment present during service life.
- Temporary restraint, corresponding to additional piers used only during launching stages. These are small piers built like piles bored in the ground and dismounted at the end of construction. They have been introduced because, in the launching, deck is working with a cantilevering scheme, which generates bigger internal actions than the continuous beam ones. For instance, maximum bending moment – bending moment at the extremity – of a cantilever subjected to a distributed load q is

$$M_{cant} = \frac{ql^2}{2}$$

Maximum bending moment – bending moment at the mid-span – of a simply supported beam subjected to a distributed load q is

$$M_{SSB} = \frac{ql^2}{8}$$

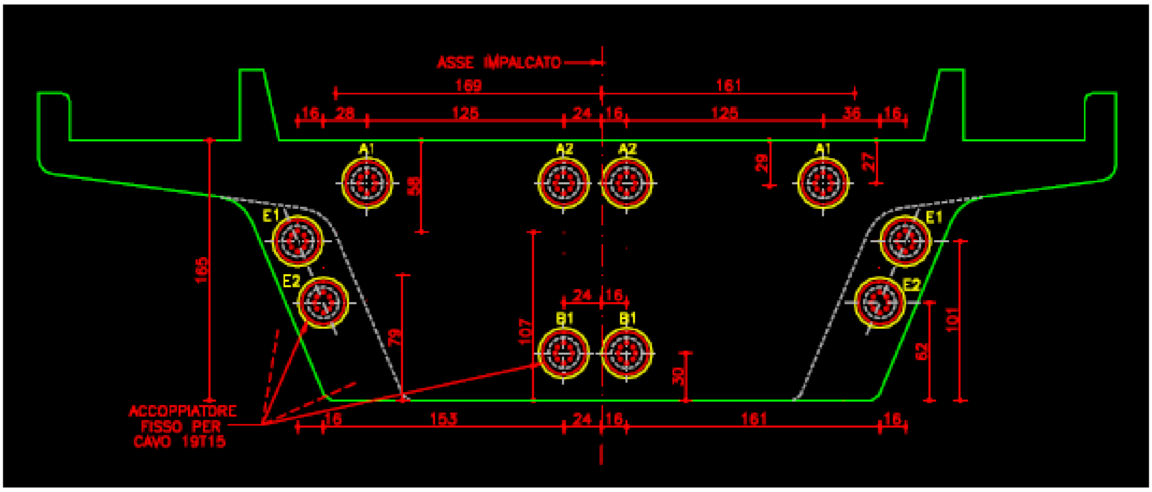
Maximum bending moment – bending moment at the mid-span – of a simply supported beam subjected to a distributed load q is



In the plot, we identify different curves.

- Curve Mg, representing bending moment due to self-weight.
  This is bending moment due to a uniformly distributed load on a continuous beam, supported every 15 m by temporary and permanent piers. Shape is not constant during construction phases because static scheme changes and the presence of the nose brigs a variation of bending moment.
- Curve Msett, representing bending moment due to differential settlement of one pier with respect to the other ones.
  This bending moment has a straight with “zig-zag” shape and shape is not constant during construction phases because static scheme changes and the presence of the nose brigs a variation of bending moment.
  Forcing a settlement of one pier in a continuous beam leads to a “zig-zag” shape of bending moment, but this is big in the point of application of perturbation and gets smaller as far as we move far from that point, since perturbation is going to die.
  The envelope of bending moment due to relative settlement is almost constant because it takes into account different settlement effects and it assumes a big value, as we are considering a settlement on each bearing. By consequence, maximum effect arise from the different settlement of beams.
  Thus, considering a differential settlement of 5 mm, if beam i is fixed, beam $i+1$ is 5 mm below or above – a beam is moving, whereas the other one is fixed.
- Curve Mtemp, representing bending moment due to thermal variation, assuming temperature variation between intrados and extrados of ± 5°.
  This bending moment has a trapezoidal shape and the value starts from zero and grows up linearly, up to a maximum in correspondence of the first pier. From there, it keeps a constant value up to the last pier and then decrease to zero. This is the typical shape of bending moment due to thermal effects in a continuous beam.

Prestressing tendons used during the launching stages are baricentric, as the bending moments are almost constant in all the sections and the positive values are only half of the negative ones.



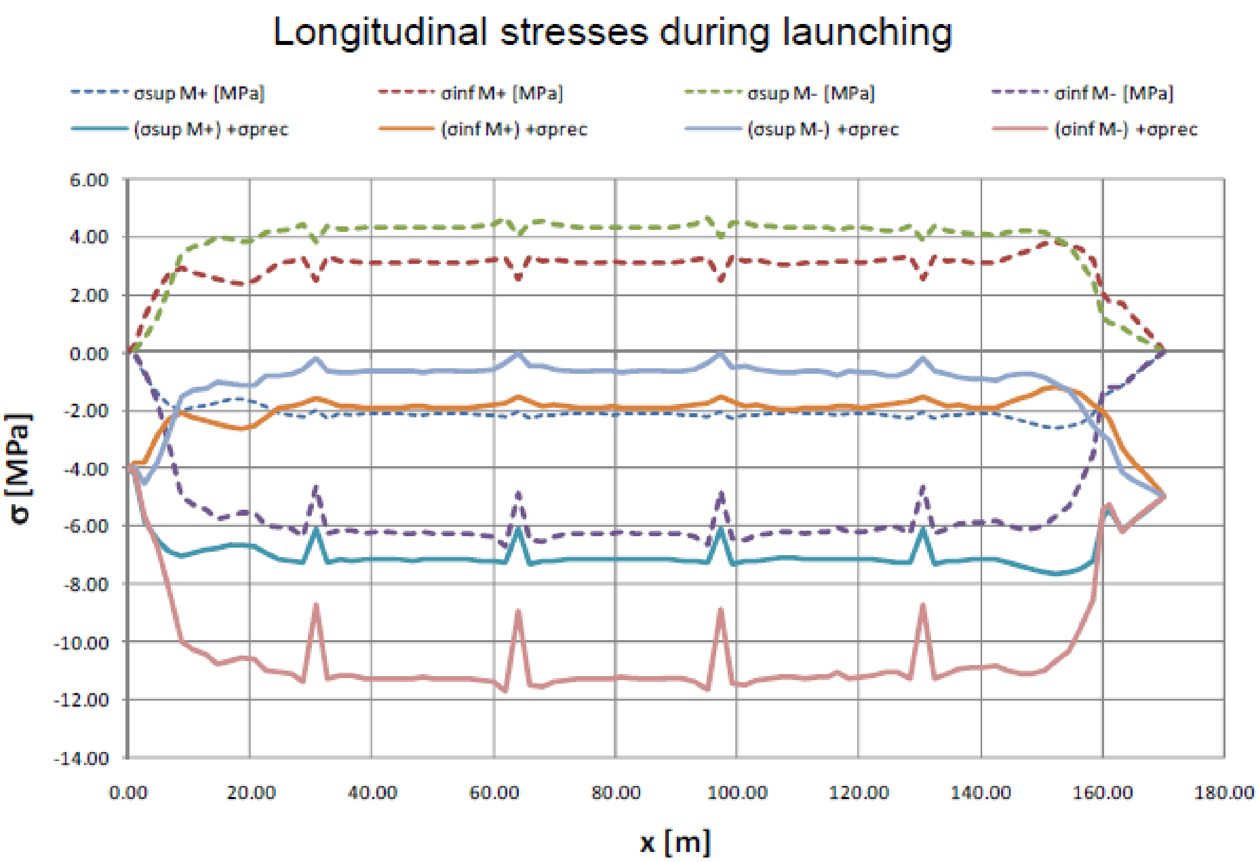
Enlarged section

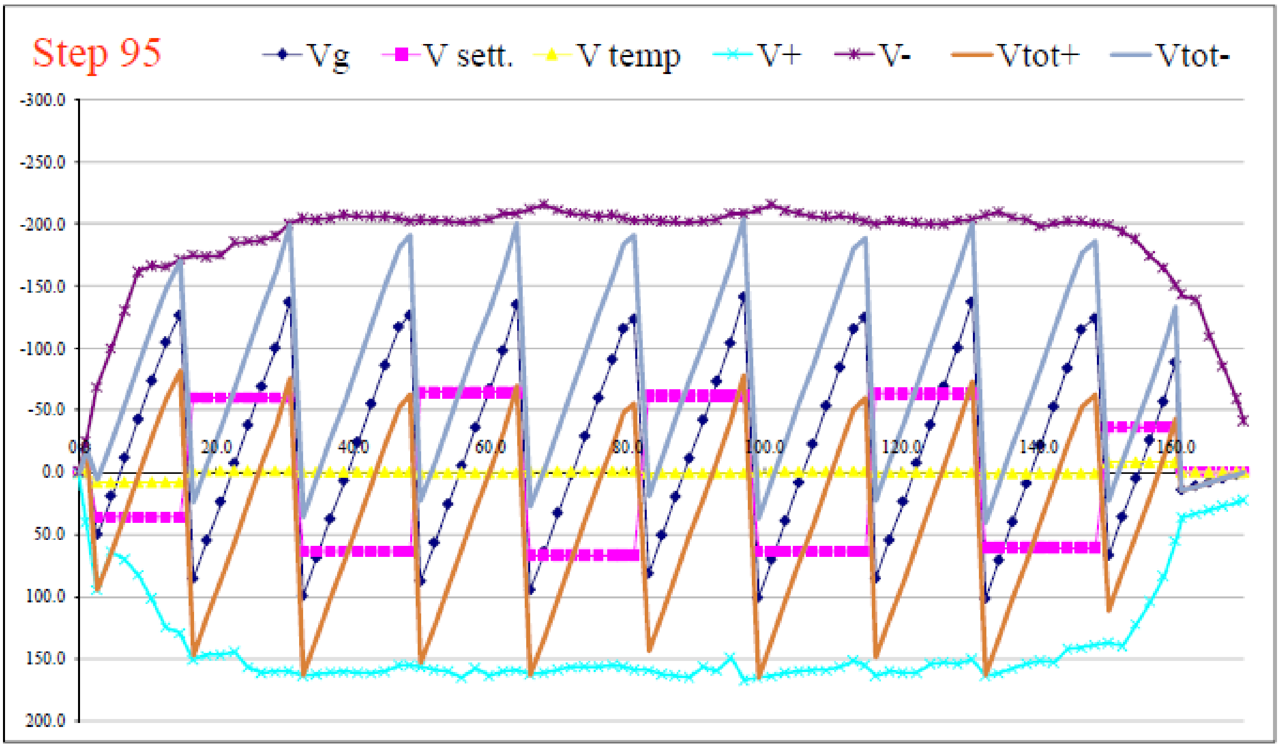
A $[m^2]$	$W_{sx,sup}$ $[m^3]$	$W_{dx,sup}$ $[m^3]$	$W_{sx,inf}$ $[m^3]$	$W_{dx,inf}$ $[m^3]$
7.897	-2.828	-2.631	2.171	2.237

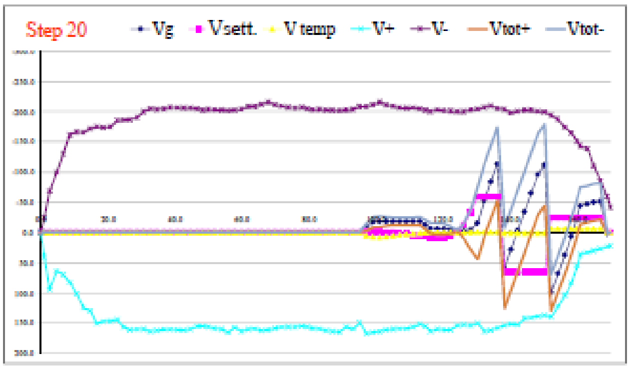
Current section

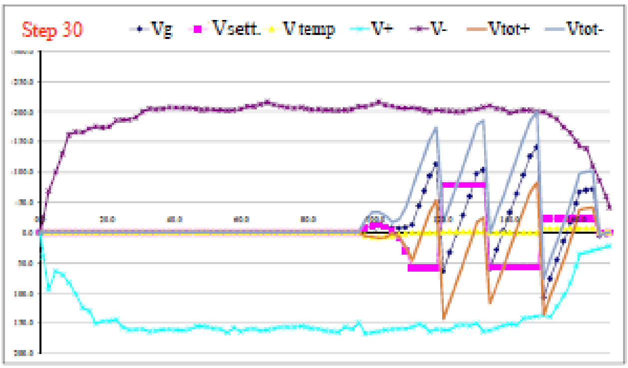
A $[m^2]$	$W_{sx,sup}$ $[m^3]$	$W_{dx,sup}$ $[m^3]$	$W_{sx,inf}$ $[m^3]$	$W_{dx,inf}$ $[m^3]$
6.458	-2.498	-2.290	1.590	1.629

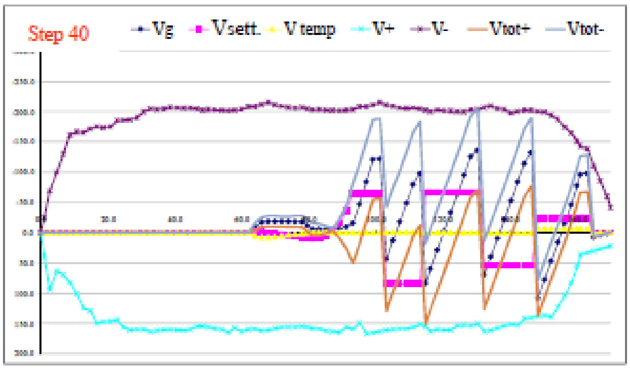
Focusing on stresses, during launching we can identify different couples of curves, corresponding to different positions in the deck section.

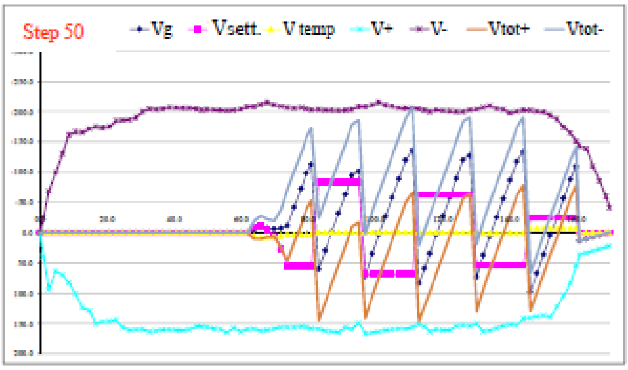












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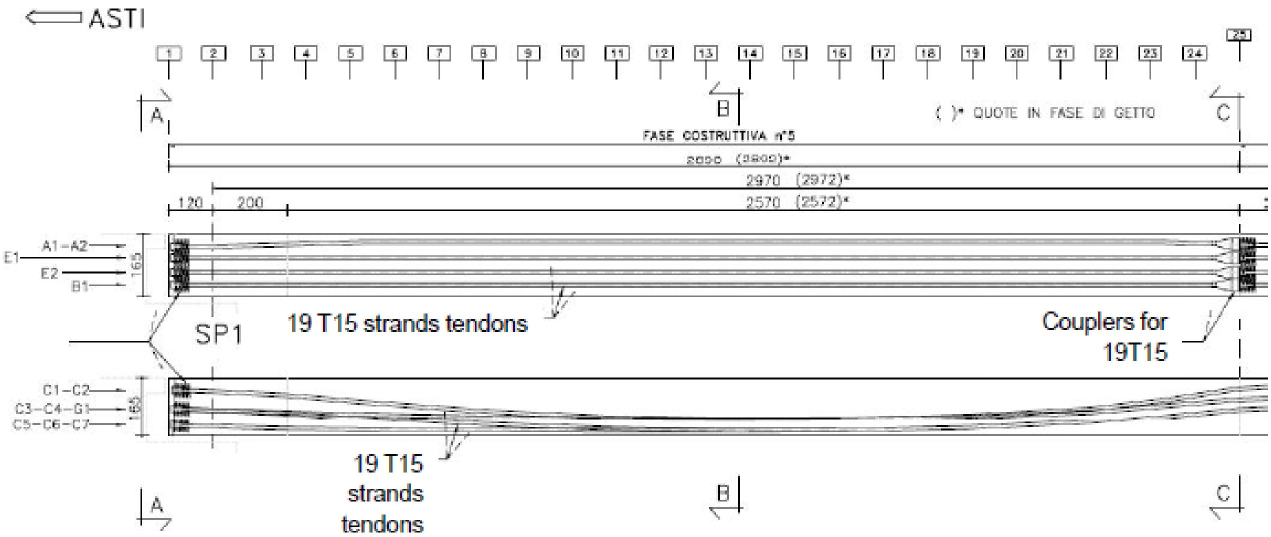
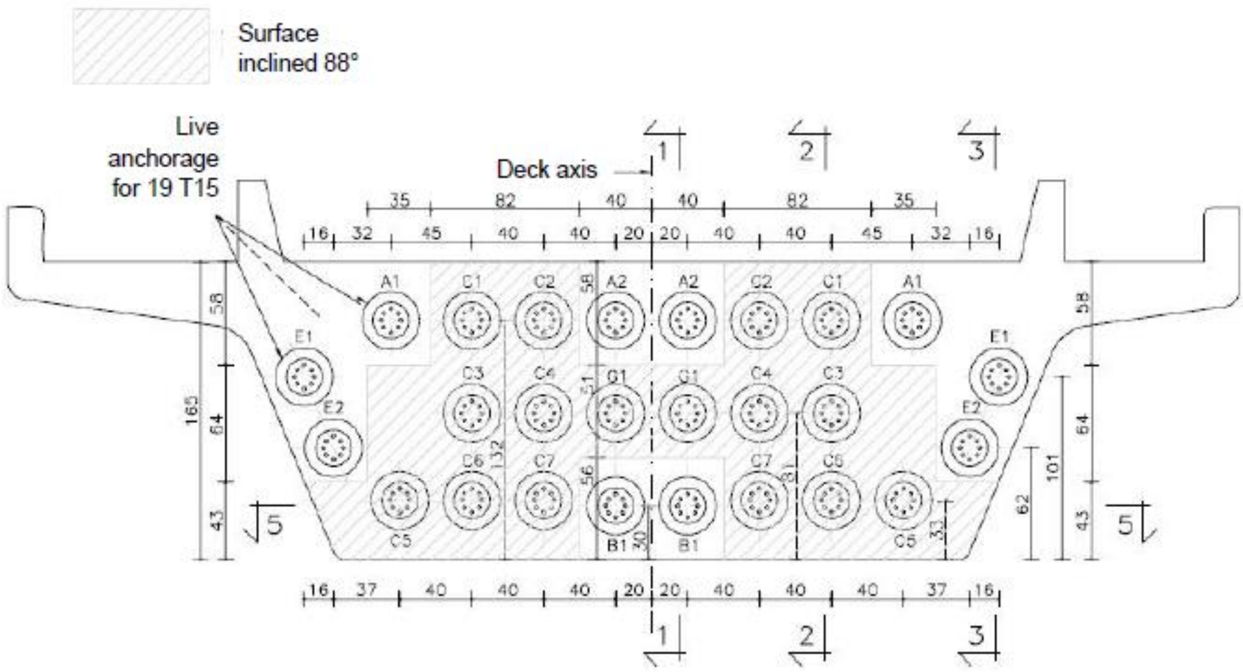


Prestressing layout presents also different couples of curved tendons – 19 strands tendons.

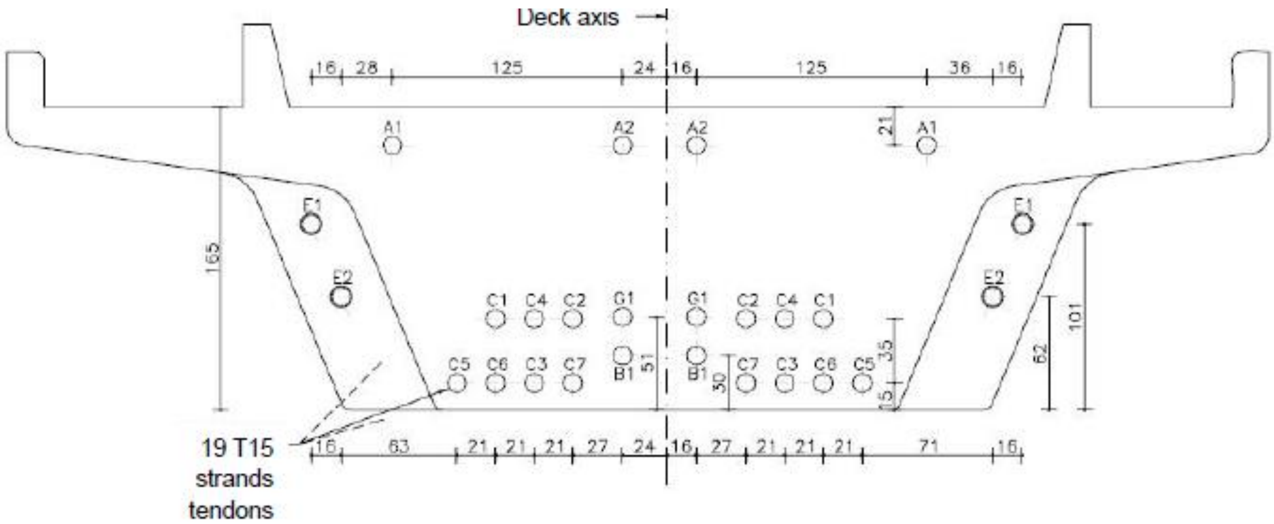
- Tendons C1 and C2.
- Tendons C1, C4 and G1.
- Tendons C5, C6 and C7.

They are not constant in the vertical position, but they move up and down.

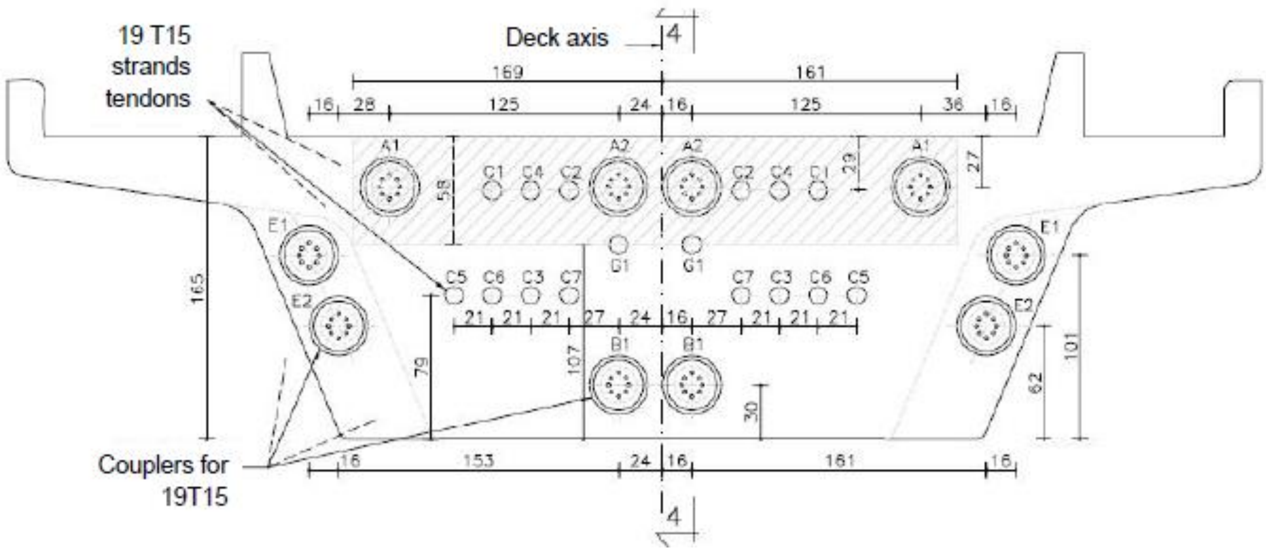
We can see them in the cross section AA at the beginning, in which the dashed areas are not a vertical surface but an inclined surface as, in this section, tendons are going down. Tendons A, B and E are going straight, whereas tendons C and G are inclined.



