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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

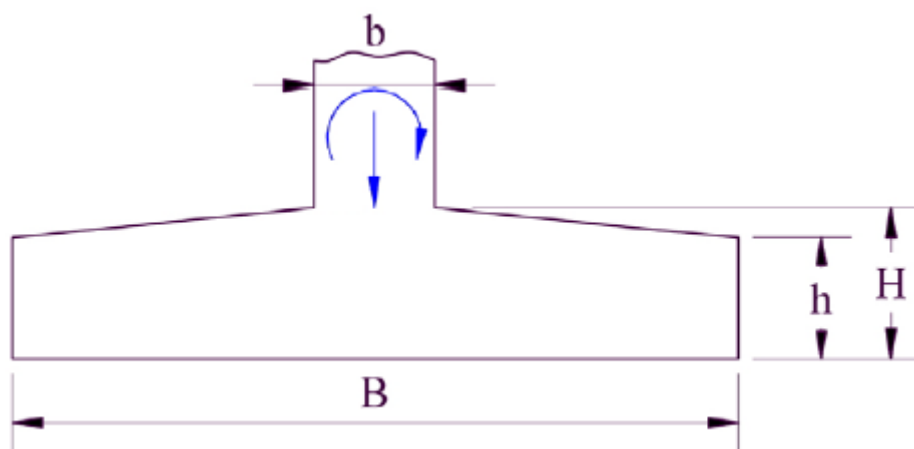
Given the pier dimension  $b$ , the larger dimension of the plinth  $B$  should be not too much bigger – in this way, the cantilever part is not important.

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

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.



Footing  
reinforcement 2



Footing  
reinforcement  
and mould



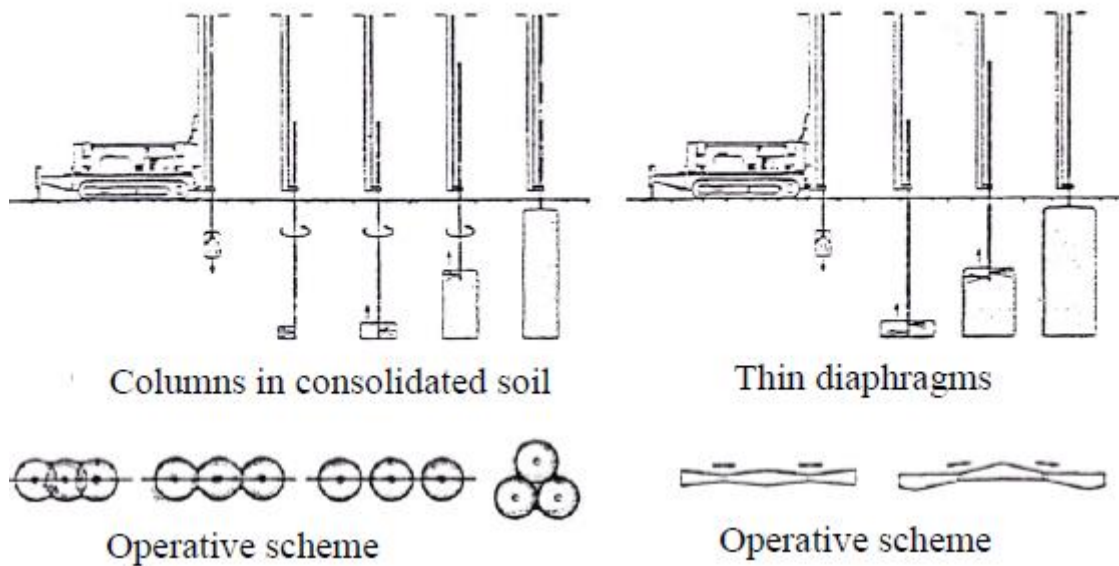
Concreting of the footing



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.

## OPERATING SYSTEMS



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.

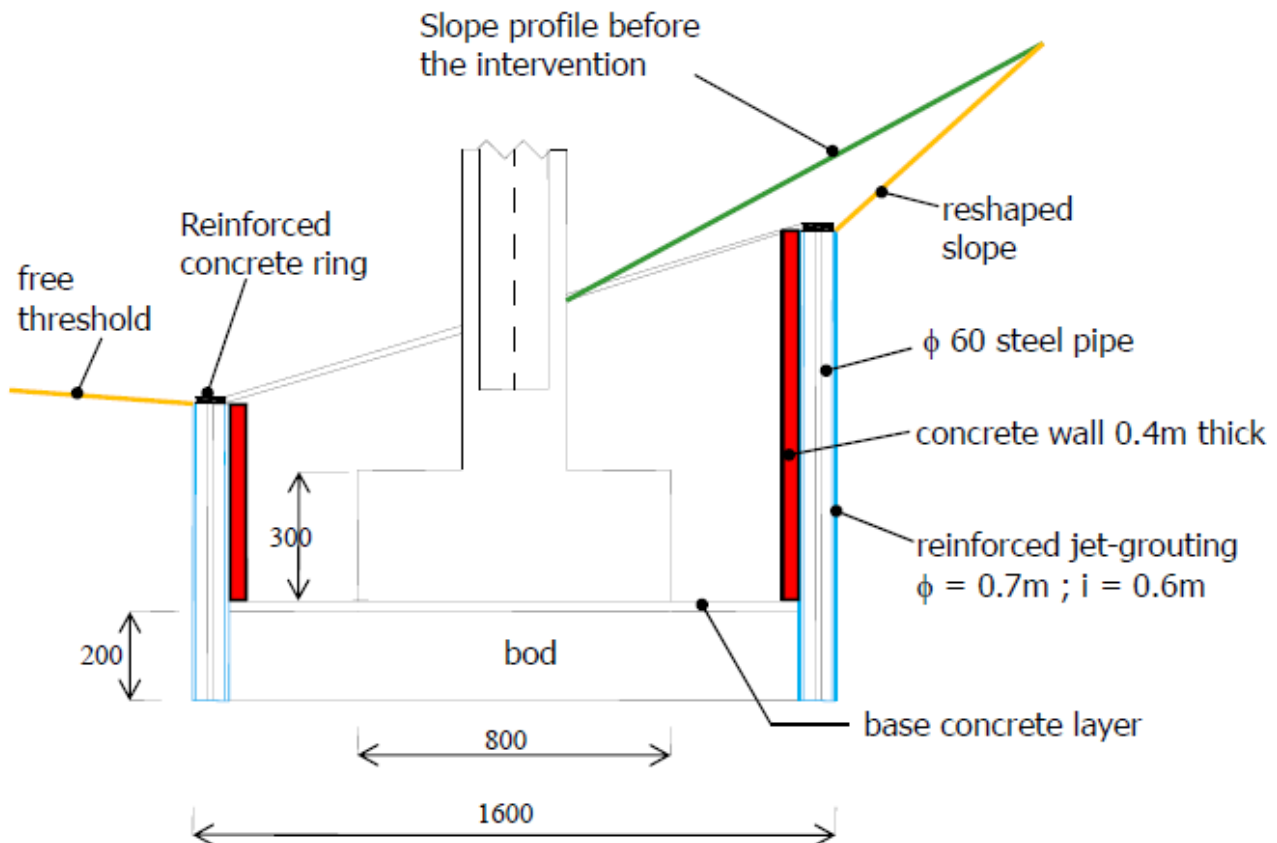
In case of cohesive soil, the only risk is not to obtain big columns or columns with the same diameter along the depth due to cohesion, which has to be destroyed by injection.

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 $\phi$