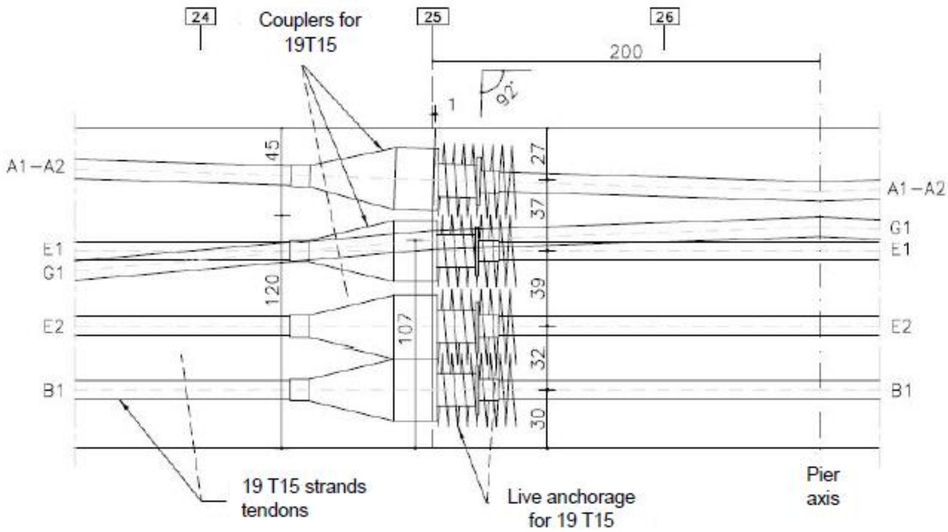
Looking from the top, pier section is bigger in order to live room for anchoring heads. Then tendons deviate in the horizontal plane, in order to enter in the current section, i.e. they move in the vertical plane and squeeze in the horizontal plane.

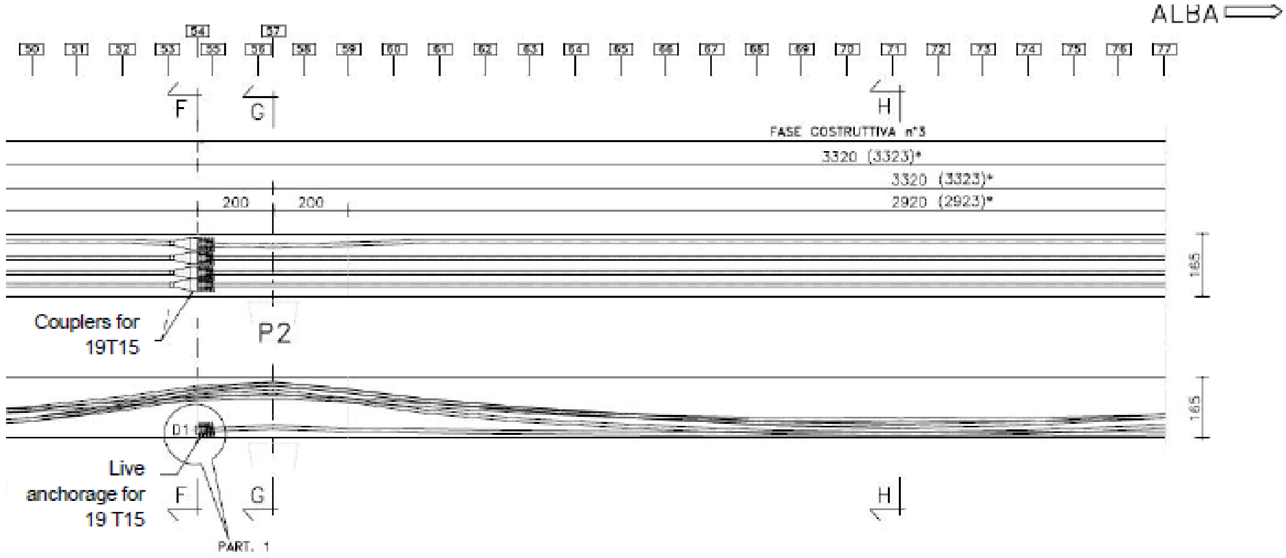


In the cross section CC in correspondence of the first pier, construction tendons are coupled and the other tendons are running through, since they are placed at the end of construction. They occupy the top region, in order to give positive bending moment and counterbalance negative bending moment due to self-weight.

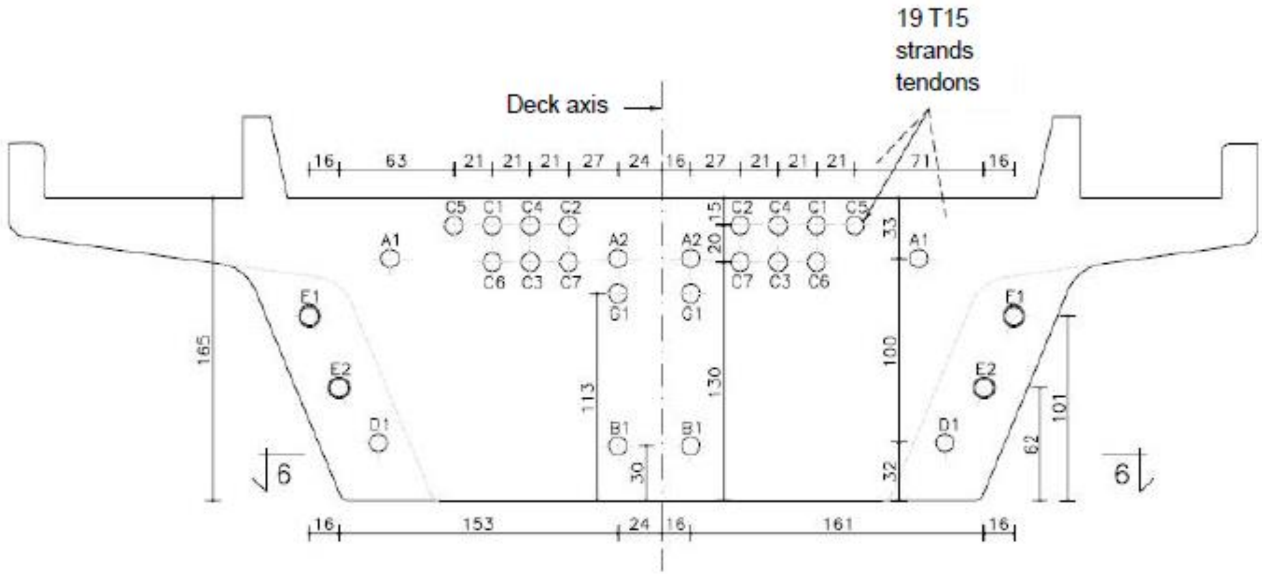


Seeing from the top, we can notice some tendons coupled and other tendons running through.





Cross section 66 shows how tendons are coming inside the bridge and placed on piers – anchorage in a big section.



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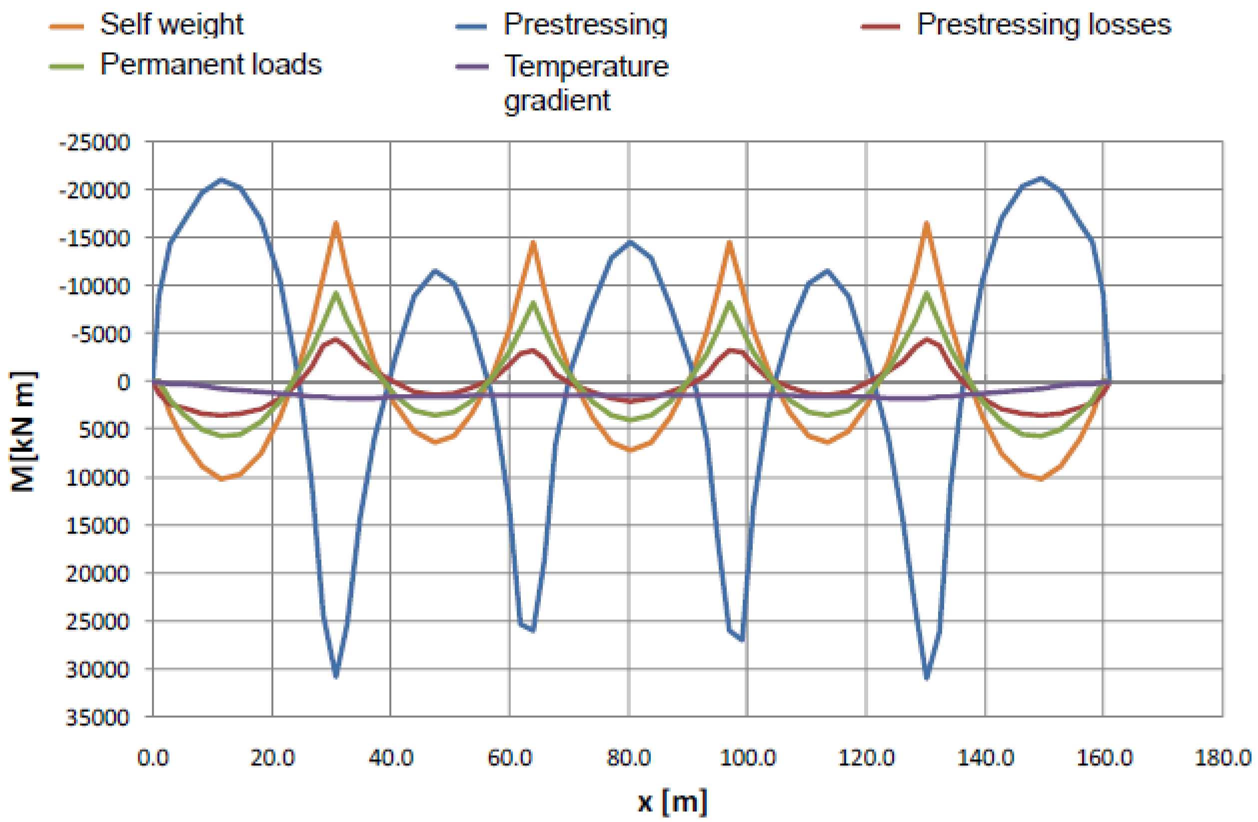
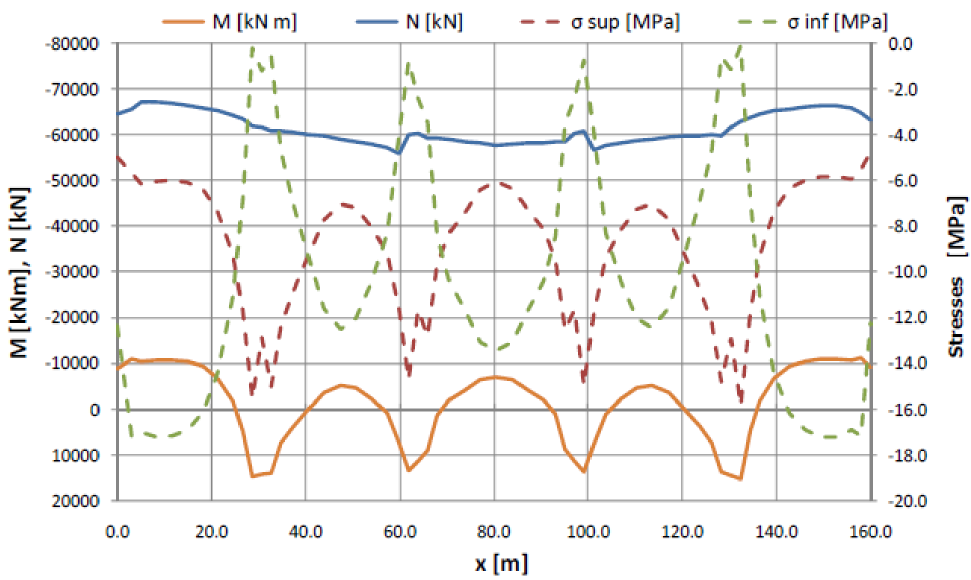


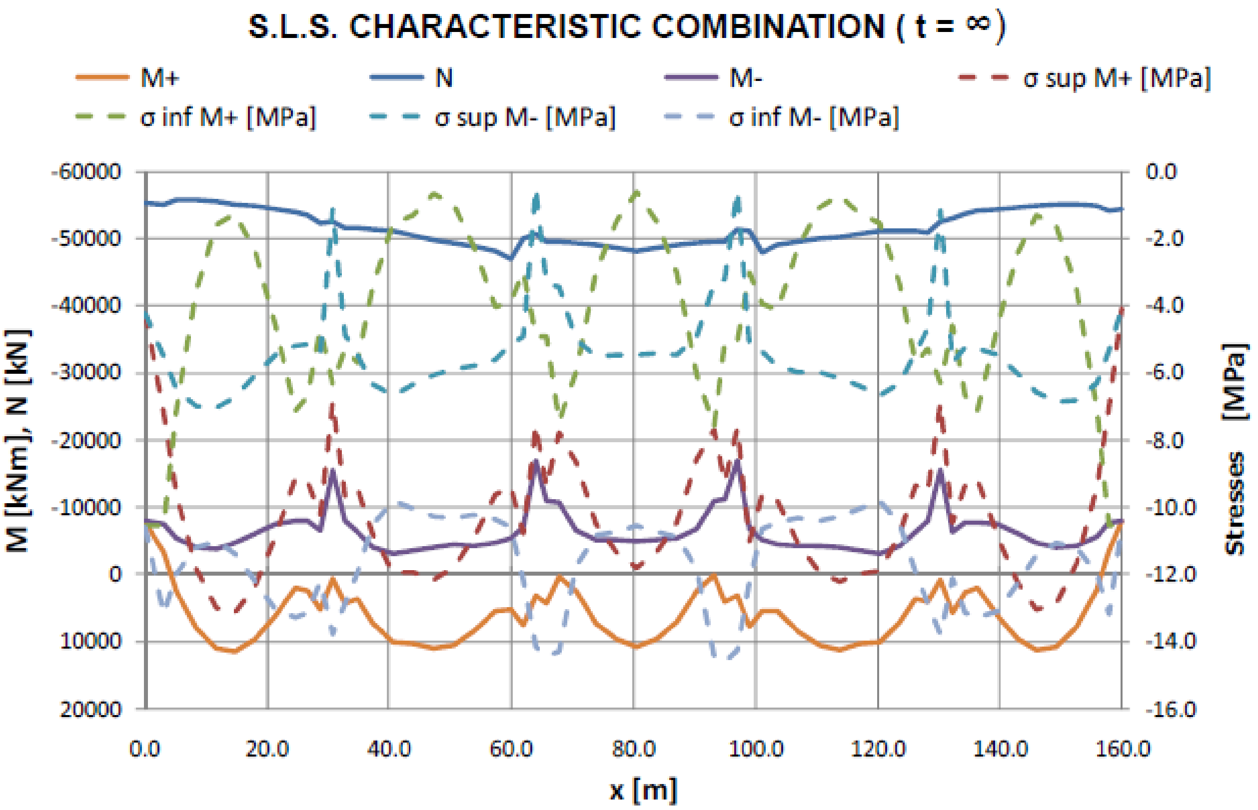
Then, we evaluate stresses induced by prestressing.

- At the end of construction, axial force N due to prestressing is going down due to friction and it increases in correspondence of the central pier because of additional prestressing induced by the two additional tendons.
  We can notice a high value of axial force and small bending moment, as a lot of tendons are baricentric, depth is small and internal lever arm is about 0,6, i.e. bending moment is given by axial force divided by 2, if all tendons are up or down. As many tendons are constant, bending moment will be much smaller than axial force.
  In this way, this is a heavily compressed bridge with a little bending moment and all stresses are compressive.

DEFINITIVE PRESTRESSING



Also stresses in the top region and bottom region of the section are evaluated.



In the SLS characteristic combination, we have the effects due to self-weight, permanent loads, creep and shrinkage and also traffic.

In case of no traffic, the minimum and maximum bending moments are close to each other. With traffic, they are different because traffic is a load moving on the bridge.

The axial force is the same, as it depends only on prestressing.

Stresses now are varying with bigger variations than before.

ULS VERIFICATIONS

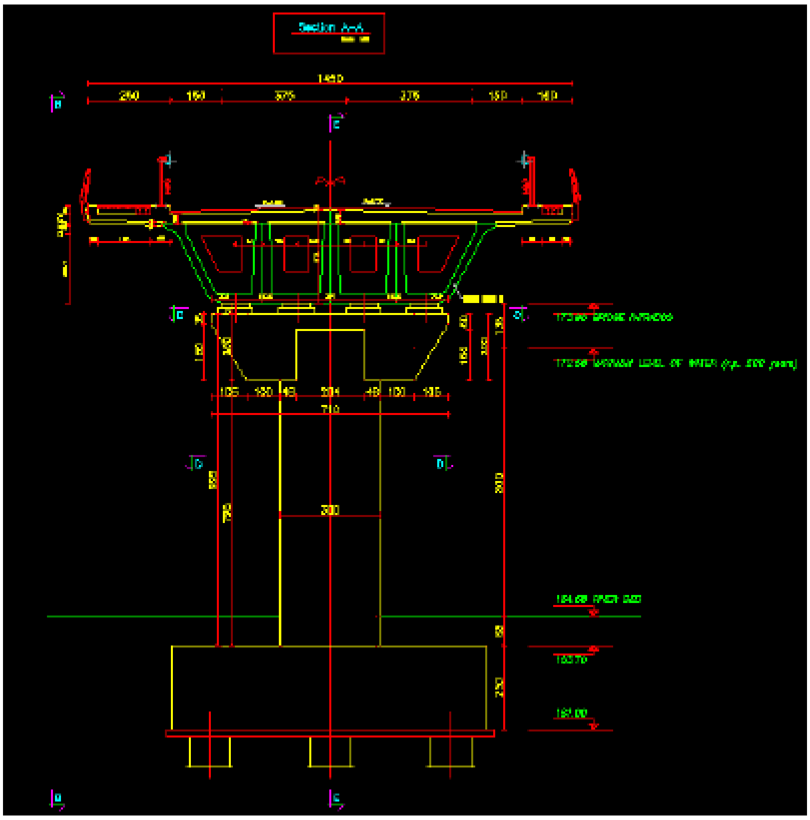
Bending moment at ULS is described with an unique curve, as the negative part is taken from the negative envelope and the positive part is taken from the positive envelope – we plot one envelope or the other only where it is needed.

$$\rho_l = \frac{A_{s,l}}{b_w d}$$

$$k_1 = 0{,}15$$

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

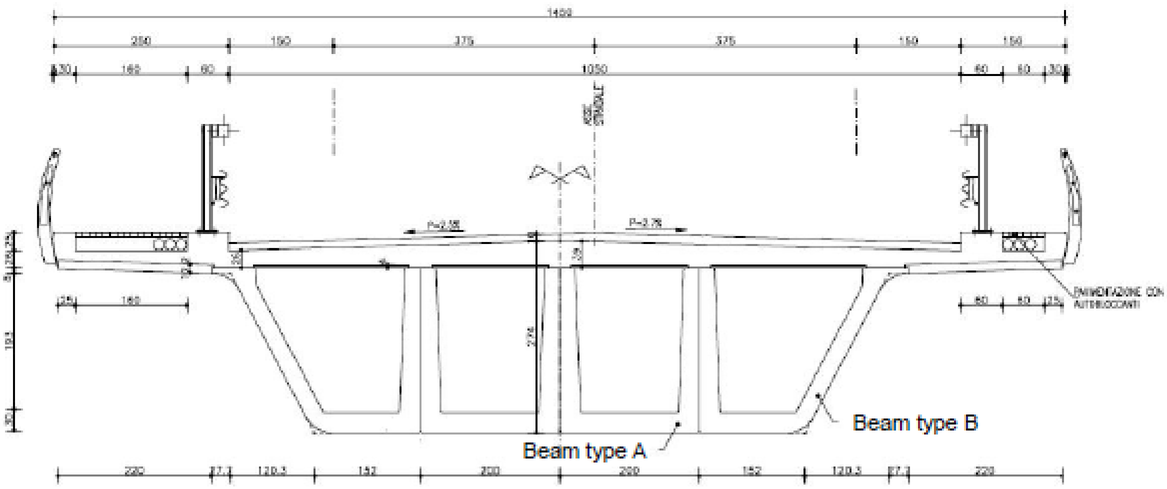
In this case, the bridge is able to resist to shear action even without stirrups. The reason is that beams are thick and solid body, with a lot of concrete, and tend not to have problems with shear.



Typical cross section on a pier

Focusing on the deck cross section, we can identify two different kinds of beams.

- Squared beams.
- Inclined beams.



BEARINGS

As regards bearings configuration, they are associated with a code defining the degree of restraint exerted by them.

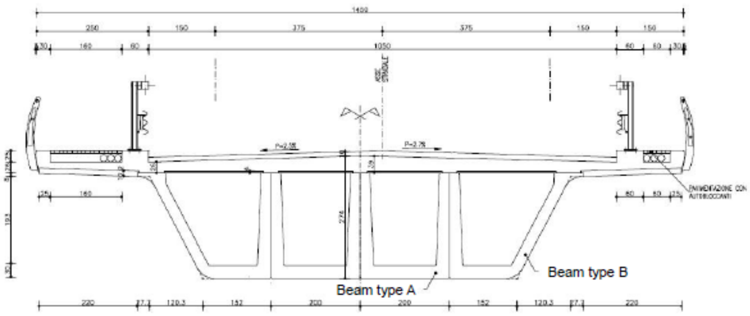
- F stands for fixed movement in both directions.
- M stands for free movement in both directions – the bearing provides only vertical reaction.

The deck presents also transverse beams, placed in between the different parts of the bridge.

BEAMS GEOMETRY AND PRESTRESSING LAYOUT

The deck is made of precast elements which are 19 m long and 2,35 m tall.

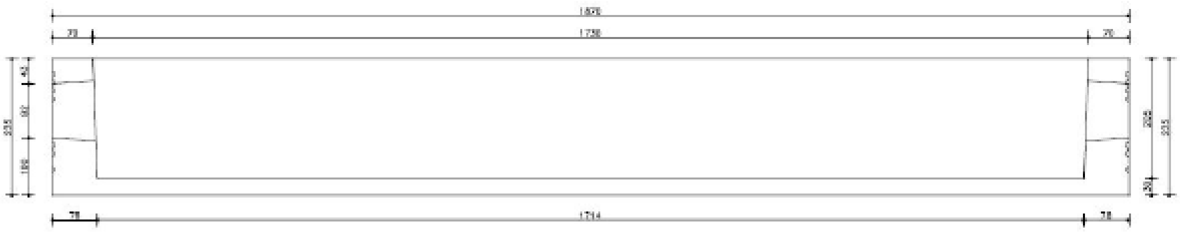
Internal beams (beam A) are 2,35 m tall and 2 m wide, whereas external beams (beam B) have an irregular shape and they are 2,35 m tall and 3 m wide.



These beams were not so common in transportation but they managed to because cross section is not so big to have problems with the shape of the lorry – they section is inside the shape of the lorry and beam is only long, but not tall or wide.

For each cross section, they realized two different elements.

- Span elements, having two diaphragms and they are empty in the middle.

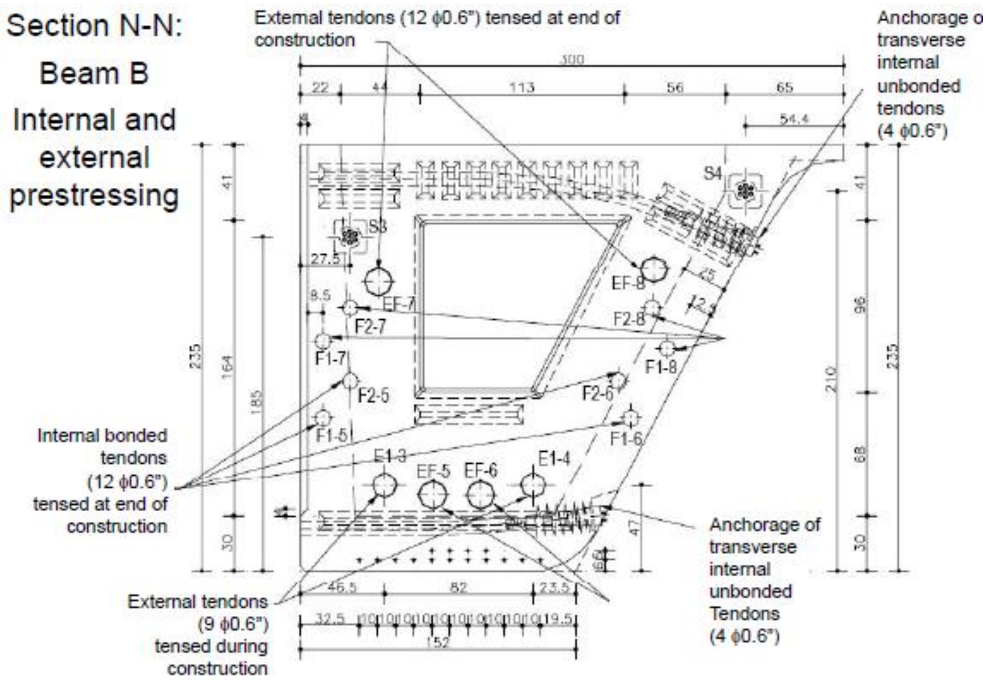


- Pier elements, having three diaphragms.
  They need a third diaphragm because, when we assemble elements together, piers elements lie on the pier axis and they are coupled with two span elements.

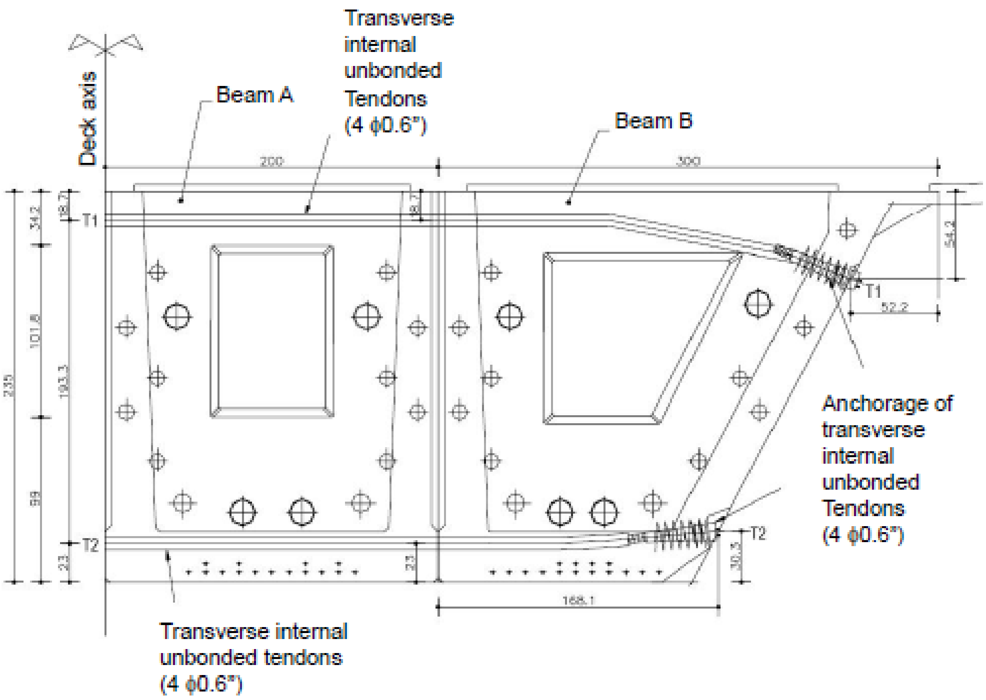
The deck before the casting of the top slab and the disposal of the predalles has reinforcement bars going up to join together predalles and beams. At the bottom, we can see the tubes containing tendons and the port-halls.

In the joint, there are two diaphragms and, between them, from outside we can see a little spacing, which will be cast in situ. The space is crossed by tendons used to prestress the transverse beams. In detail, three tendons are in each precast diaphragm and two tendons are in the cast in situ part. Before concreting, in the space between precast beams, we can see the reinforcement of transverse beams and the cover for the anchoring head. The space is transversely crossed by a plastic tube, which is the duct for longitudinal tendons.

The reinforcement of the top slab is an array of orthogonal bars supported by predalle elements and connected to the longitudinal beams through U-shaped hooked joints.



Typical transverse beam section



- Prestressing strands F1 and F2.
- External prestressing tendons E1 and EF, which are tendons running inside the box section but they are not immersed in concrete.
- Internal bonded tendons S, present since the beginning of construction in some sections.
  All piers elements have an additional prestressing on the top – S1 and S2 from the square section, S3 and S4 form the inclined section – due to negative bending moment.
- Transverse prestressing tendons, which are internal but unbonded, since there is no bondage between the duct and concrete. Indeed, there is an external duct bonded in concrete and an

MATERIALS

- Concrete
  - Precast beams
$$R_{ck} \geq 55\ MPa$$
  - Cast in situ (slab, transverse beams etc.)
$$R_{ck} \geq 45\ MPa$$
- Prestressing steel with 0,6'' strands – standard strands for prestressing. Their area is 1500 mm^2, which is not the actual area but a given number, as a strand is not full section but has sparrels.
$$f_{pt,k} = 1860\ MPa$$
$$f_{pt1,k} = 1670\ MPa$$
$$E_s = 210000\ MPa$$
- Ordinary steel Fe B 44 k, described with the indications of the old codes. The main difference with respect to new codes is the yielding stress, which was assumed smaller in design.
$$f_{yk} \geq 430\ MPa$$
$$E_s = 200000\ MPa$$
$$f_{yd} \geq 374\ MPa$$

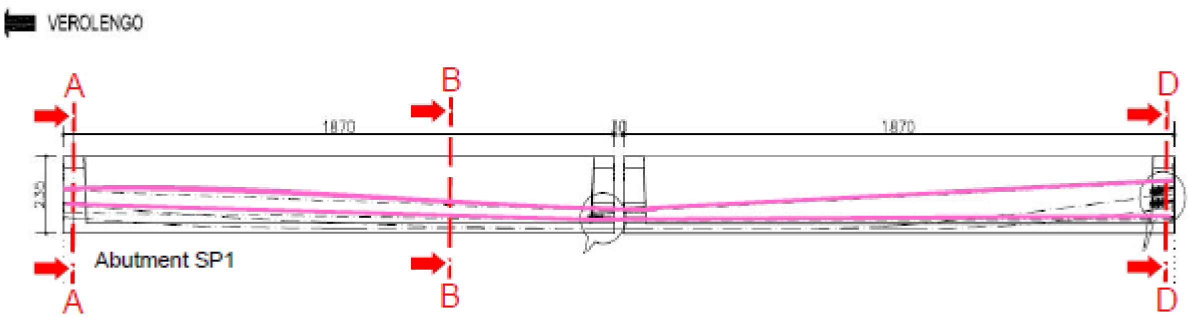
The elements have different cross sections, in terms of geometric area $Area\ 1$, homogenised area $Area\ 2$ on the concrete of the precast beams, main longitudinal bending inertia J_3, transverse bending inertia J_2 and centroid distance from deck intrados Y_g.

	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
Beams alone	6.58	6.58	3.54	39.20	0.93
Slab	3.63	3.28	0.02	45.55	2.53
Beams + Slab	10.21	9.86	9.16	84.75	1.46

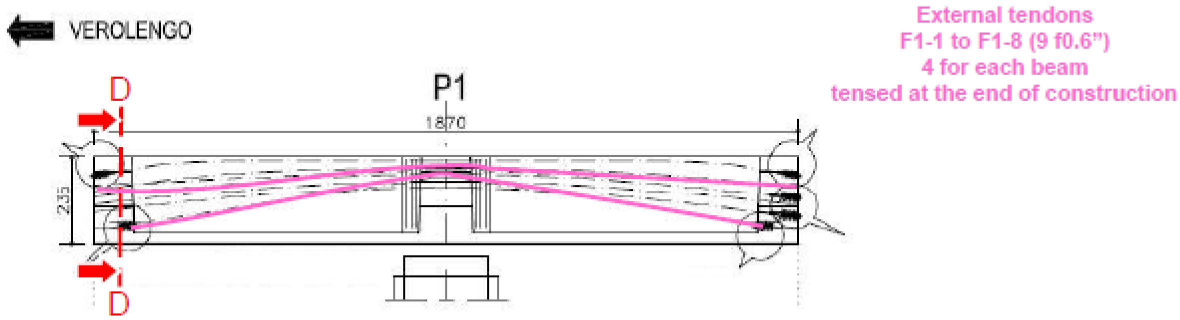
On beams A and B, the geometric area and the homogenised area are equal, since no homogenisation has been performed.
In the system A + slab, for instance, as slab is made of a different material, the two values will be different because the homogenised area is homogenised on the concrete of the precast beam.
Pure geometric area is used to compute self-weight, whereas homogenized area is used to compute stiffness.

PRESTRESSING AND ORDINARY REINFORCEMENT

We have a better view of prestressing layout, in which F1 is made of two different tendons – actually, there are four tendons.



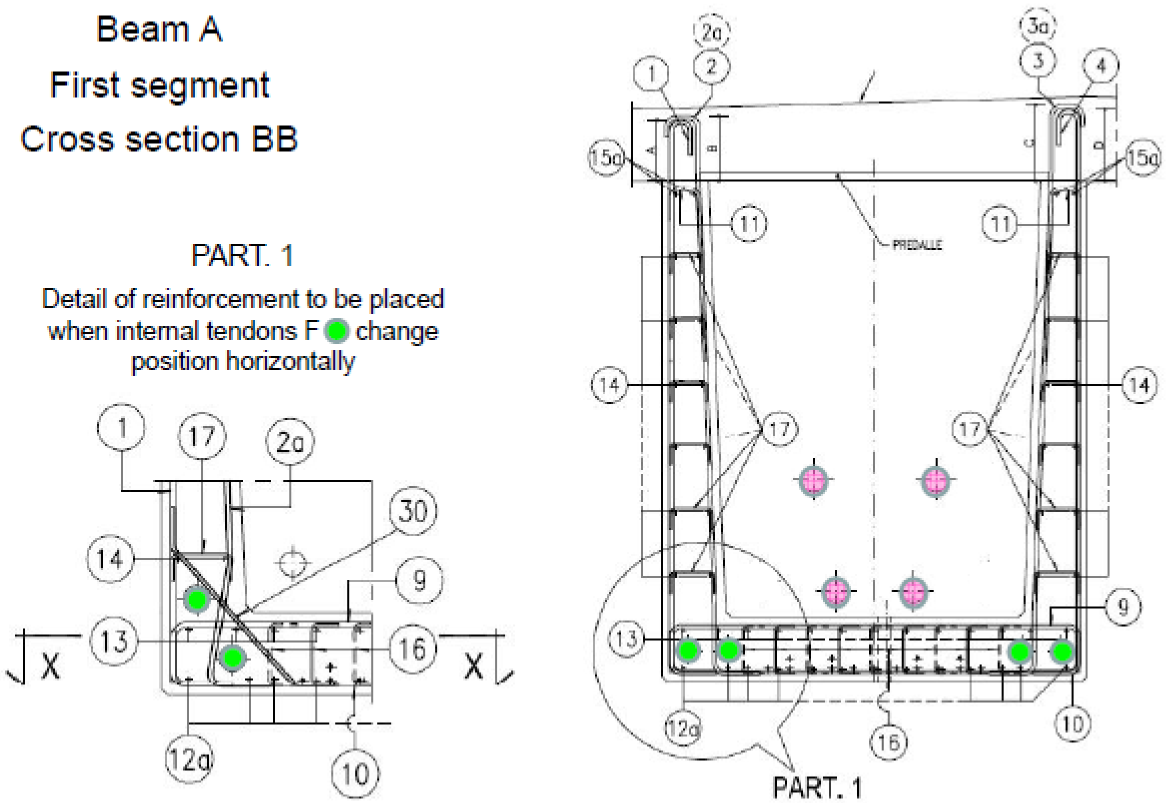
Prestressing of beam A: segment 3 – END of construction



We analyse the position of tendons in the cross section.

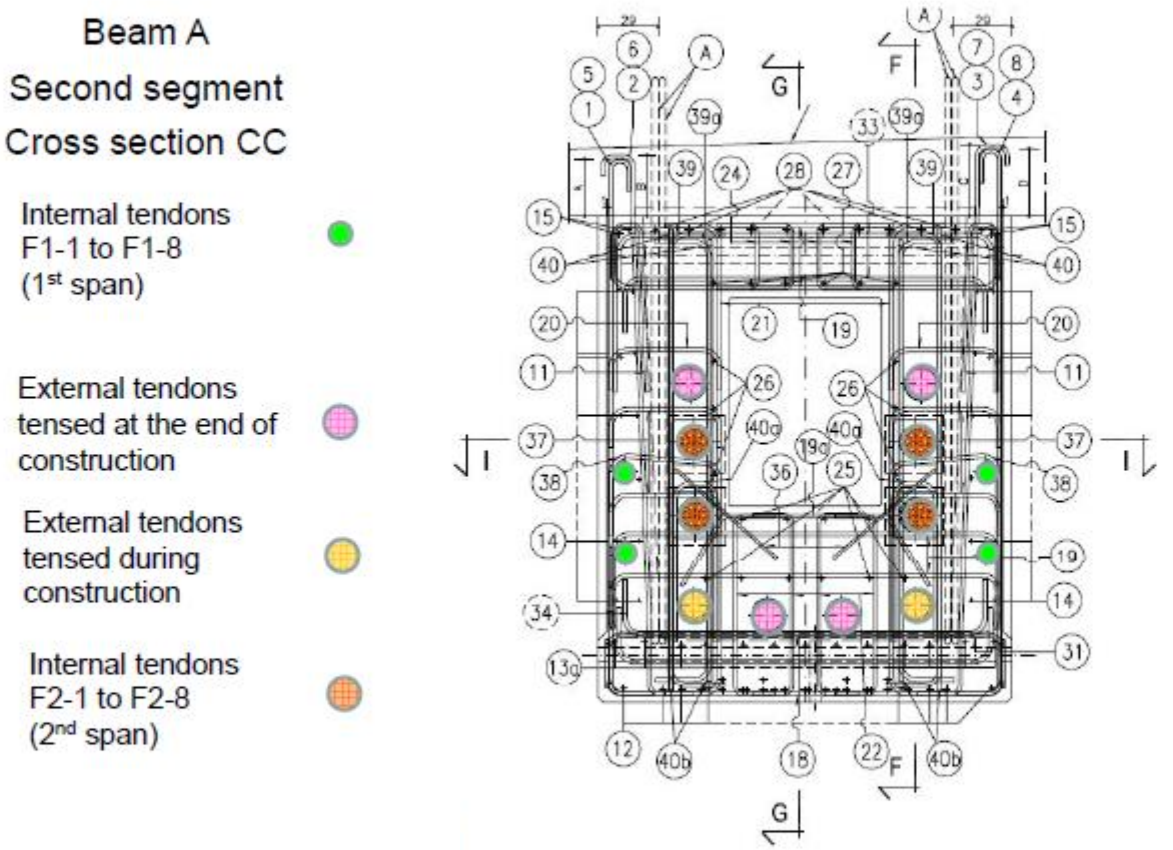
<u>Section AA</u>

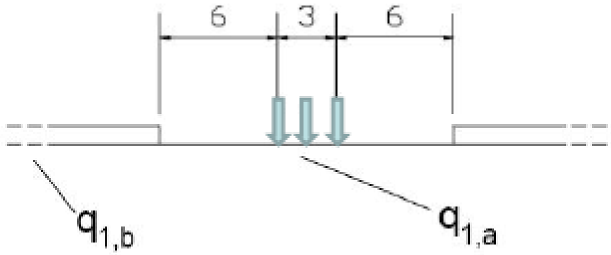
This is the section at the beginning of the bridge, in correspondence of the abutment. The section is crossed by ordinary reinforcement and tendons F1 ÷ F4 and EF1 ÷ EF4.



Section CC

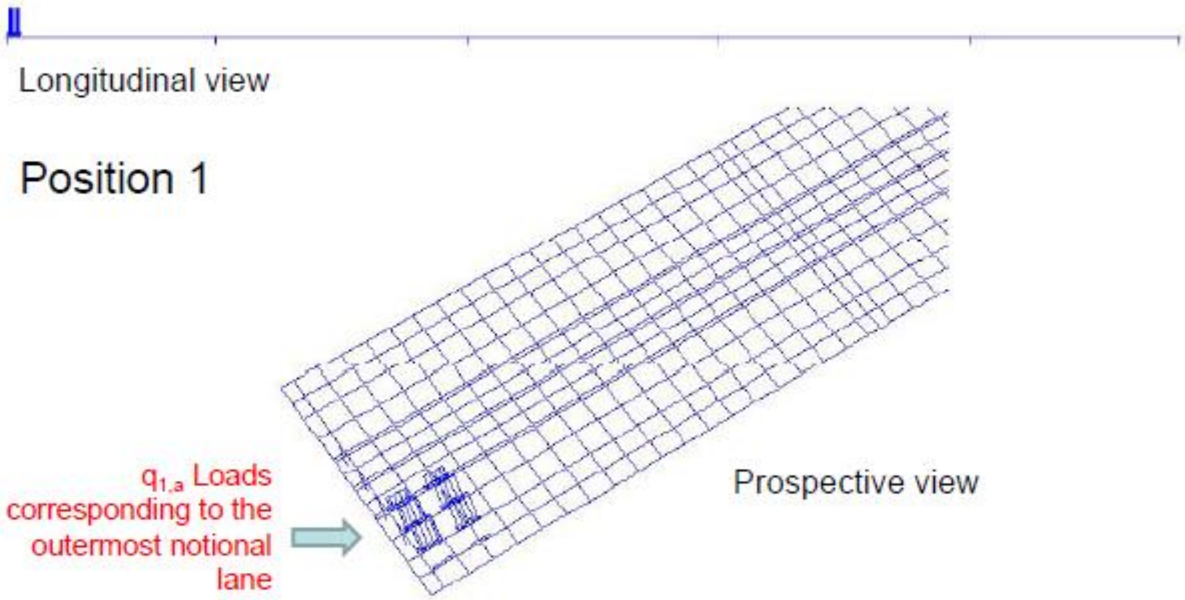
This is the section along the end of the second segment.
The section is crossed by ordinary reinforcement and by a huge number of tendons, including external tendons from segment 3, which are anchored to segment 2.





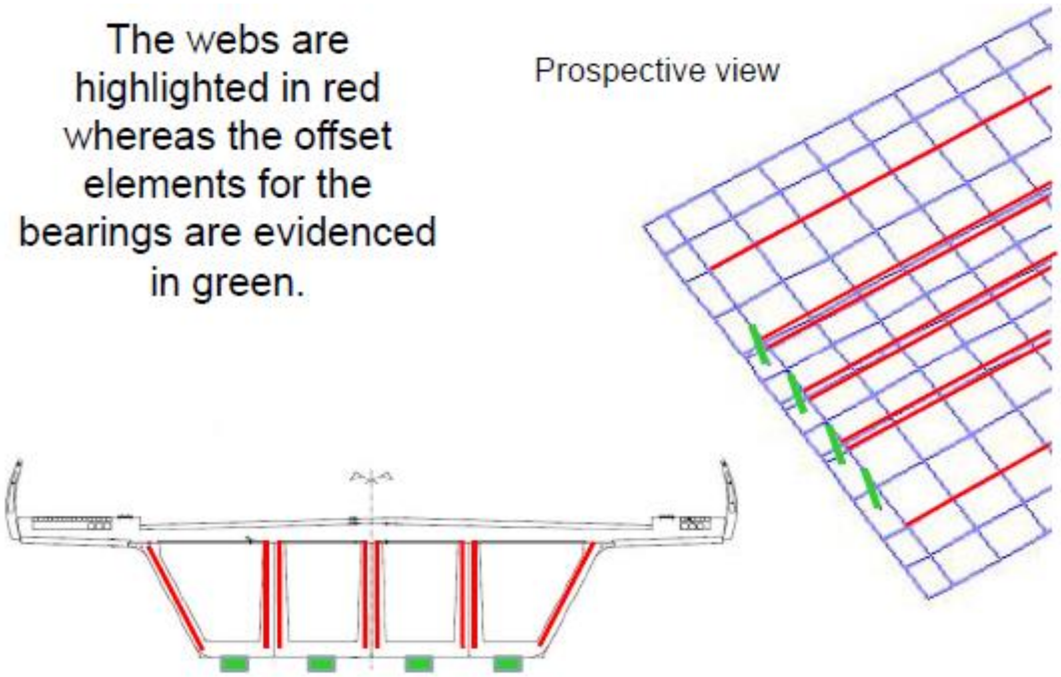
The model presents the mesh of the principal beam, i.e. four box section beams coupled in a unique pluricellular box beam, with a slab cast in situ. The single beam is modelled through beam elements, whereas the slab is modelled through 2D elements and it is linked to the beams.

The concentrated load $q_{1,a}$ has been placed in different longitudinal positions on the girder model. Since the total length is 266 m, we considered 89 load configurations, with stepping of the position of the concentrated load of 3 m (89 x 3 = 267 m ≅ 266).



The axle loads have been placed on each notional lane and the internal actions have been computed, repeating this operation 89 times. As result, we find 89 internal actions diagrams and we consider the envelope, in order to determine the design values of internal actions due to variable loads. We can notice that the load disposal will generate torque moment, too.

The same is done for the distributed load.



Moreover, in the bearing region, beam elements are not at the same level of gravity centre of the bearings and we take into account the offset through rigid links between the gravity centre of the beam and the bearings level. The slab is applied at the same level of the beam elements, that is gravity centres are at the same level, even if they are not in reality.

CONSTRUCTION PHASES

Construction phases were quite straightforward.

Construction phases			
Phase n°	**Time**	**Structure evolution**	
	[days]		
1	1	Birth of precast elements	Precasting
2	28	A. prestressing strands release B. tensioning of tendons S1 to S4 on pier elements	
3	35	Positioning of precast elements of span SP1-P1 and concreting of relative tranverse beams	Span 1
4	45	A. Tensioning of tendons F1-1 to F1-8 B. Removal of temporary piers of span SP1-P1	
5	51	Tensioning of tendons E1-1 to E1-8	
6	52	Positioning of precast elements of span P1-P2 and concreting of relative tranverse beams	Span 2
7	62	A. Removal of temporary piers of span P1-P2 B. Tensioning of tendons F2-1 to F2-8	
8	68	Tensioning of tendons E2-1 to E2-8	

Storage, lifting and transport scheme

L=0.35 for span elements
L=0.50 for pier elements

As there is no more prestressing reinforcement in that stage, in order to avoid cracking, we have to design correctly the structural element and introduce prestressing strands.
A table shows stress verification in the span element during storage lifting and transport, in which we evaluate internal actions due to self-weight and prestressing and, by superposition, we compute stresses at intrados and extrados of the element. In each section, tensile stress is smaller than tensile strength of concrete, so there is no cracking.

Beam A – Span element
Stress verification during storage lifting and transport

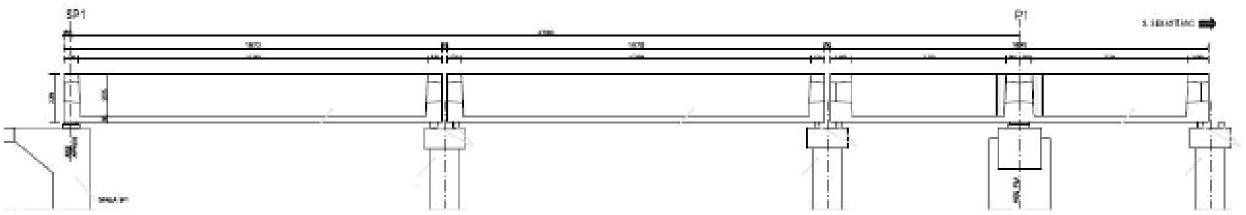
Section	x (long) [m]	Self weight N [kN]	Self weight M [kN m]	Self weight V [kN]	Prestress N [kN]	Prestress M [kN m]	Prestress V [kN]	Stresses σ inf [MPa]	Stresses σ sup [MPa]
1	0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	0.00
2	0.35	0.0	-2.6	-374.9	-249.0	-197.0	0.0	-0.35	0.19
3	0.7	0.0	126.1	-360.3	-689.9	-545.9	0.0	-0.84	0.31
4	1.35	0.0	351.5	-333.2	-1508.8	-1193.9	0.0	-1.76	0.55
5	2.35	0.0	663.9	-291.6	-2768.7	-2190.8	0.0	-3.20	0.97
6	3.35	0.0	934.6	-249.9	-2960.7	-2342.8	0.0	-3.20	0.65
7	4.35	0.0	1163.7	-208.3	-2960.7	-2342.8	0.0	-2.97	0.26
8	5.35	0.0	1351.1	-166.6	-2960.7	-2342.8	0.0	-2.78	-0.07
9	6.35	0.0	1496.9	-125.0	-2960.7	-2342.8	0.0	-2.63	-0.32
10	7.35	0.0	1601.0	-83.3	-2960.7	-2342.8	0.0	-2.53	-0.50
11	8.35	0.0	1663.5	-41.7	-2960.7	-2342.8	0.0	-2.46	-0.61
12	9.35	0.0	1684.3	0.0	-2960.7	-2342.8	0.0	-2.44	-0.64

Beam A – Pier element
Stress verification during storage lifting and transport
Pre-tensioned strands and internal tendons S1 and S2 present

Section	x (long) [m]	Self weight + strands + tendons			Stresses	
		N [kN]	M [kN m]	V [kN]	σ inf [MPa]	σ sup [MPa]
1	0	-2449.0	2288.0	476.0	0.84	-5.42
2	0.35	-2699.6	2254.6	58.3	0.66	-5.51
3	0.7	-3143.1	2279.9	77.0	0.42	-5.82
4	1.35	-4003.2	2088.7	-11.0	-0.29	-6.01
5	2.35	-5310.2	1819.3	-105.2	-1.35	-6.33
6	3.35	-5541.8	2206.5	-205.1	-1.10	-7.13
7	4.35	-5564.7	2547.3	-257.8	-0.77	-7.73
8	5.35	-5569.2	2790.5	-216.1	-0.52	-8.16
9	6.35	-5573.7	2992.0	-174.5	-0.32	-8.51
10	7.35	-5578.2	3151.8	-132.8	-0.16	-8.79
11	8.35	-5582.6	3269.8	-91.2	-0.05	-8.99
12	9.35	-5578.0	3333.7	49.5	0.02	-9.10

In the pier element, we perform two verifications, one with only prestressing strands and the other one with tendons tensed at release of prestressing strands.
Then, the pier elements are placed over the bearings and there is tensioning of tendons S1 to S4 on pier elements.

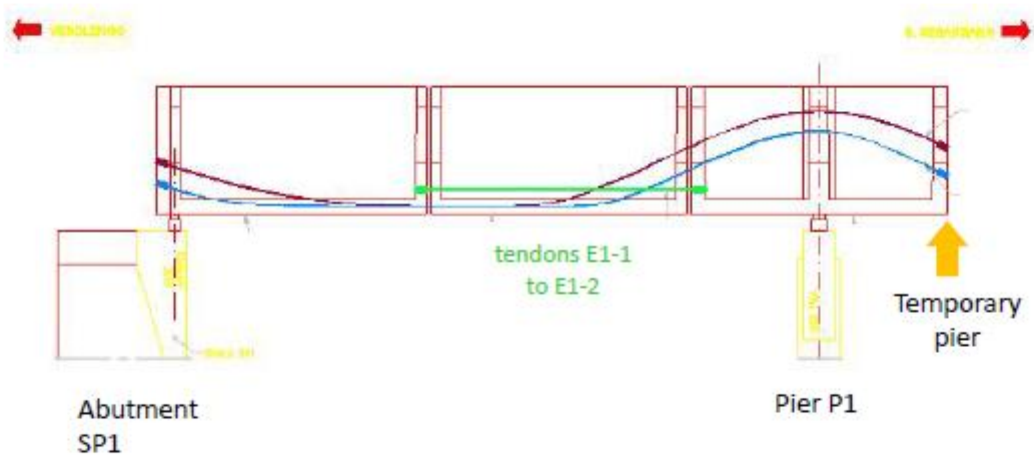
3. Positioning of precast elements of span SP1-P1 and concreting of relative transverse beams.



Abutment SP1 | Temporary pier | Temporary pier | Pier | Temporary pier

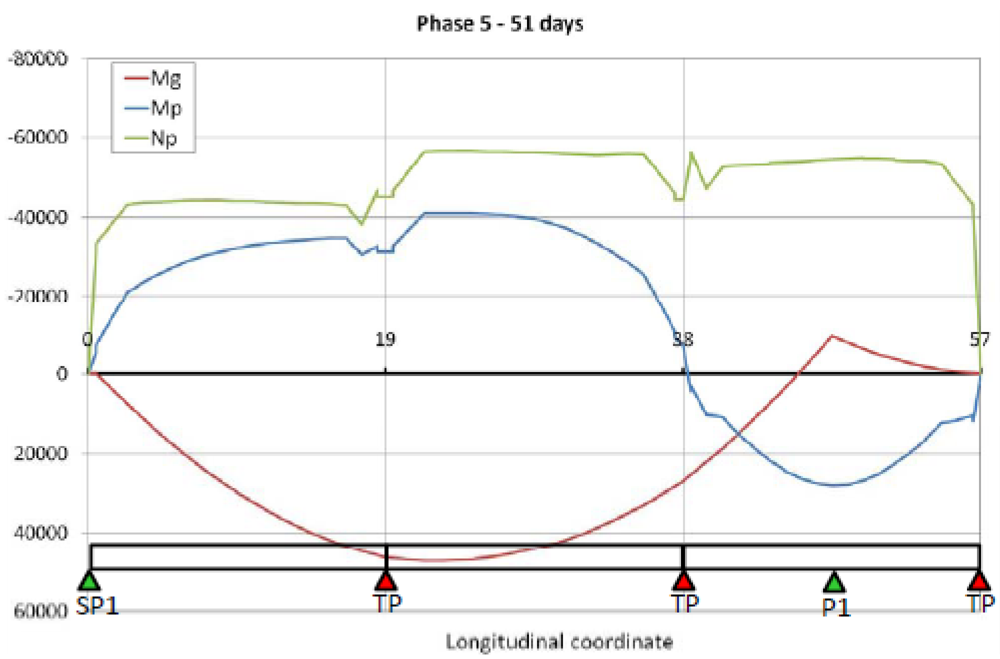
In this stage, we place the first structural elements on the abutment SP1 and the temporary piers and the segments, at the beginning, work according to a simply supported scheme. The pier element is already working with a hyperstatic scheme and that is the reason of the introduction of the posttensioned tendons before the disposal.
As regards internal actions on each element, we distinguish different contributions.

- Self-weight is acting with a simply supported scheme on the span elements, whereas it generates negative bending moment in the pier element.
- Axial force due to prestressing assumes a constant value along each element, unless prestressing losses.
- Bending moment due to prestressing is opposite in sign with respect to bending moment due to self-weight, in order to counterbalance it.
  In the pier element, the upper tendon gives arise a positive bending moment and, taking into account prestressing losses in prestressing tendons and the effect of posttensioning technology, it is a little bit smaller than the one in the span segment.

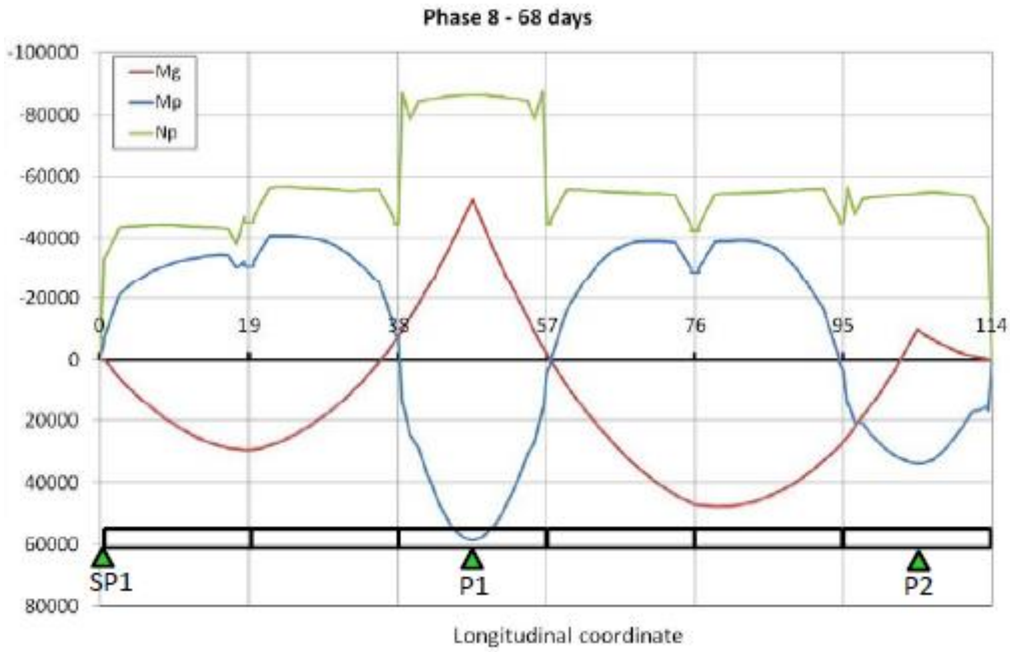


As regards internal actions,

- Bending moment due to self-weight is acting with the classical shape.
- Axial force due to prestressing is quite constant along the span and it is bigger in the middle region due to additional tendons E1. In correspondence of the joints and the anchorages, we can see prestressing losses.

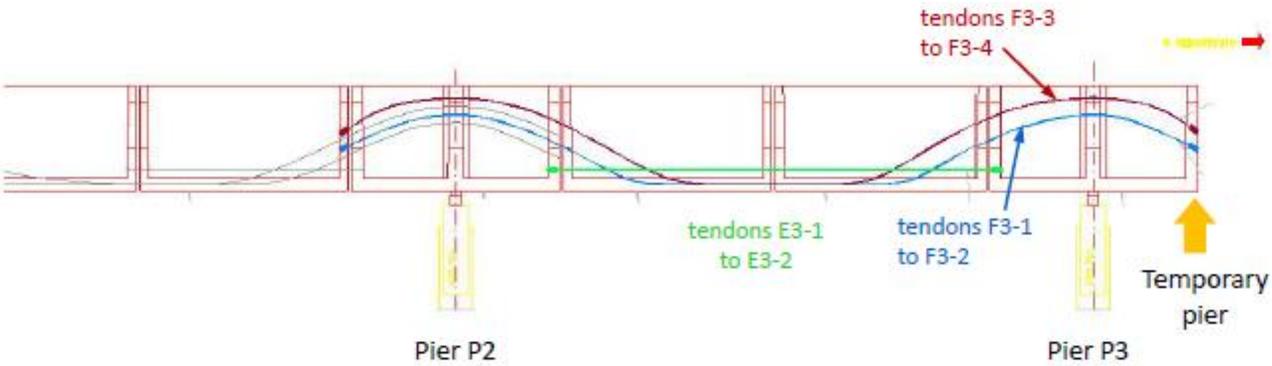


6. Once we have realized the first span, we realize the second span in a similar way. The new span is working on an isostatic scheme and it is supported by temporary and permanent piers.
   The elements in the first span are working under the same internal actions of before, whereas the new elements work in an independent way, with internal actions of the isostatic scheme.

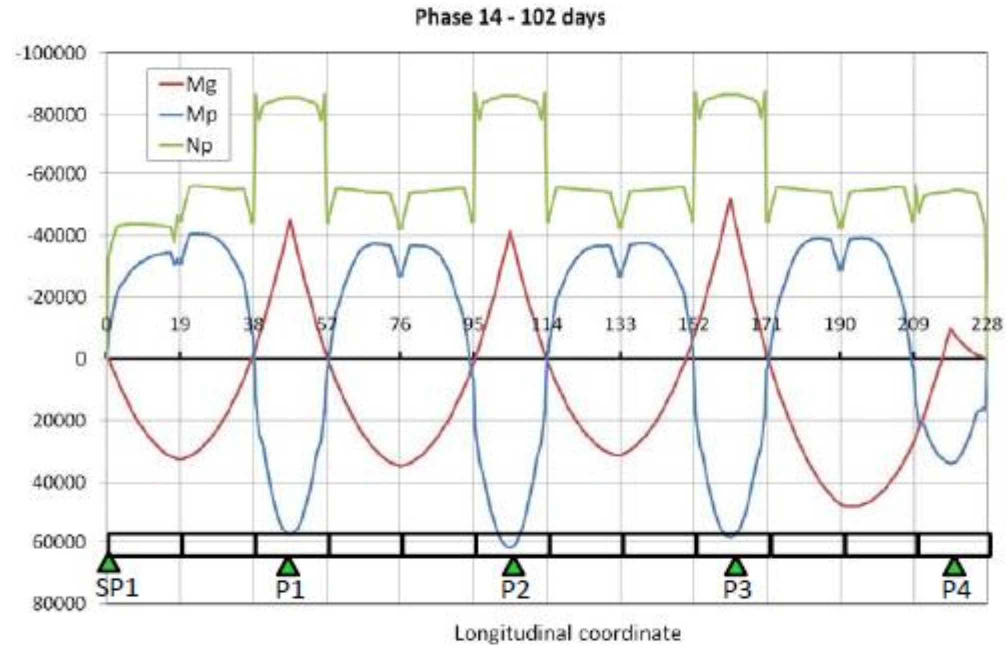


By consequence, the structure is strongly compressed and subjected to a relatively small bending moment.

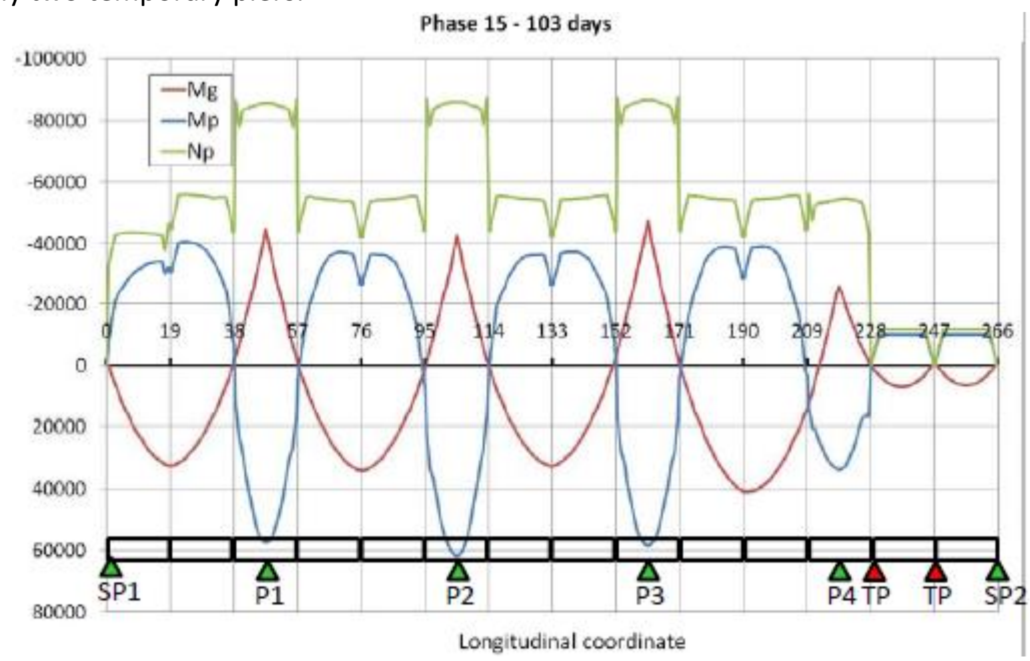
9. We add the elements of the third span.
10. The segments on the third span are disconnected and they are connected through posttensioned tendons F3-1 and F3-2.
11. We add the connecting tendons E3 in order to solidarize the third span with the second one.



As regards internal action, there is a surplus of compression on bearing at chainage 114 m due to the anchor of connecting tendons in the second span.

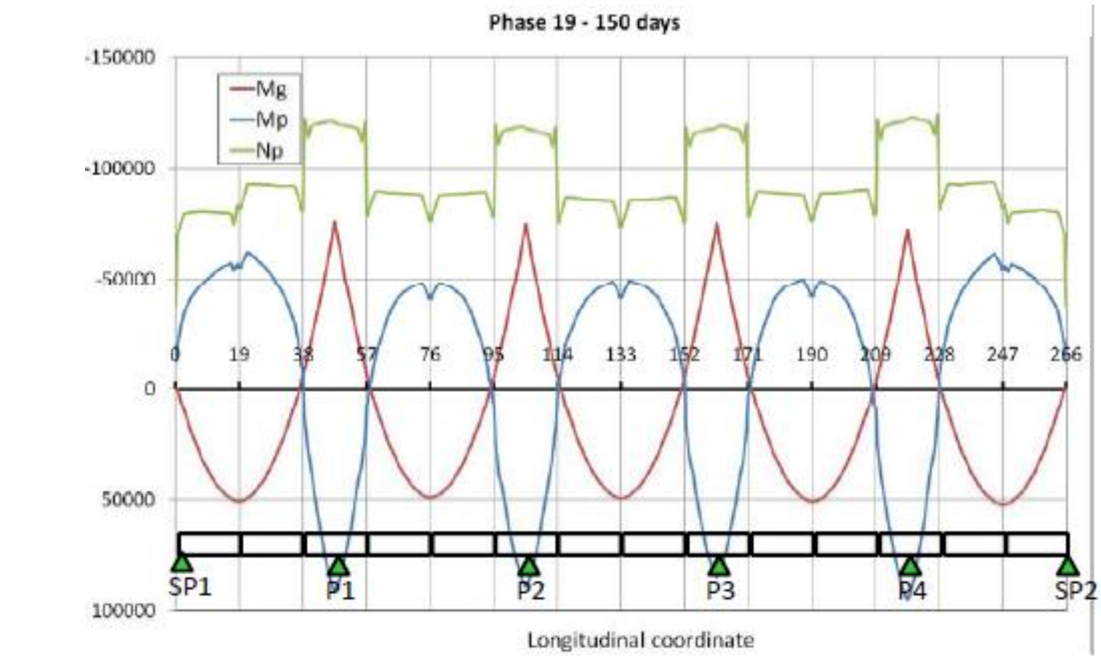


15. The realization of the last span is different because there are only two elements to be placed, on only two temporary piers.

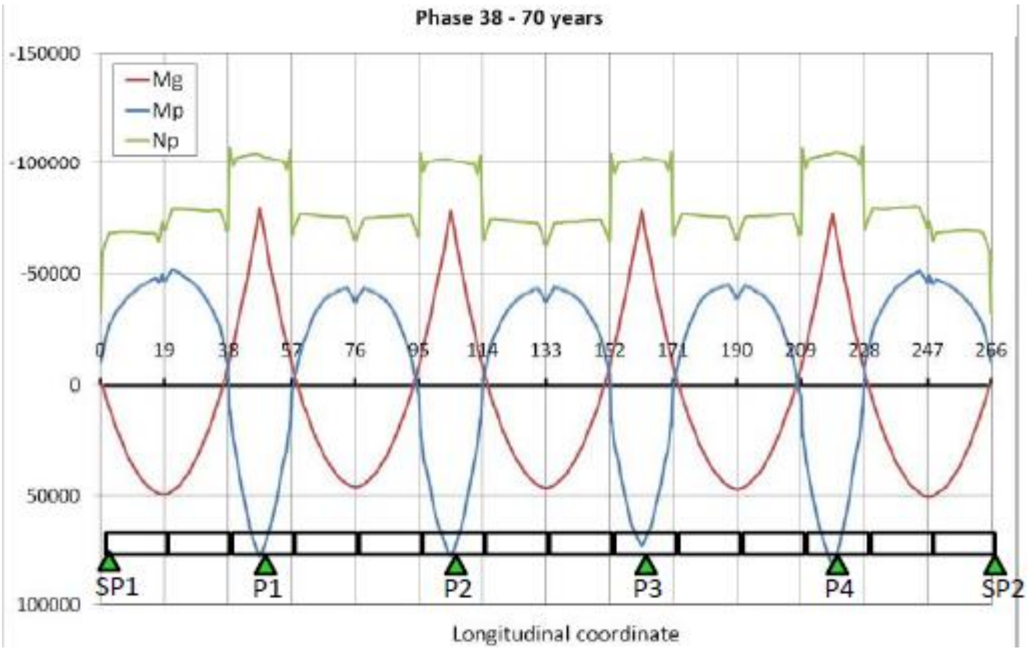


16. The segments on the last span are disconnected and they are connected through posttensioned tendons F5-1 and F5-2.
17. We add the connecting tendons E5 in order to solidarize the fifth span with the fourth one.

open section to a closed section – and the gravity centre has a new position. This leads to a redistribution of internal actions.



38. At the end of construction, the segments have already experienced creep and shrinkage in the factory. The most important problem is given by the cast in situ slab, which will develop all the shrinkage during service life.
At the end of service life, the step by step analysis of creep and fluage effects gives the final distribution of internal actions.



We can notice that the time subdivision is more detailed at the beginning, as creep and shrinkage are more important when concrete is young, then the effect is reduced.

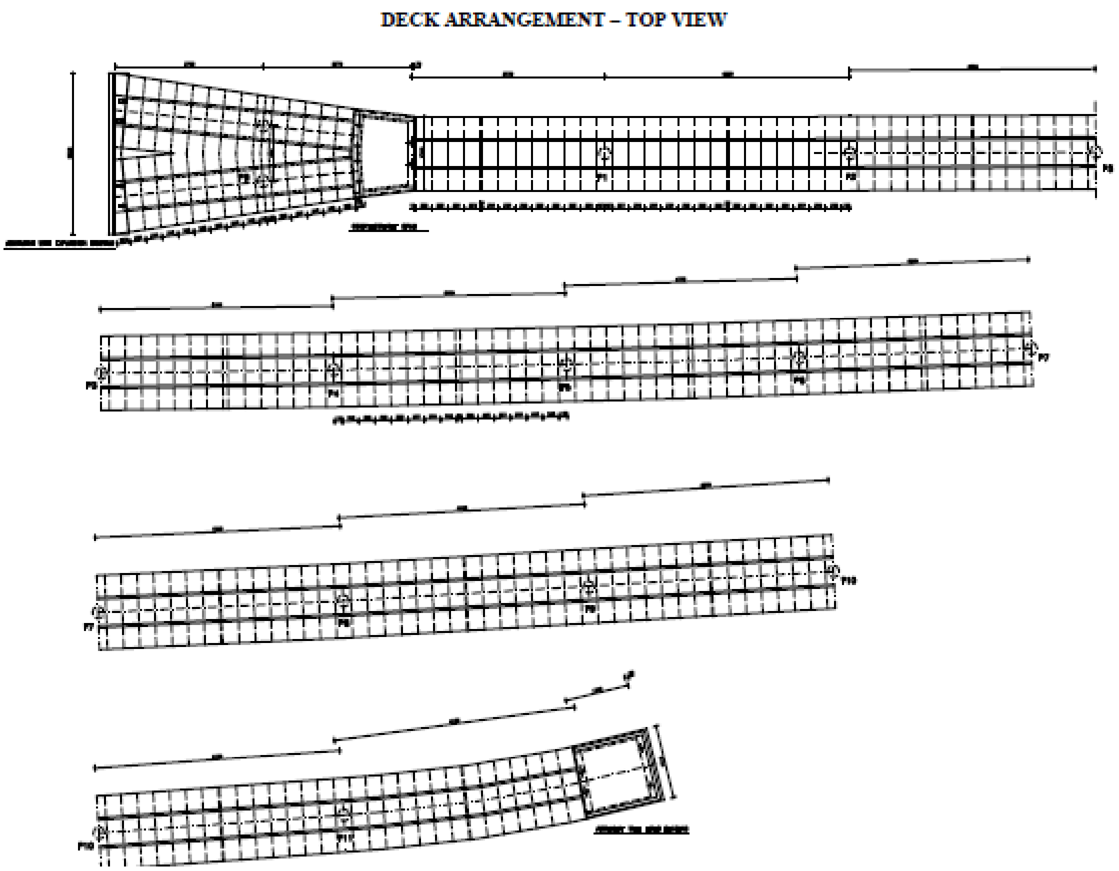
Thus, the analysis is performed with increasing steps of time.

TEMPORARY PIER

BOX GIRDER BRIDGES: SEGNO VIADUCT

The Segno Viaduct is a bridge having 12 spans, with a length close to 40 m each one, except the spans near the abutments.

The first part of the bridge presents an abutment on which is supported a curved deck with two carriageways, that part has two spans and the one with bigger length is supported by the central bearing.
The remaining part of the bridge is made by an unique deck, supported on a massive structural element which, during the construction stages, works like a counterweight, since the deck was realized through in situ assembly of segmental precast elements with classical balanced symmetric incremental launching cantilever from the piers.

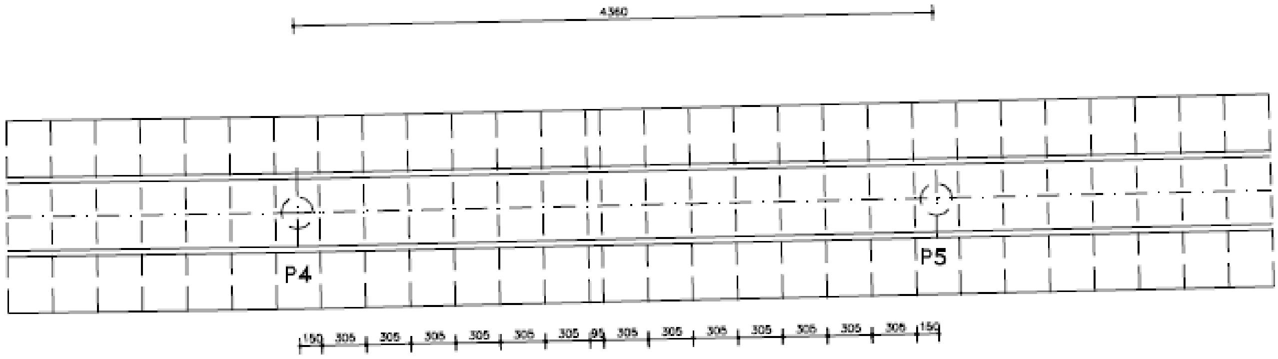


LONGITUDINAL SECTION

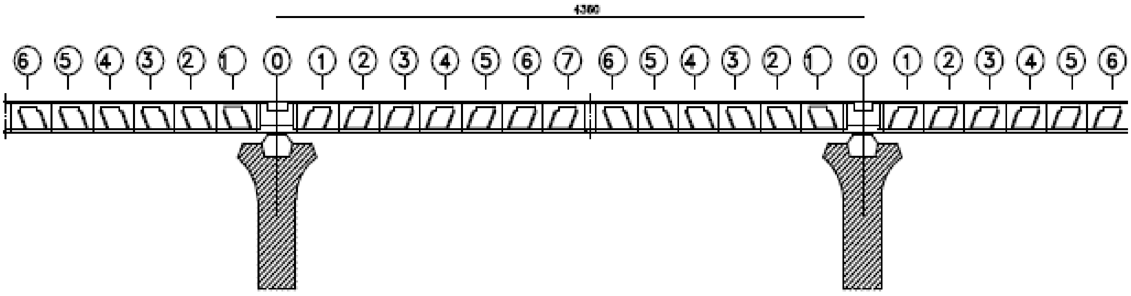
The first abutment is made with precast elements and the internal part is filled with soil in order to avoid the overturning of the first part of the bridge, during the construction.

The deck is made of box section precast segments 3 m long, by means of a crane placed on the river bed – not by a launching girder, since piers are short.

DECK ARRANGEMENT – 43.60 m SPAN – TOP VIEW

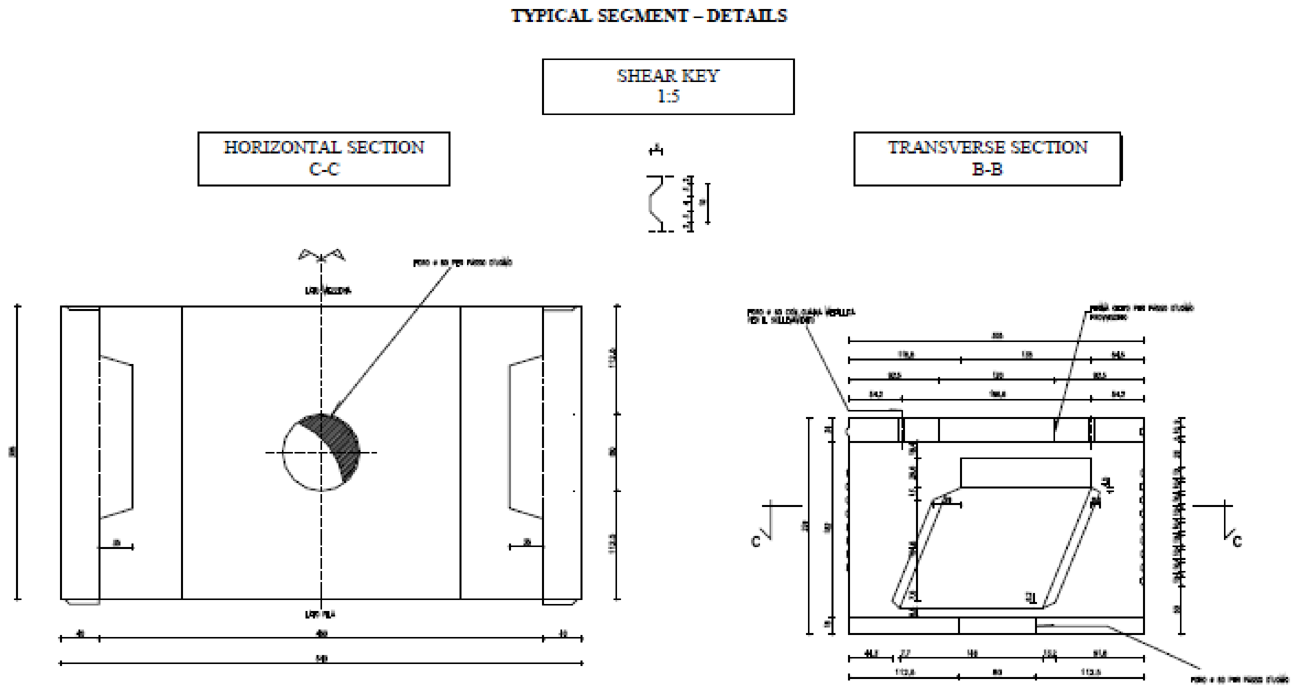


DECK ARRANGEMENT – 43.60 m SPAN – LONGITUDINAL SECTION



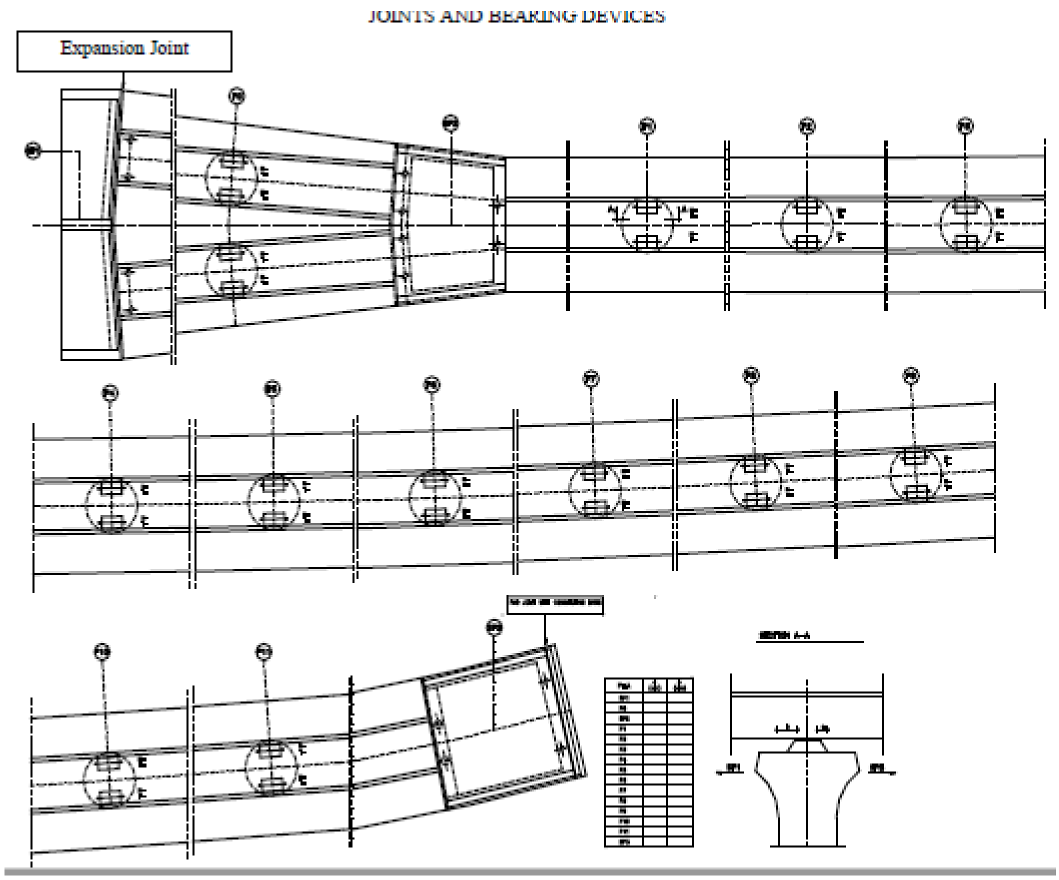
At certain point, the hammers (“stampella”) starting from adjacent piers get in contact. The contact node may be a Gerber siege with a little concreting. This is not such an optimal connection, which can be modelled as a hinge and, due to shrinkage, some deflection may occur.

The best technique consist of realizing the connection by cast in situ segment, which ensures the continuity of the structure.



BEARING DEVICES

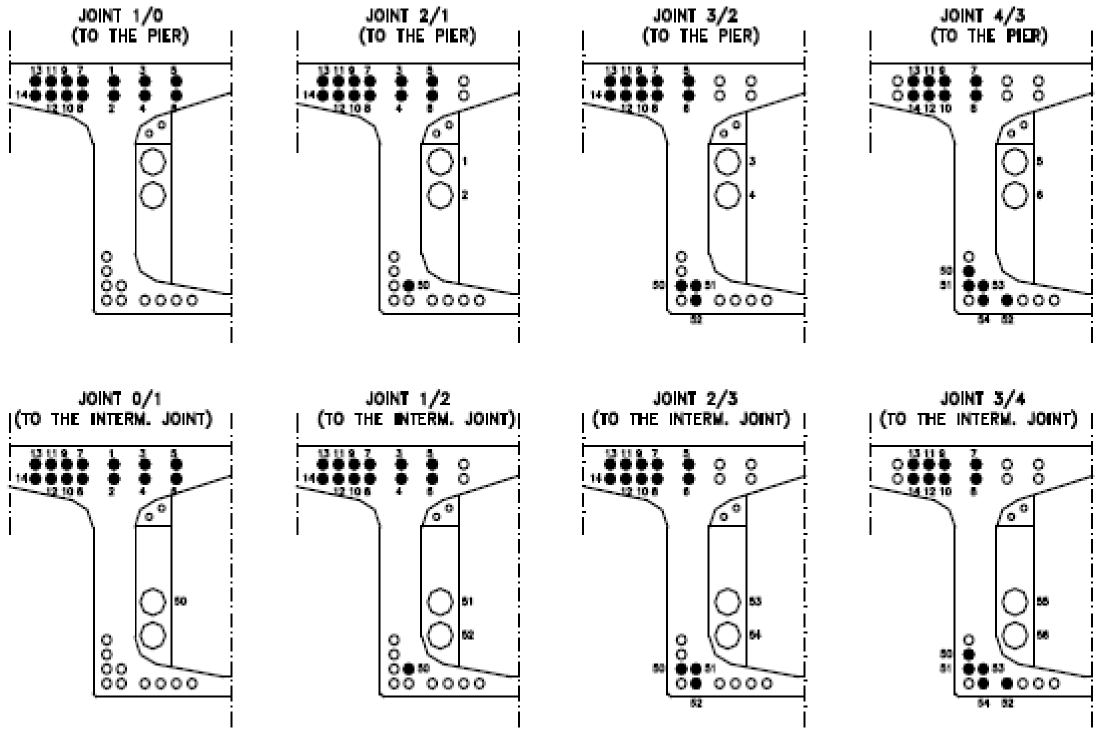
Each bearing is able to provide a certain degree of freedom, in order to allow lengthening and shortening in the longitudinal and transverse direction.
In the pier bearings, one bearing is free in the longitudinal direction and restrained in the transverse direction, whereas the other ones are free in the two directions.
The bearings with translational restraint are placed in the abutments.



TENDONS MASKS

Tendons masks are schemes representing the holes in each section, which are filed with tendons.

46.00 m HAMMER: TENDONS MASKS (PART 1)

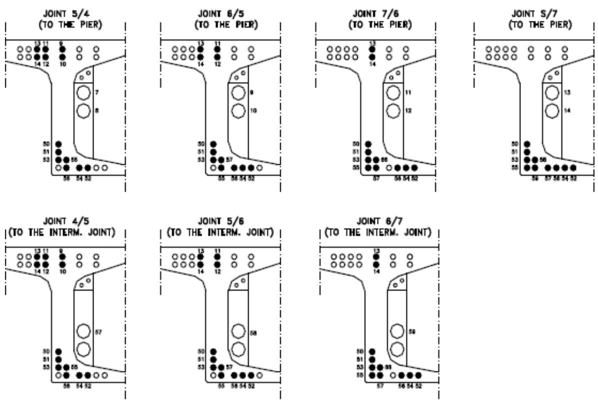


The segments have the same geometry, for simplicity in the construction.

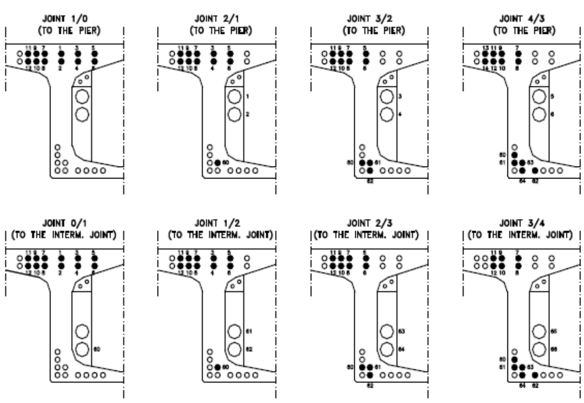
The joint between elements 0 and 1 starting from the pier shows a huge number of holes in the upper part filled with tendons because, at the end of tensioning, the first tendon starts from segment 1 and all negative tendons are passing there.
The joint between elements 1 and 2 has two cables less in the intrados and 1 tendon in the extrados.

The representation is performed for any interface between the segments.

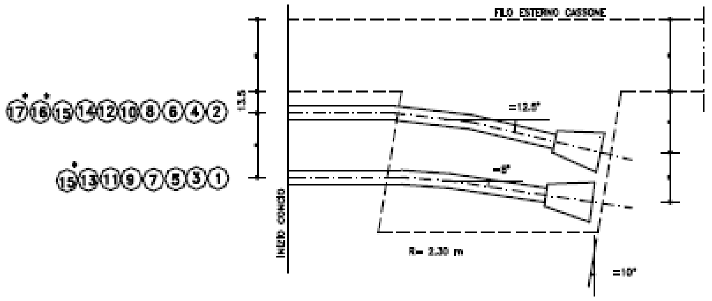
46.00 m HAMMER: TENDONS MASKS (PART 2)



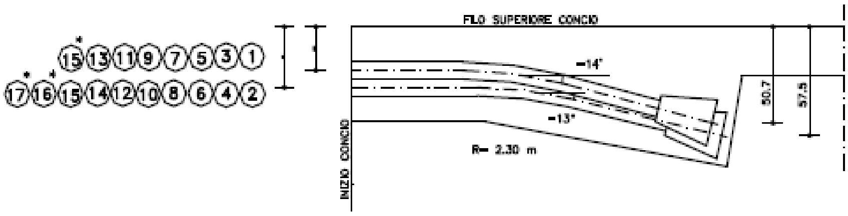
43.60 m HAMMER: TENDONS MASKS (PART 1)



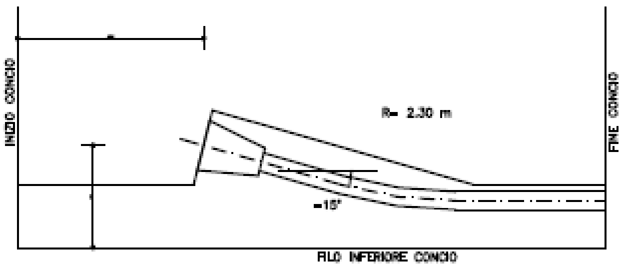
TENDONS LAYOUT DETAILS: TOP SLAB TENDONS ANCHORAGE (VERTICAL SECTION)



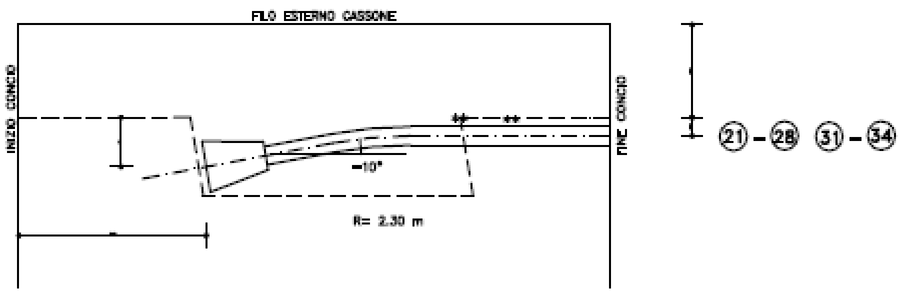
TENDONS LAYOUT DETAILS: TOP SLAB TENDONS ANCHORAGE (HORIZONTAL SECTION)



TENDONS LAYOUT DETAILS: BOTTOM SLAB TENDONS ANCHORAGE (VERTICAL SECTION)



TENDONS LAYOUT DETAILS: BOTTOM SLAB TENDONS ANCHORAGE (HORIZONTAL SECTION)



PIER

The pier segment is placed above the pier and temporarily anchored to it through prestressing bars – which will be removed then – in order to avoid overturning of the deck. Indeed, we need a tensile reaction to counterbalance bending moment due to the cantilever.

The bearing provides a restraint in the longitudinal and transverse direction and towards the uplift. The degree of restraint is not full, so that we can place a jack above the pier and uplift the deck, to substitute bearings.
Piers are connected to a massive plinth foundation with piles.

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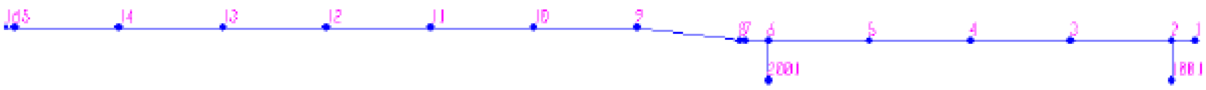
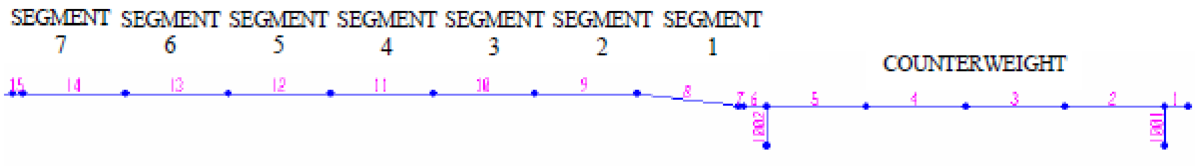
to wobble – and losses due to penetration of the anchorages – this effect is important in the first meters and then for the first segment.

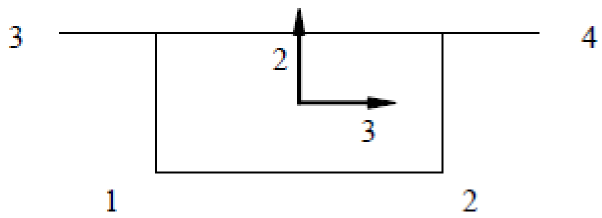
The following analysis regards the half hammer starting from the counterweight and made of 7 segments, but it is valid for all the other hammers. The model is shown below, with the axis convention and for the keys points for stress calculation.



Nodes numbering



Elements numbering



Section conventions and key points

Global stresses and local joint stresses due to permanent actions and prestressing are shown in the following tables.

Then, we analyse the effects due to the assembling of the other segments.

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	-176.6	-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	6865.3
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	-633.9	0.0	8.5	1.1	-820.5	-7577.1	1262.0	-0.1	-0.3	0.1	2803.5
9	1	0.0	-538.1	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_1 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_1 [MPa]	σ_1 [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

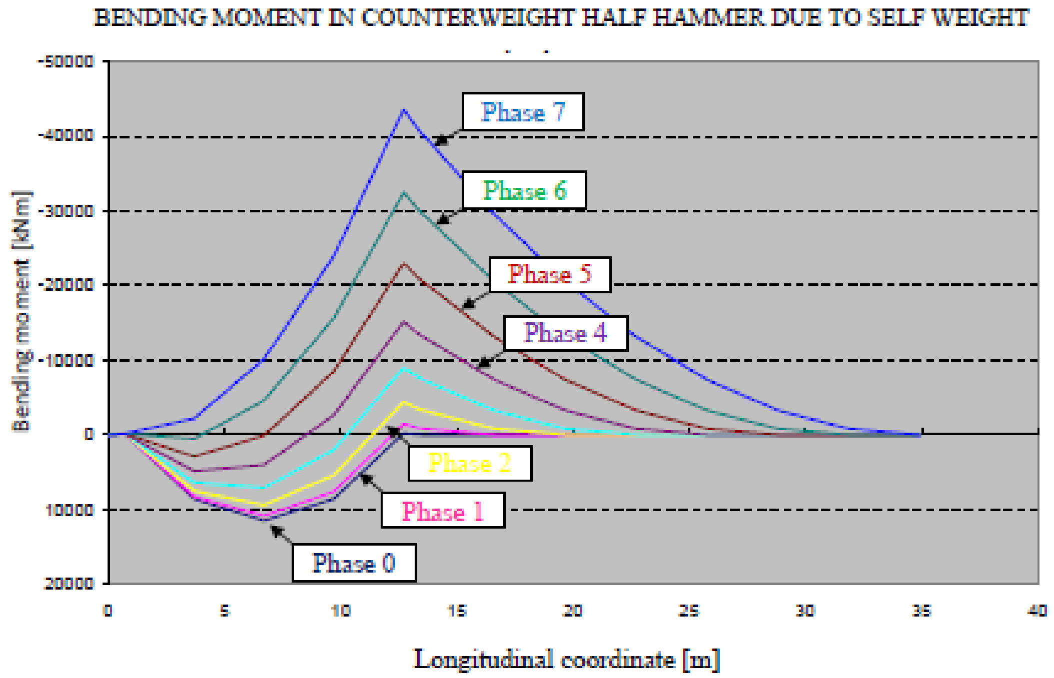
		FASE 6 - tempo: 35 [gg] - sollecitazioni totali											
		Permanenti						Precompressione					
Elemento	P.to	F1	F2	F3	M1	M2	M3	F1	F2	F3	M1	M2	M3
1	1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	2	0.0	448.7	0.0	0.0	0.0	-155.2	-45934.8	212.0	-3.9	0.3	1.8	-3711.8
2	1	0.0	-2645.4	0.0	493.3	0.0	-166.0	-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	-2674.9	0.5	651.0	0.6	2055.7
3	2	0.0	1248.6	0.0	493.3	0.0	4026.8	-48136.4	-2908.5	0.5	651.0	-1.0	10868.3
4	1	0.0	1248.6	0.0	493.1	0.0	4026.9	-48180.5	-2288.9	-1.1	650.6	-1.0	10917.1
4	2	0.0	3195.6	0.0	493.1	0.0	-2643.5	-47862.9	-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	-33699.5	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	-7365.0	-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	-16896.5	2066.2	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	2950.0
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

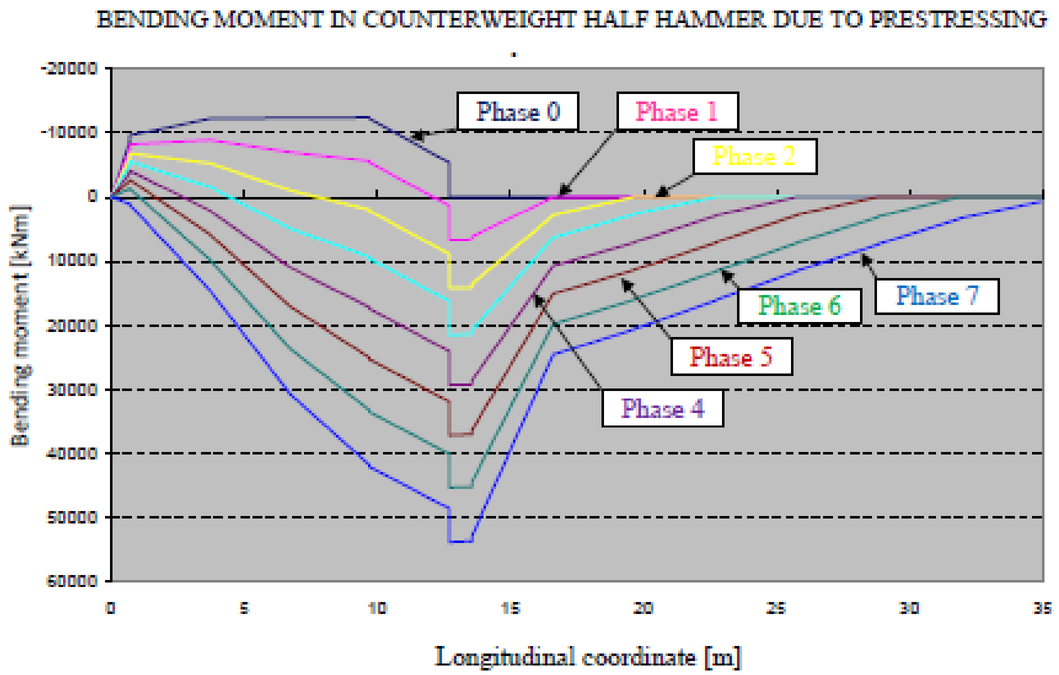
		Fase 6 - tempo: 35 [gg]							
		Concio 4		Cavi concio 4		Prima della prec.		Dopo la prec.	
Elemento	P.to	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_3 [MPa]	σ_2 [MPa]	σ_1 [MPa]	σ_2 [MPa]	σ_1 [MPa]
1	1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	2	0.004	0.006	-0.212	-0.461	-1.956	-0.924	-2.167	-1.366
2	1	0.004	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	-0.980	-0.609	-2.381	-0.326	-3.341
4	1	-0.320	0.323	0.283	-0.982	-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	-0.803	0.696	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	-4.606
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	-4.163
9	1	-1.496	0.723	0.256	-2.144	-2.636	-2.016	-2.381	-4.169
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	-1.566
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

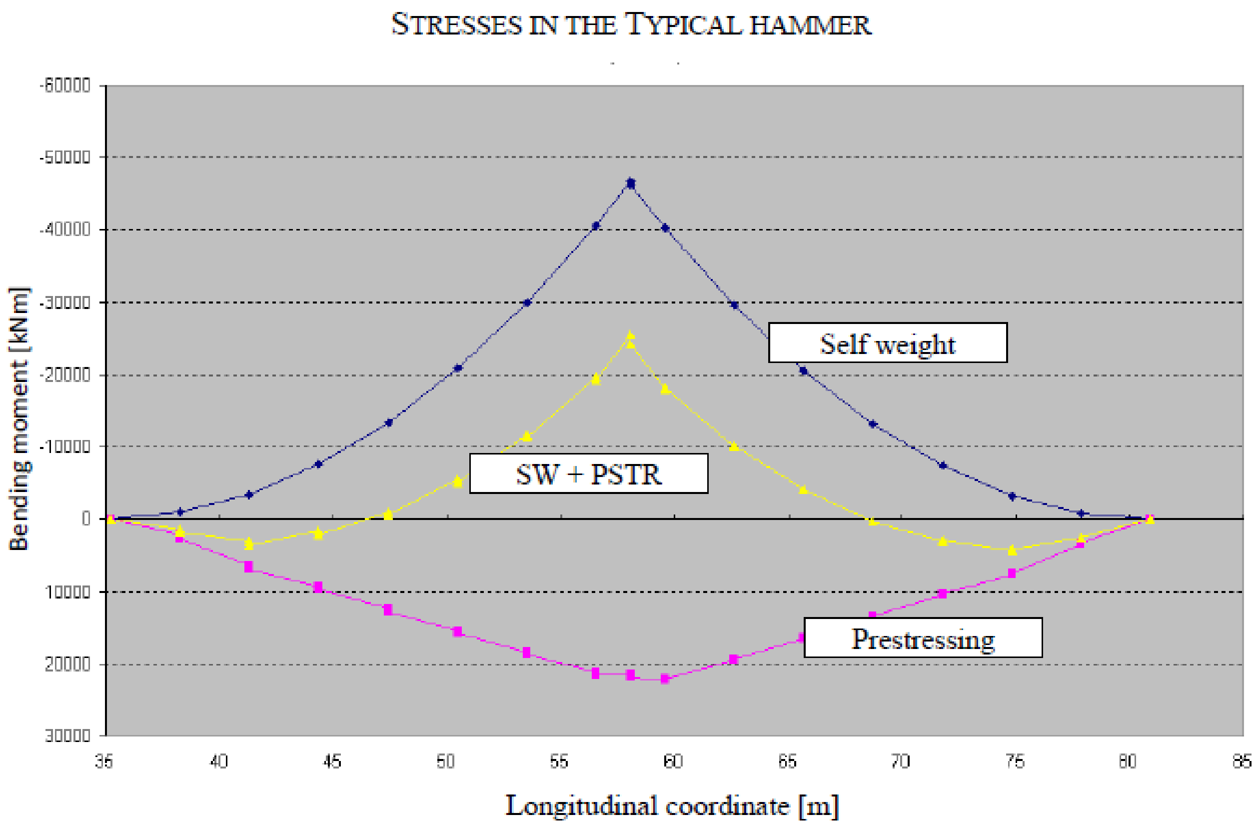
PHASE 6. ASSEMBLING OF SEGMENT 6

		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	-148.5	-63839.5	-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	2891.1	0.0	-310.1	0.0	-4616.2	-66829.1	-4526.0	0.6	2001.0	-1.1	23506.1
4	1	0.0	2891.1	0.0	-309.9	0.0	-4616.2	-66907.3	-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	-51836.6	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	-40406.1	2066.4	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	-32524.8	2062.4	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	-7386.0	-24057.6	2171.7	-0.3	-0.7	-0.3	11641.5
11	2	0.0	-1076.2	0.0	42.9	0.0	-3202.0	-15290.0	325.3	-0.2	-0.7	0.6	7156.8
12	1	0.0	-1076.2	0.0	0.6	0.0	-3202.3	-15004.4	2070.8	0.2	-0.1	0.6	7003.4
12	2	0.0	-538.1	0.0	0.6	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	-0.205	-0.495	-2.372	-1.835	-2.575	-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	0.089	-0.738	-1.527	-2.793	-1.458	-3.531
3	1	-0.241	0.248	0.089	-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	-2.130	-0.688	-3.284
6	2	-0.941	0.926	0.511	-1.162	-0.955	-2.347	-0.444	-3.508
7	1	-1.704	1.310	0.938	-1.772	-1.713	-3.969	-0.765	-5.731
7	2	-1.685	1.292	0.937	-1.770	-1.619	-4.039	-0.682	-5.809
8	1	-3.319	1.595	1.654	-2.780	-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

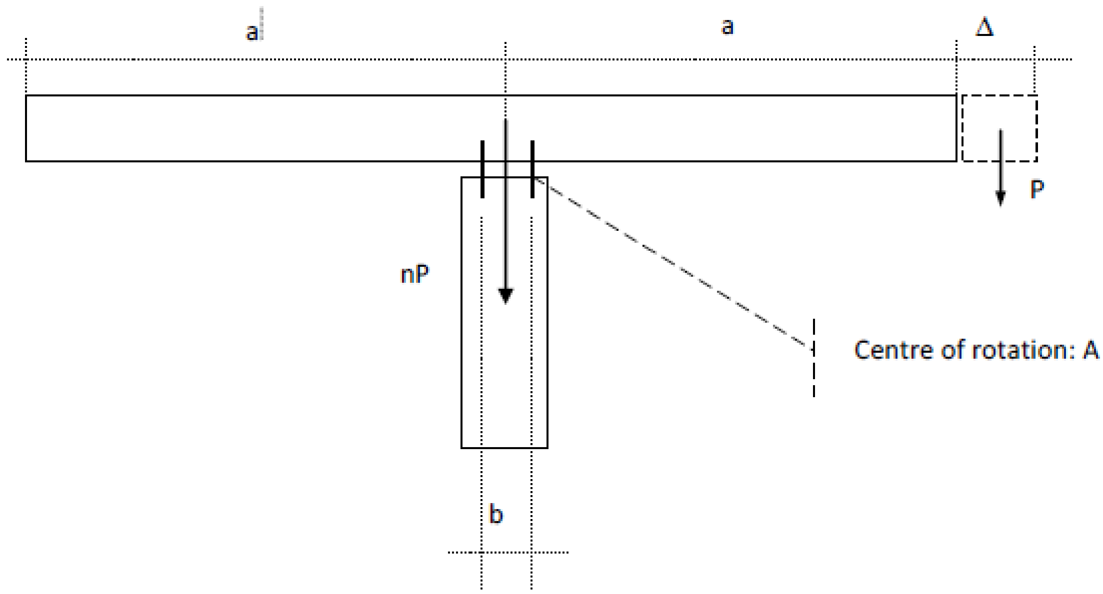

BENDING MOMENT IN COUNTERWEIGHT HALF HAMMER DUE TO SELF WEIGHT
Phase 7
Phase 6
Phase 5
Phase 4
Phase 2
Phase 1
Phase 0
Bending moment [kNm]
-50000
-40000
-30000
-20000
-10000
0
10000
20000
0
5
10
15
20
25
30
35
40
Longitudinal coordinate [m]


BENDING MOMENT IN COUNTERWEIGHT HALF HAMMER DUE TO PRESTRESSING
Phase 0
Phase 1
Phase 2
Phase 6
Phase 7
Phase 5
Phase 4
Bending moment [kNm]
-20000
-10000
0
10000
20000
30000
40000
50000
60000
0
5
10
15
20
25
30
35
Longitudinal coordinate [m]



VERIFICATION OF THE EQUILIBRIUM OF A TYPICAL HAMMER

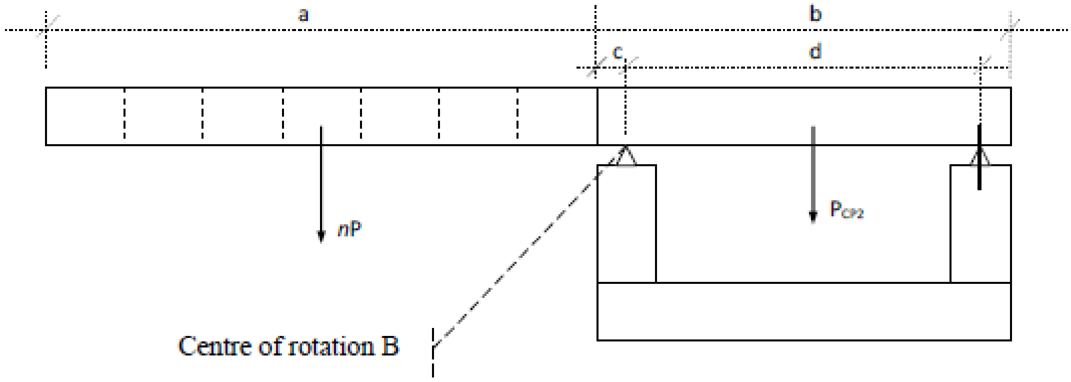
During construction phases, we have to check the global equilibrium of the hammer in case of asymmetric disposition of segments.



In this case, self-weight is a favourable action and it is computed with a safety factor equal to 0,9.

The worst situation is at the final stage, when we install the last segment, since the weight is the same but the lever-arm with respect to the rotation centre – the bearing – is big.

It consists of the analysis of the equilibrium to rotation of the hammer around the rotation centre placed at the border of the abutment.



In this case, the abutment should theoretically provide a tensile reaction to give equilibrium. Generally, we can design a counterweight to balance the overturning action, but it would be better to fix a vertical restraint, in order to avoid problems.

The values of interest are the following.

- Number of segments in the cantilevering hammer.
$$n = 7$$
- Length of cantilevering hammer.
$$a = 7 \times 3{,}05\ [m] = 21{,}35\ m$$
- Length of the abutment.
$$b = 13{,}4\ m$$
- Distance between the attachment point of the hammer and the centre of rotation B.
$$c = 0{,}80\ m$$
- Distance between the bearings.
$$d = 12{,}00\ m$$
- Weight of the single segment.
$$P = 538\ kN$$
- Weight of the abutment.
$$P_{CP,2} = A_{CP,2} b \gamma_{cls} = 25{,}944\ [m] \times 13{,}4\ [m] \times 25\ [kNm^{-2}] = 8691\ kN$$

Firstly, the analysis for overturning around point B without anchorages bars.

The overturning moment is

$$M_s = nP\left(\frac{a}{2} + c\right) = 7 \times 538\ [kN] \times \left(\frac{21{,}35\ [m]}{2} + 0{,}8\ [m]\right) = 43215\ kNm$$

The stabilizing moment is

$$M_R = P_{CP,2}\left(\frac{b}{2} - c\right) = 8691\ [kN] \times \left(\frac{13{,}4\ [m]}{2} - 0{,}8\ [m]\right) = 51277\ kNm$$

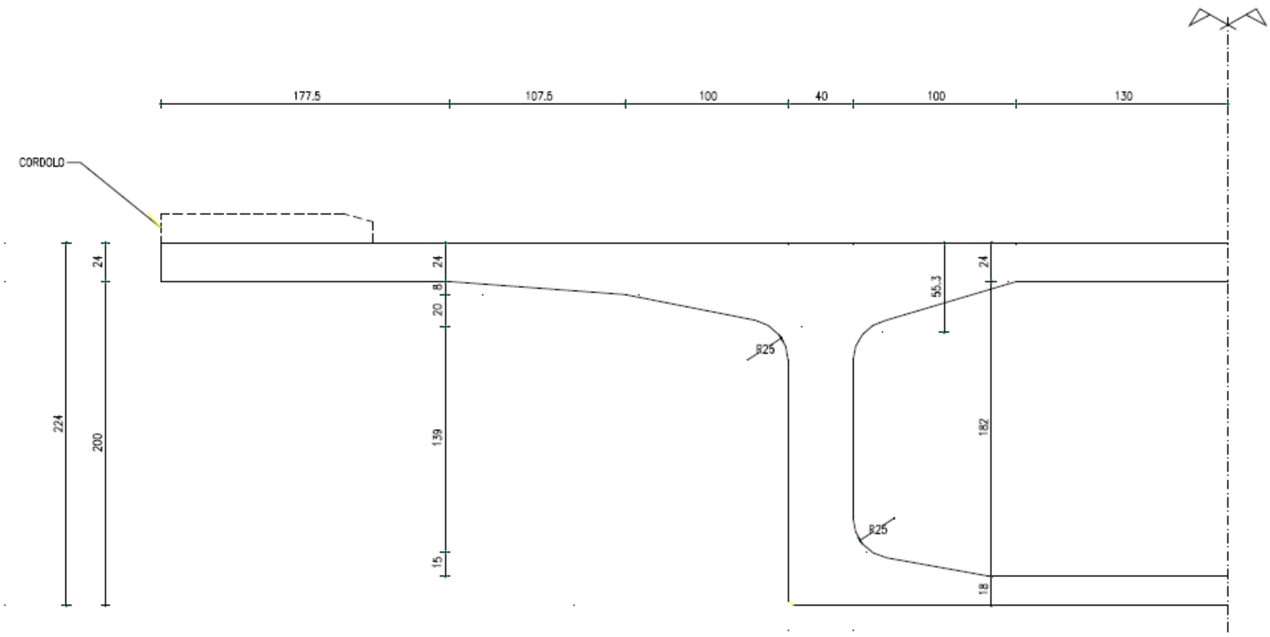
The acting bending moment depends on the number of segments, whereas the resisting bending moment depends on the counterweight and the tendon.

The safety factor is

longitudinal action is not at its maximum value, since the variable load is not applied in all the possible positions on the deck.
Thus, the longitudinal bending and longitudinal shear related to the maximum torsion is always smaller than the maximum longitudinal bending or the maximum longitudinal shear. In this way, the combination which maximizes torsion is generally a "light" combination, compared with the combinations maximizing longitudinal actions.
If we forget this phenomenon, we will make a mistake but most of the time the mistake is covered because there are other combination with much stronger longitudinal actions covering this mistake.
The calculations which we will see are not determinant most of the time in the design but this is a general situation and we may encounter some exceptions.

GEOMETRIC CHARACTERISTICS

The section of the segment has a bulk area, in order to give space for the tendons.



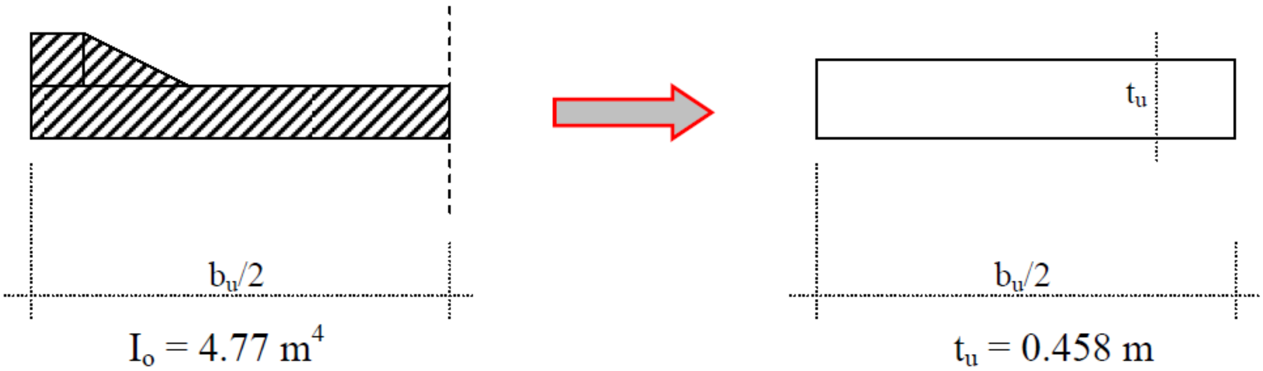
In the design, the first step is the simplification of geometry of the section with another shape, having constant depth in each wall and an equivalent length in the elements.

The reference length are the following.

$b = 13.10$ 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

$$I_{rect,u} = \frac{t_u \left(\frac{b}{2}\right)^3}{12} = I_u$$



Once we know the thicknesses t_0 and t_u, we can compute the length of the web b_s, evaluated from the centroids of the two slabs.

$$b_s = b - \frac{t_o}{2} - \frac{t_u}{2} = 2{,}24\ [m] - \frac{0{,}458\ [m]}{2} - \frac{0{,}287\ [m]}{2} = 1{,}87\ m$$

In this way, we have the simplified version of the cross section of the deck.

At this point, we can calculate inertia moments.

- Longitudinal (y and x) bending
  - Inertia moment of the top slab with respect to y axis I_0.
  - Inertia moment of the web with respect to x axis I_s.
  - Inertia moment of the bottom slab with respect to y axis I_u.

$$I_0 = I_{top,yy} = \frac{t_0 \cdot b^3}{12} = \frac{0.287 \cdot 13.1^3}{12} = 53.767\ m^4$$

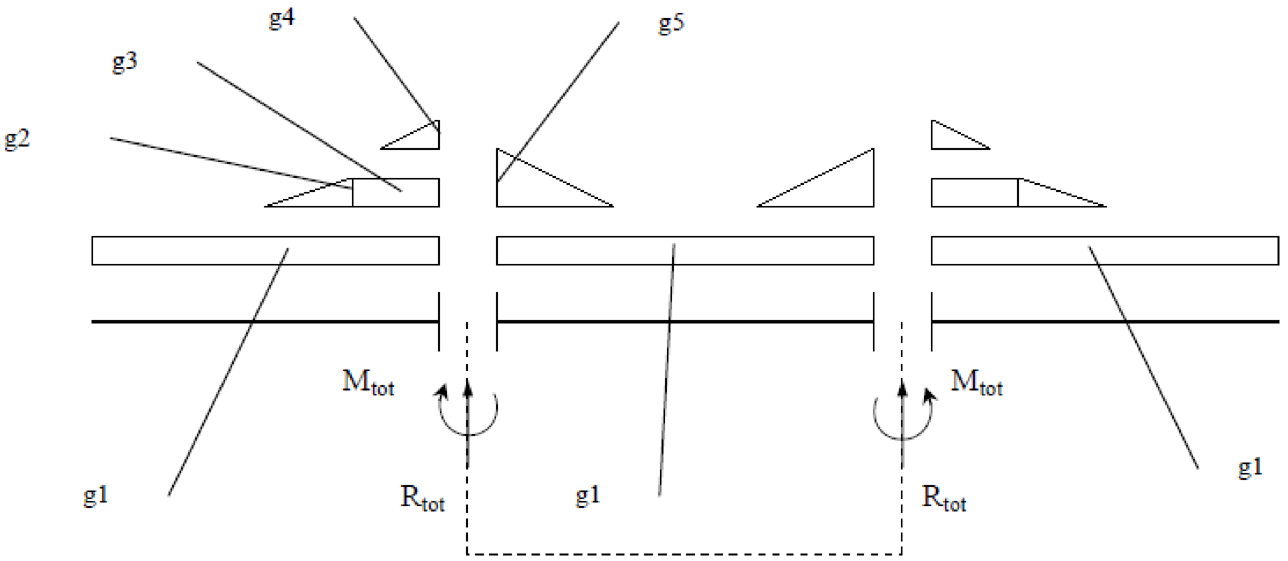
$$I_s = I_{web,xx} = \frac{t_s \cdot b_s^3}{12} = \frac{0.4 \cdot 1.87^3}{12} = 0.217\ m^4$$

$$I_u = I_{bottom,yy} = \frac{t_u \cdot b_u^3}{12} = \frac{0.458 \cdot 5.00^3}{12} = 4.771\ m^4$$

Inertia moments of slabs are referred to the horizontal plane, whereas inertia moment of the web is referred to the vertical plane.

- Transverse bending
  - Transverse inertia moment, i.e. bending inertia of 1 m in the longitudinal direction, of the top slab $\bar{I}_0$.
  - Transverse inertia moment, i.e. bending inertia of 1 m in the longitudinal direction, of the bottom slab $\bar{I}_u$.
  - Transverse inertia moment, i.e. bending inertia of 1 m in the longitudinal direction, of the web $\bar{I}_s$.

To compute it, we decompose the variable thickness section into a sum of uniform and triangular loads and, from this, we compute the bending and the shear reactions.



In the computation, we evaluate the area and the gravity centre of each element composing the section.
The area is multiplied by the areal distribution of gravity load, given by the product between the thickness and the unit weight, and we obtain the weight of each element.
The weight is applied in correspondence of the gravity centre of each element and we can compute its position with respect to the restraint point.

$g_1 = 0.24 \cdot 25 = 6\ kN/m^2 \quad \rightarrow G_1(est) = 6 \cdot 3.85 = 23.1\ kN/m \quad$ distance $G1 = 3.85/2 + 0.2 = 2.12\ m$

$G1(int) = 6 \cdot 4.60 = 27.6\ kN/m$

$g_2 = 0.08 \cdot 25 = 2\ kN/m^2 \quad \rightarrow G_2 = 2 \cdot 1.075/2 = 1.075\ kN/m \quad$ distance $G2 = 1.07/3 + 1.2 = 1.56\ m$

$g_3 = 0.08 \cdot 25 = 2\ kN/m^2 \quad \rightarrow G_3 = 2 \cdot 1 = 2\ kN/m \quad$ distance $G3 = 1/2 + 0.2 = 0.7\ m$

$g_4 = 0.20 \cdot 25 = 5\ kN/m^2 \quad \rightarrow G_4 = 5 \cdot 1/2 = 2.5\ kN/m \quad$ distance $G4 = 1/3 + 0.2 = 0.533\ m$

$g_5 = 0.31 \cdot 25 = 7.75\ kN/m^2 \quad \rightarrow G_5 = 7.75 \cdot 1/2 = 3.875\ kN/m$

Finally, we add the contributions through equilibrium equations to translation and rotation and we find the vertical reaction and the bending reaction at each restraint point.

			(kN/m)		Reactions			
					Left side		Right side	
Left side	q [kN/m]	dist	Right side	q [kN/m]	R [kN/m]	M [kNm/m]	R [kN/m]	M [kNm/m]
G1(sx)	23.10	2.03	G1 (dx)	27.60	28.68	51.19	21.63	16.23
G2	1.08	1.56	G5	3.91				
G3	2.00	0.70						
G4	2.50	0.53						

Total reactions	Rtot	50.30
	Mtot	34.96

By following this solution, we can calculate bending moment and axial forces.

$$m_{AB} = -\frac{1+2\,r_u}{K_3} \cdot \frac{m_R}{2} = -2.759\ \mathrm{kNm/m}$$

$$m_{AD} = m_{AB} + \frac{m_R}{2} = 32.198\ \mathrm{kNm/m}$$

$$m_D = -\frac{r_0}{K_3} \cdot \frac{m_R}{2} = -4.379\ \mathrm{kNm/m}$$

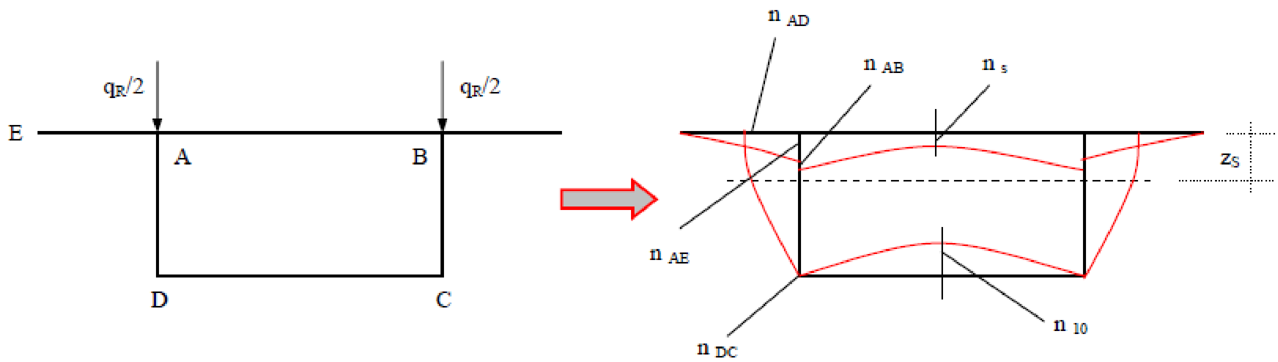
$$n_0 = \frac{3\,r_0(1+r_u)}{K_3} \cdot \frac{m_R}{2} \cdot \frac{1}{d} = 19.592\ \mathrm{kN/m}$$

$$n_u = -n_0 = -19.592\ \mathrm{kN/m}$$

$$n_S = n_u \cdot \sin\varphi = 0\ \mathrm{kN/m}$$

We can notice that the axial force in the top slab n_0 is the same with opposite sign of the axial force in the bottom slab n_u, as there are no horizontal loads and only bending moment, so there should not be horizontal resultant, otherwise we would not have equilibrium.
The same thing applies in the web but they have the same sign of axial force and, in absence of vertical load, the resultant should be null, i.e. axial forces are null. The angle φ is the skewness angle of the web, equal to the angle from the web to the vertical.

Then, we evaluate stresses due to vertical reaction R_{tot}, which is symmetric and is the reaction due to the weight of 1 m of the slab.



According to Schlaich, the vertical force gives rise to axial force distribution, having a certain shape. The shape remembers Jourawski's formulation for shear – nil tangential stresses at the extremes and increase of them with the static moment. Indeed, this axial force is a relative of the shear because the computation of shear stresses in a closed box section is complicated, as it is a hyperstatic shape, and these axial forces can be seen as shear for the bridge – a vertical force in the web is an axial force for the a portion of the web itself and a shear for the bridge. Thus, these actions remind the shape of the shear because they are shear actions.
The distribution is curved because we are dealing with warping, i.e. distortion of the section, and we refer to a sort of static moment of the area with respect to the centroid of the section.

$$n_{AB} = n_{AE} = 41.129 \text{ kN/m}$$
$$n_{DC} = 0 \text{ kN/m}$$
$$n_S = n_{AD} + \frac{q_R}{I_X}\left(\frac{t_s\ b\ z_s^{\,2}}{2} + \frac{t_s\ z_s^{\,3}}{3}\right) = -16.022 \text{ kN/m}$$
$$n_{10} = \frac{q_R}{8I_X} t_u\ b_u^{\,2}\left(b_s - z_s\right) = 39.646 \text{ kN/m}$$
$$n_s = n_{AB} - \frac{q_R}{8I_X} t_0\ b_0^{\,2}\ z_s = 25.458 \text{ kN/m}$$

In this way, we get the value of the axial force in each section and each section has an axial force related to shear and a transverse bending moment, coming from actions q_R and M_R.

This operation takes into account only the self-weight of the top slab.

Section / Caused by	AB N	AB M	AD N	AD M	DA N	DA M	DC N	DC M	AE N	AE M
Fixed	0.00	-16.23	0.00	0.00	0.00	0.00	0.00	0.00	0.00	-51.19
Qr	41.13	0.00	-50.30	0.00	0.00	0.00	0.00	0.00	41.13	0.00
Mr	19.59	-2.76	0.00	32.20	0.00	-4.38	-19.59	-4.38	0.00	0.00
Total	60.72	-18.99	-50.30	32.20	0.00	-4.38	-19.59	-4.38	41.13	-51.19

STRESSES DUE TO PERMANENT LOADS

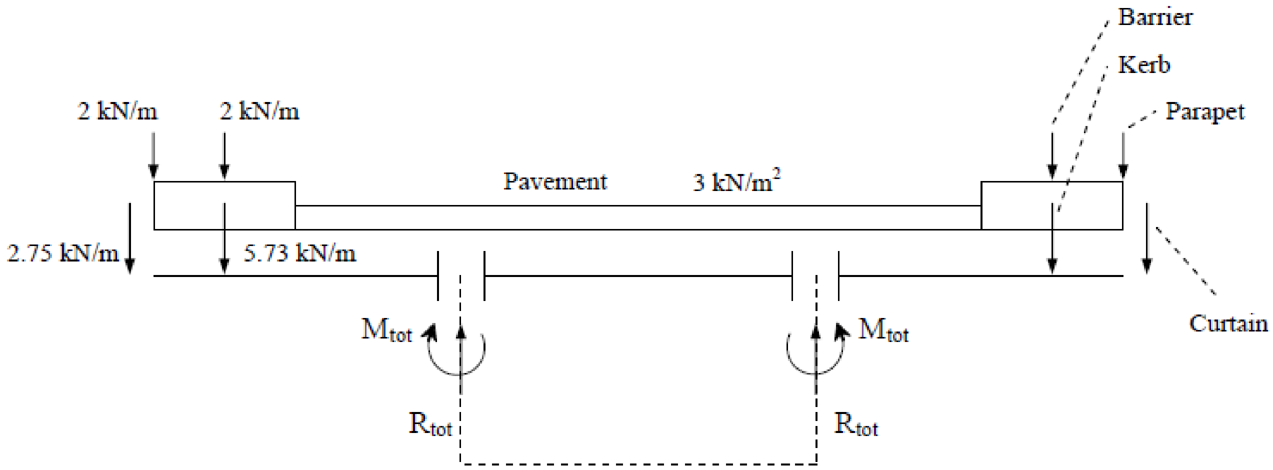
1. Rigid restraint reactions evaluation

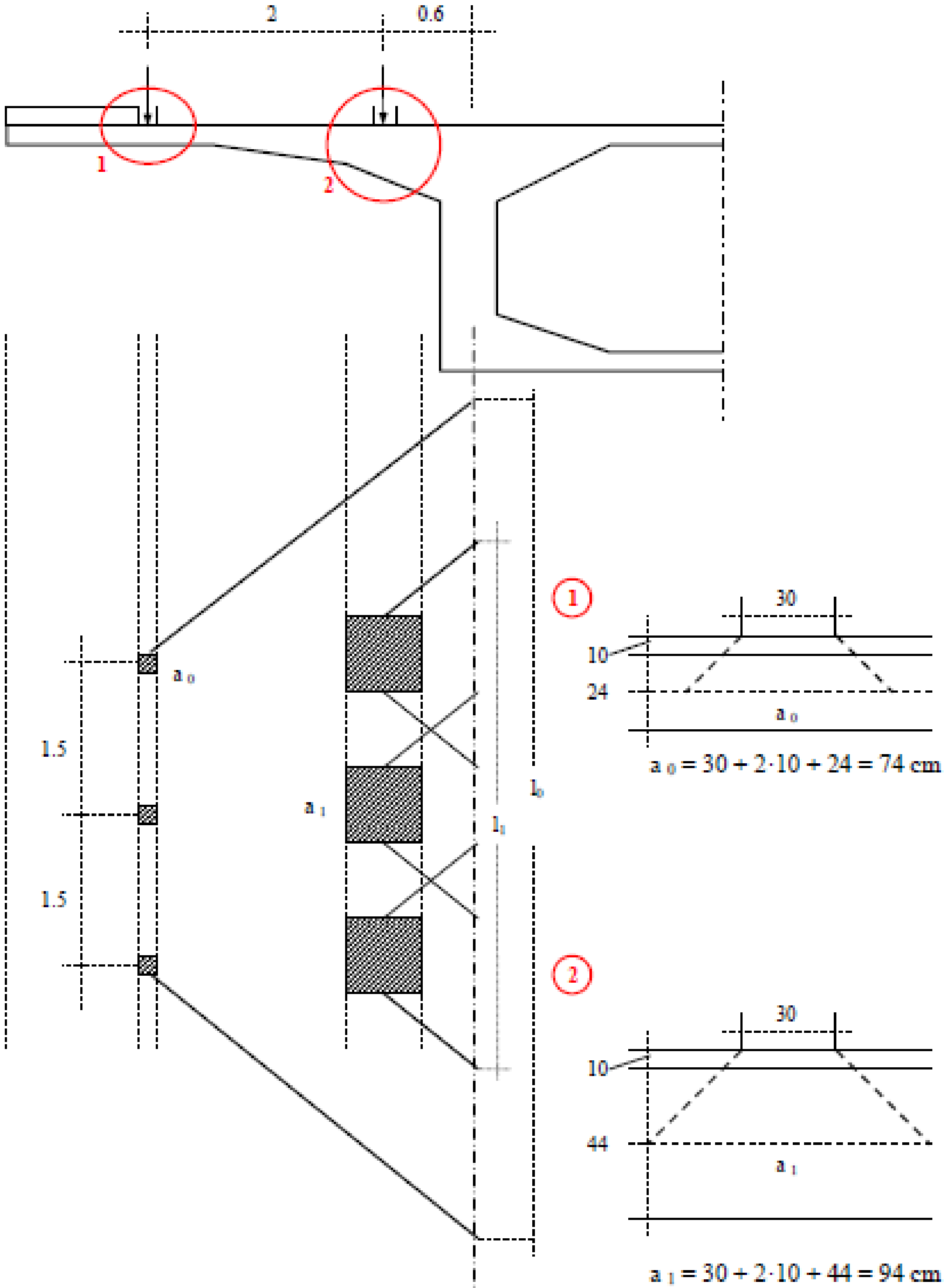
Then, we pass to permanent loads, which are symmetrical loads applied on the top slab.

As before, the effect of any action applied to the top slab is evaluated considering, in a first step, the top slab rigidly restrained in the webs, with

- Cantilevering part of the top slab rigidly restrained at each web.
- Central part of the top slab rigidly restrained at each web.

The remaining part of the bridge is not considered in this stage.



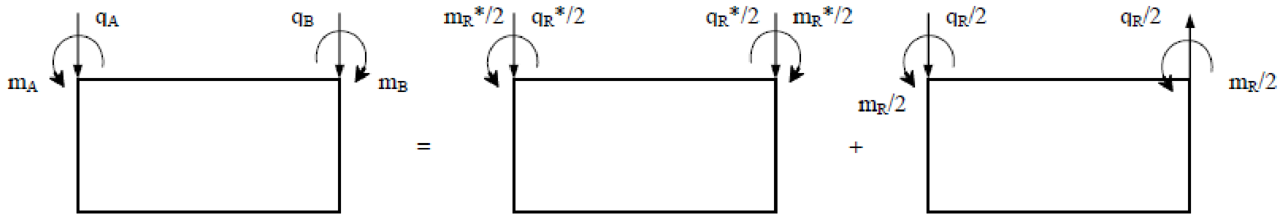


The wheel is the same but the representation does not show the footprint – squared area 30 cm × 30 cm – but the imprint in the centroid axis of the top slab. This imprint is bigger due to diffusion inside the pavement and on one half of the thickness of the top slab, which is different in the two positions of the wheels.

- The left wheels diffuse over a global thickness of 10 cm + 12 cm and the imprint is
$$a_0 = 74\ cm$$
- The right wheels diffuse over a global thickness of 10 cm + 22 cm and the imprint is
$$a_0 = 94\ cm$$

- Asymmetric load

$$q_R = q_A - q_B$$
$$m_R = m_A - m_B$$



We compute the symmetric load and asymmetric load in points A and B, taking into account that the load is applied only in the cantilever on the side of point A.
Indeed, the reactions q_B and m_B due to traffic load in point B are null, since the load is completely applied on the cantilever on the other side, whereas its cantilever is unloaded, and actions are only present in point A.

In A				In B			
Reaction				Reaction			
Left		Right		Left		Right	
R	M	R	M	R	M	R	M
128.69	171.17	0.00	0.00	0.00	0.00	0.00	0.00

Reactions	A
R	M
128.69	171.17

Reactions	B
R	M
0.00	0.00

Symmetric part	qr*	128.69	kN/m
	mr*	171.17	kNm/m
Asymmetric part	qr	128.69	kN/m
	mr	171.17	kNm/m

In the evaluation of the symmetric load and asymmetric load, we can notice that their value is the same because, in point B, the resultant is null and so symmetric load and asymmetric load must be equal.
This is a particular case, related to the fact that the load is completely applied on one cantilever. If the wheel was inside the box, there would be also a reaction in point B and symmetric load and asymmetric load would be different.
Having decomposed the effect of traffic load into symmetric and antisymmetric components, we evaluate the internal actions due to each contribution.

<u>Stresses due to the symmetric part of the load</u>

This load has the same configuration of the symmetrical part of self-weight or permanent loads. By consequence, to evaluate internal actions coming from the symmetrical set of forces due to traffic, we adopt the same distribution of internal forces of before.

Thus, these stresses do not need calculation because they are given by the software.

Actually, the software knows the cross section and uses the hollow core section as full body section. In this way, it computes the primary torsion, but it ignores secondary torsion and Bredt anomaly. Some advanced finite element codes have special beam elements, designed for hollow sections with warping, in order to take into account Bredt anomaly in an automatic way. Yet, the reference theory might be different, with different assumptions and different results, even because the approach used in this field is approximated and hand-user.

The forces inside the single elements are computed according to Bredt approach.

$$\Delta M_t = \frac{q_R}{2} b_0 + m_R = 492.89\ \mathrm{kNm/m}$$

$$q_{0t} = \frac{\Delta M_t}{2 \cdot A_K} b_0 = 132.00\ \mathrm{kN/m}$$

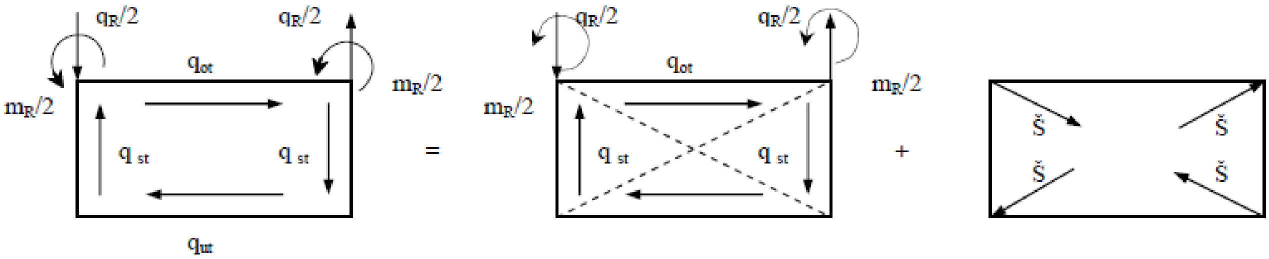
$$q_{St} = \frac{\Delta M_t}{2 \cdot A_K} b_S = 42.29\ \mathrm{kN/m}$$

$$q_{ut} = \frac{\Delta M_t}{2 \cdot A_K} b_u = 132.00\ \mathrm{kN/m}$$

Distortion part

The distortion part is then divided into two components, as combination of a longitudinal and transverse load bearing capacity.

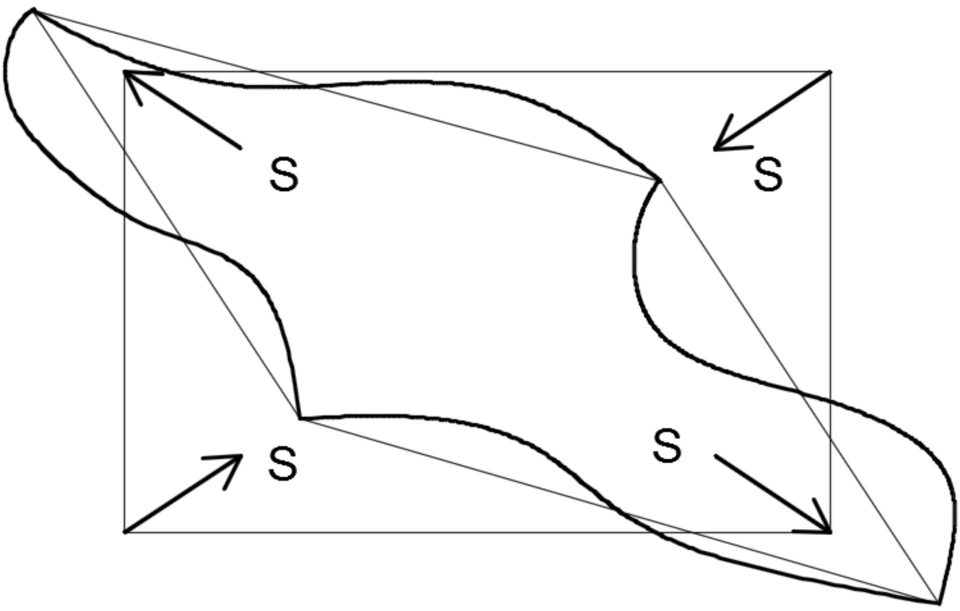
- Local effect, represented by a set of forces S, equal and opposite, acting along the diagonals.
- Global effect – folded plate – related to a system of forces with the distortion part and the diagonal forces, changed in sign.



LOCAL EFFECTS

GLOBAL EFFECT (FOLDED PLATE)

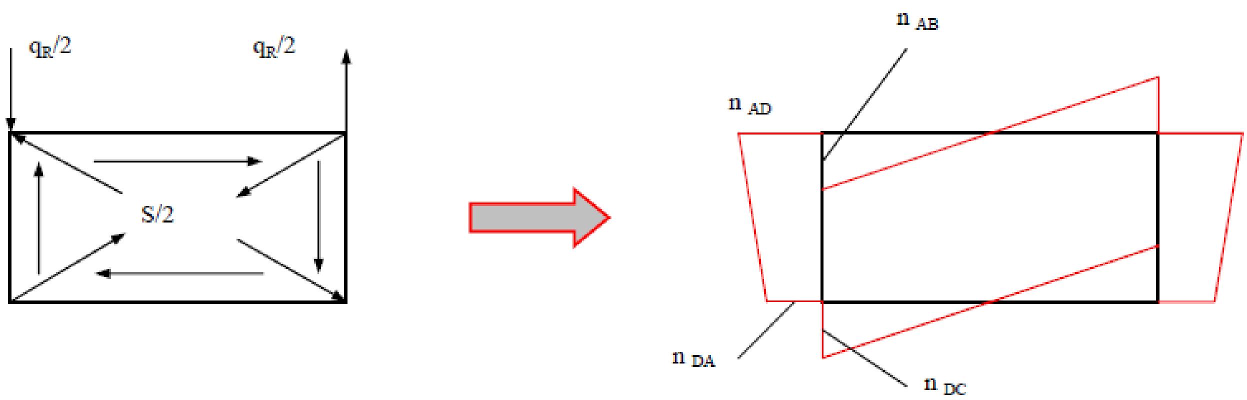
Indeed, if we consider the equilibrium of the single element of the box section under the distortion forces, e.g. equilibrium in vertical direction of the web at left, we can notice that there is no equilibrium.
On the other side, if we combine the load, Bredt forces and diagonal forces, each element will be in equilibrium.
Diagonal forces S are forces exchanged within the structure – between its elements – and this forces ensures the equilibrium both of the structure and of the single elements.



At this point, we evaluate the effect of each force.

In the local system, we consider the vertical load $\frac{q_R}{2}$, the diagonal forces $\frac{S}{2}$ and Bredt forces. The effect of this set of forces to the box section is described by a diagram, obtained by Schlaich's theory.

Axial forces assume the same values in the opposite elements. For instance, n_{AD} is the axial force in point A inside the web AD, whereas n_{DA} is the axial force in point D inside the web AD.



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$$K_4 = (r_0 + 2)(r_u + 2) - 1 = 34.00393$$

$$S = -\left(\frac{\beta}{1+\beta} + \frac{r_0 - 3 \cdot \beta - 2 \cdot \beta\, r_u}{K_4} \right) \cdot \frac{g\, m_R}{b_u d} = -50.89 \text{ kN/m}$$

$$m_{AB} = -\frac{3 + 2\, r_u}{K_4} \cdot \frac{m_R}{2} = -16.53 \text{ kN/m}$$

$$m_{AD} = m_{AB} + \frac{m_R}{2} = 69.05 \text{ kN/m}$$

$$m_D = -\frac{r_0}{K_4} \cdot \frac{m_R}{2} = -18.24 \text{ kN/m}$$

$$\cos\vartheta = \frac{4\, d^2 + b_0^2 - b_u^2}{4\, g\, b_S} = 0.3498$$

$$n_{AB} = \frac{m_R}{2\, d\, (1+\beta)} = 22.92 \text{ kN/m}$$

$$n_{AD} = \frac{S}{2} \cos\vartheta + \frac{2\, m_{AB}}{b_0} \cos\varphi - n_{AB} \operatorname{sen}\varphi = -15.51 \text{ KN/m}$$

$$n_{DA} = n_{AD} + \frac{m_R\, b_S}{d\,(b_0 + b_u)} = 1.60 \text{ kN/m}$$

$$n_{DC} = -\beta \cdot n_{AB} = -22.92 \text{ kN/m}$$

The diagonal force S is divided in two equal parts and one half is used in the model with vertical forces and the second half is used in the model with couples. In this way, under this load condition, we get axial forces and bending moment.

The result is the set of internal actions related to the local effect scheme.

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

Focusing on the warping part, the forces S are given by the other sections and they have to be transferred, in order to get full equilibrium.
The value of these forces is

The term $EI_{S,i}$ is bending stiffness of the deck and K is the equivalent stiffness of the vertical frame and it is opposed to the distortion movement. Stiffness K is given by the following expression.

$$K = \frac{24b_S}{b_u^2 \cdot d^2} \cdot K_1 \cdot K_2 \cdot E \cdot \bar{I}_S = 9.27 \cdot 10^4$$

$$I_{Si} = \frac{2 \cdot \beta[(\alpha_0 + 2)(\alpha_u + 2) - 1]}{(1+\beta)(3 + 3\beta + \alpha_0 + \alpha_u \cdot \beta)} \cdot I_S = 0.916\,m^4$$

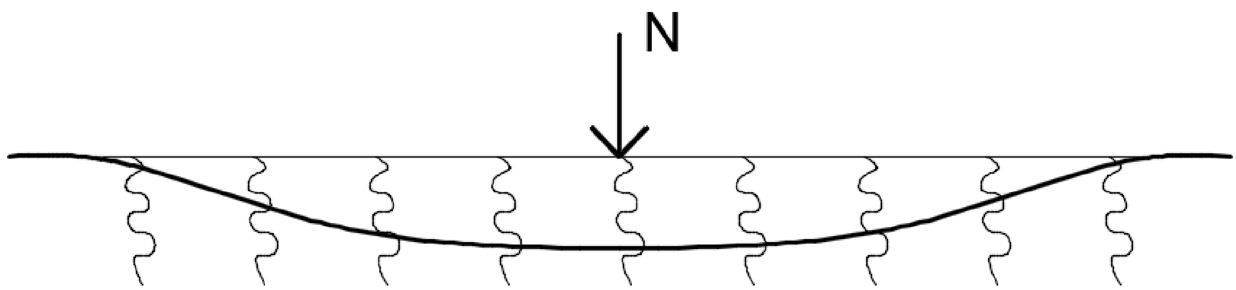
$$K_1 = \frac{(1+\beta)(2 + 2\beta + 2\beta^2 + \alpha_0 + \alpha_u \cdot \beta^2)}{3 + 3\beta + \alpha_0 + \alpha_u \cdot \beta} = 2$$

$$K_2 = \frac{2 + 2\beta + 2\beta^2 + r_0 + r_u \cdot \beta^2}{\beta[(r_0 + 2)(r_u + 2) - 1]} = 0.442$$

With reference to the beam on elastic foundation, the term $EI_{S,i}$ is equivalent to bending stiffness of the beam and K is equivalent to soil stiffness, which opposes the displacement.

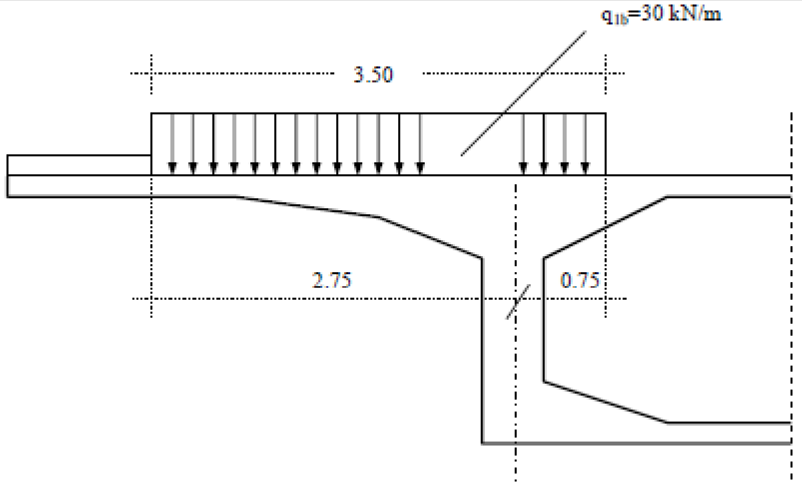
Where is the analogy?

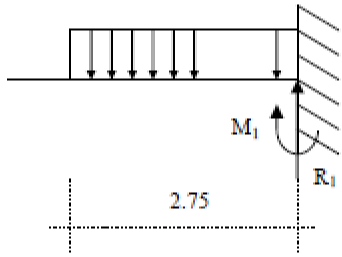
A box shaped beam with longitudinal axis x is subjected to an eccentric load on a segment. By consequence, torque moment will be applied only on that segment.
The remaining part of the structure is opposing to the torsion of the section, exactly as a beam on elastic foundation, where the force is applied on one point and the loaded segment is not going freely – only regarding on the stiffness of the springs under the segment. Due to beam stiffness, the other springs are requested to help and to limit the vertical displacement of the point of application of the load.

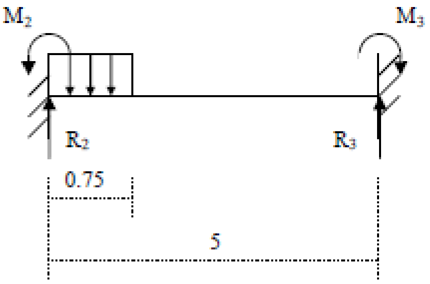


In case of warping, the situation is the same. A segment is subjected to a concentrated torque moment and, if considered alone, it would have big distortion. Since it is connected with the other part of the deck, this helps the segment to have torsion and the equation shape is the same.

Thus, we can say that warping force q_S on a section is linearly related to the displacement of the section and to its fourth derivative.







$$R_1 = 8.57 \cdot 2.75 = 23.567 \text{ kN/m}$$

$$M_1 = 8.57 \cdot \frac{2.75^2}{2} = 32.405 \text{ kNm/m}$$

$$R_3 = \frac{8.57 \cdot 0.75^3}{2 \cdot 5^2}\left(1 + \frac{4.25}{5}\right) = 0.134 \text{ kN/m}$$

$$R_2 = 8.57 \cdot 0.75 - 0.134 = 6.294 \text{ kN/m}$$

$$M_3 = 8.57 \cdot 0.75^2\left(\frac{0.75}{3 \cdot 5} - \frac{0.75^2}{4 \cdot 5^2}\right) = 0.268 \text{ kNm/m}$$

$$M_2 = 8.57 \cdot 0.75^2\left(\frac{1}{2} - \frac{2}{3}\frac{0.75}{5} + \frac{0.75^2}{4 \cdot 5^2}\right) = 1.955 \text{ kNm/m}$$

$$\Longrightarrow$$

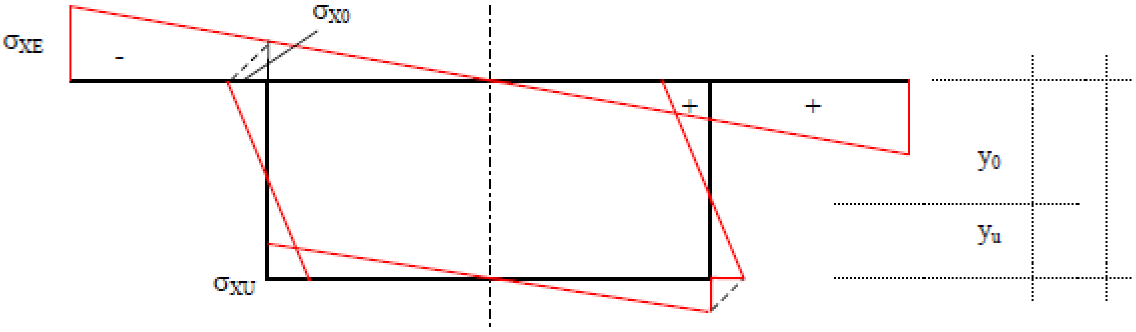
$$q_0 = 16.09 \text{ kN/m}$$

$$q_S = 6.01 \text{ kN/m}$$

$$q_u = 16.09 \text{ kN/m}$$

The distribution of the loads in the beam is quite variable in the longitudinal direction and we have the solution in terms of internal actions.

BECAUSE OF THE EFFECT OF THE BENDING MOMENT IT APPEARS OTHER LONGITUDINAL NORMAL STRESSES THAT MUST BE ADDED TO THAT FROM THE CLASSICAL LONGITUDINAL ANALYSIS



$$y_0 = \frac{1 + 2\beta + \alpha_u \cdot \beta}{3 + 3\beta + \alpha_0 + \alpha_u \cdot \beta} \cdot b_S = 0.260 \text{ m}$$

$$y_u = b_S - y_0 = 1.607 \text{ m}$$

$$\sigma_{X0} = -\frac{M_S(x)}{I_{Si}} \cdot y_0 = -8.0 \text{ kN/m}^2$$

$$\sigma_{XE} = \sigma_{X0} \frac{b/2}{b_0/2} = \sigma_{X0} \frac{b}{b_0} = -21.1 \text{ kN/m}^2$$

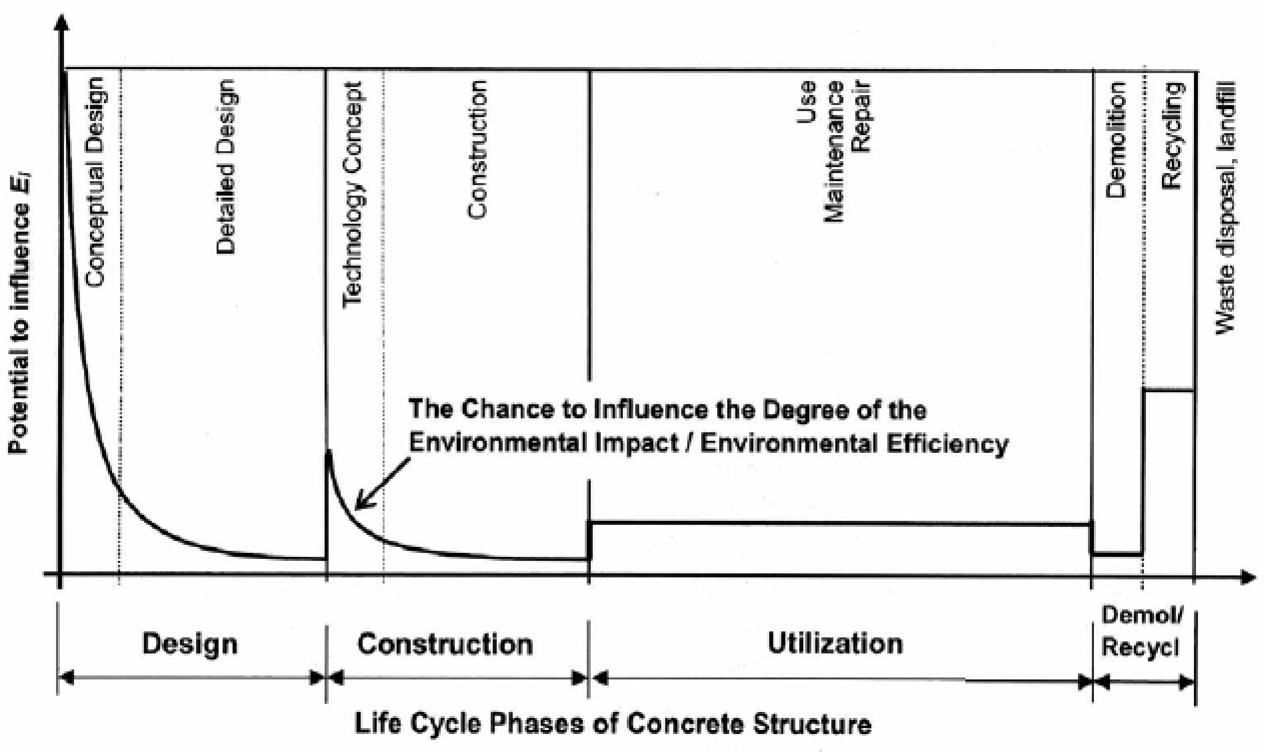
$$\sigma_{XU} = \frac{M_S(x)}{I_{Si}} \cdot y_u = 49.8 \text{ kN/m}^2$$

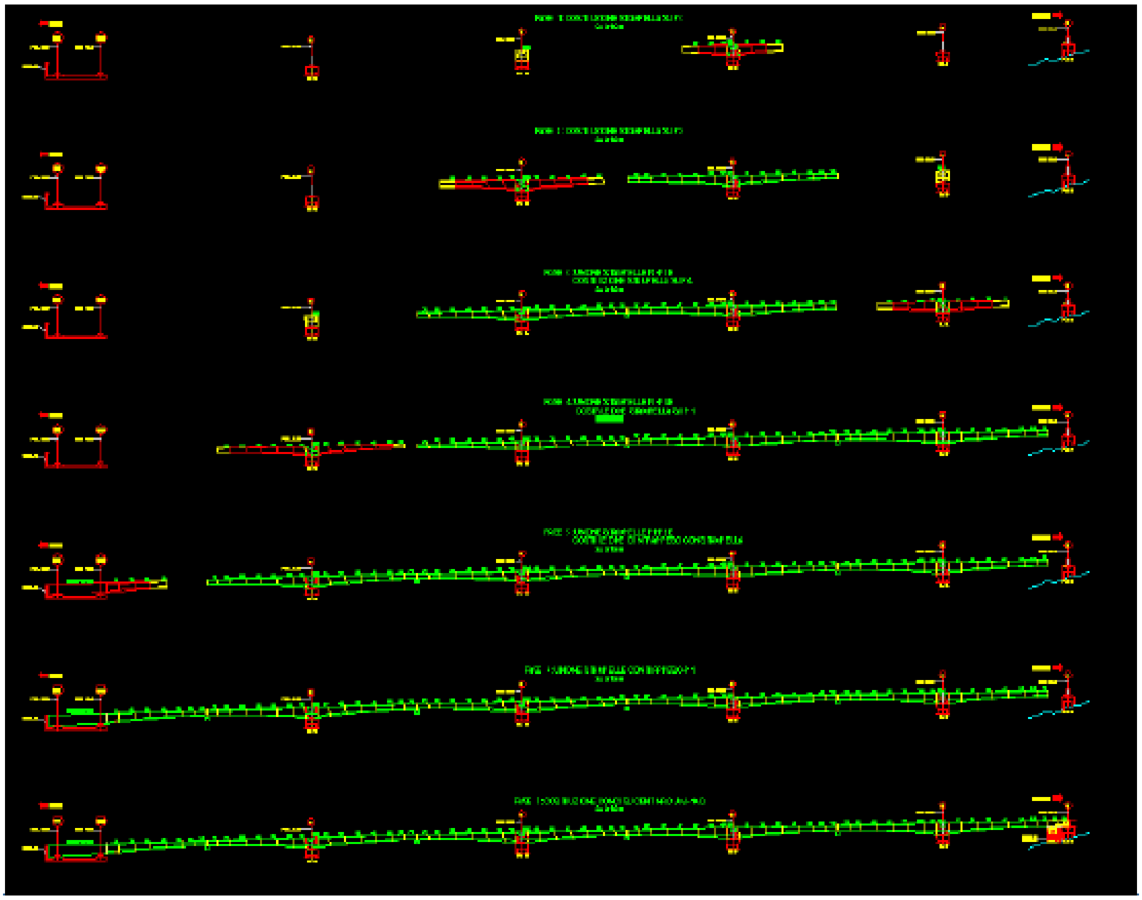
By using steel webs, total weight of the bridge is reduced by 25%. This implies the reduction of the volume of concrete produced and the energy used to move concrete from the production place to the site.

2. Ground consumption
   In case of box shaped bridge, we can adopt balanced cantilever segmental construction. In this way, we do not need space to store segments and to modify heavily the ground below the bridge.
3. Dismounting and recycling
   When dismounted, concrete is difficult to be re-used because it should be crunched and it can be used only as aggregate for new concrete. The recycle grade will never be 100%, otherwise mechanical properties would be bad – the maximum is 30%.
   On the other side, steel can be recycled 100%, preserving mechanical properties.

In bridge design, different factors are influencing the environmental impact.

If we put on x axis the life cycle phase of bridge, divided in design time, construction time, utilization time and demolition and recycling time, on y axis we may put the potential chance we have to reach a sustainable construction.
The maximum value is the best chance to get sustainability, which is placed in correspondence of <u>conceptual design</u>, i.e. the choice of the structural solution.
The executive design is less important.
In construction stage, most chance to reduce environmental impact is in the <u>choice of technology</u>.
In utilization stage, environmental impact derives from <u>maintenance</u> since, less maintenance we need, less will be the impact. For instance, if we reduce the number of joints and bearings, we will save money and energy.
<u>Demolition and recycling</u> stage should also be taken into account in design because, if it is concerted in a clever way, the dismount will be easy.



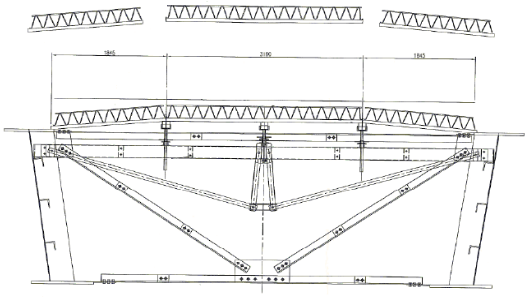


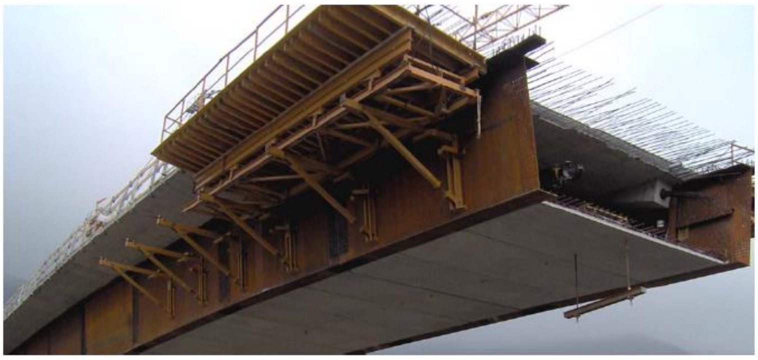
Focusing on the deck construction procedure, we identify different steps.

1. *Pier construction*
   Piers normal straightforward boxed hollow core elements, which are thinner than the ordinary ones due to the smaller weight of the deck.
2. *Pier segment*
   On the top of the pier, we place the pier segment.
   The pier segment is a steel frame, centred on the axis of the pier, with webs and two transverse diaphragms.

This segment is built into a factory and, since it is quite light, it can be lifted by a normal crane – we do not need specific cranes.
On the top of the pier, some holes are used to connect the pier segment to the pier by means of vertical tendons and to fix to it during construction. Indeed, in balanced cantilever

Inside them, we realize a temporary steel structure, in order to sustain the weight of the top slab. When the slab is hardened, we remove it and place it in the following segment.
Furthermore, we place a scaffolding for the cantilevering part of the segment, which is fixed to the web and then moved forward for each segment.





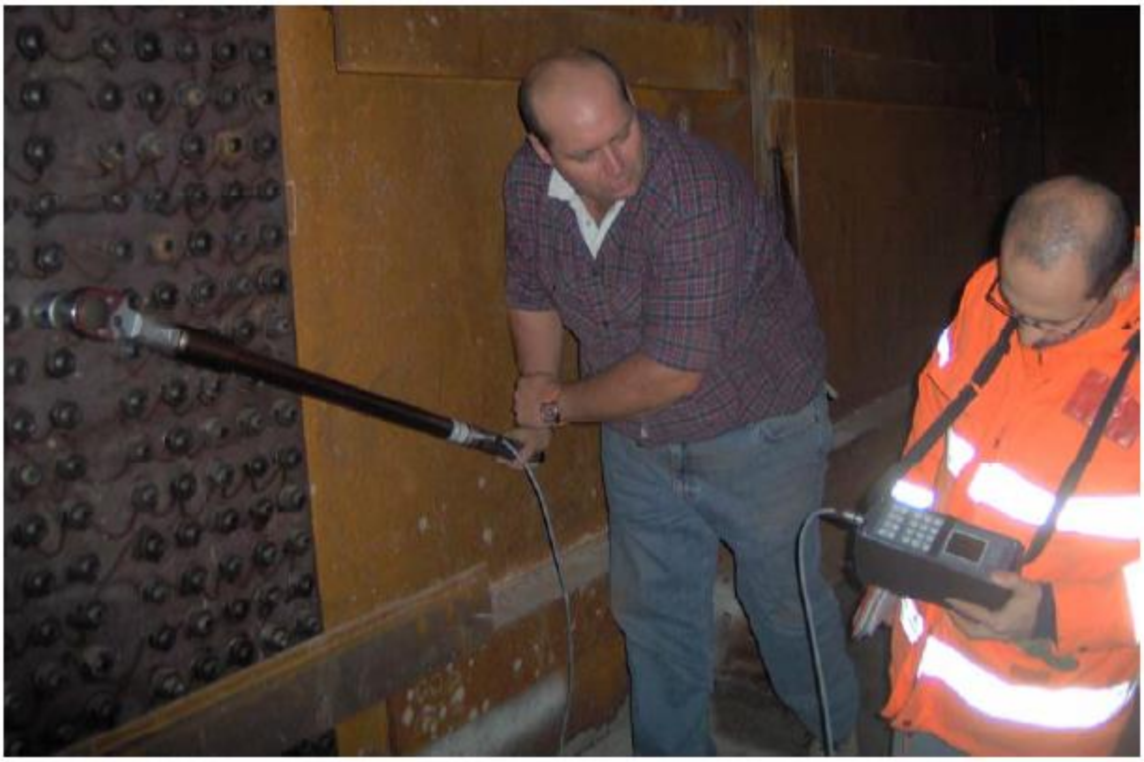
When all predalles are placed, we concrete the bottom slab of the segment and, when hardened, we concrete the top slab.
At the end, we remove the scaffolding and start a new segment.
With respect to traditional technique, the length of the segments is bigger – 7 ÷ 9 m – and nothing heavy is lifted – we use predalles and steel webs – and, by consequence, we do not need any expensive crane. We just need a pump to pump concrete in the segment.

4. *Prestressing of the deck*
   Prestressing layout is similar to the one of Segno Viaduct, with negative prestressing tendons placed in the top slab.
   In this case, the blister – also visible in the figure with external scaffolding – is prefabricated.

6. *Planimetric and altimetric adjustement*
   At the end of construction of the hammer, seeing from inside the box, the hammers are completed and close to each other – we are at joint. In the figure, we can see a web belonging to one hammer and a web belonging to the other hammer.

We have to keep the hammers aligned and closed to each other, in order to close the joint. The joint is closed by means of a steel plate. To do it, theoretically, we should check for the perfect alignment of the webs and the matching of the holes. Actually, it is impossible and, by consequence, we realize holes on the plate, we measure the distance between each hole, we realize a mask and, through it, we realize the holes in the web. If we did not follow this strategy, we never would reach the alignment of the holes.
The alignment of the webs is difficult to be reached and there are different strategies.

- Control of web construction



Tightening of the bolts in the web joints using dynamometric spanner

7. *Introduction of continuity prestressing*
   When hammers are joined, we introduce a further amount of prestressing, which is the continuity prestressing.
   Hammers are connected by black ducts having the continuity prestressing, which follows a certain layout.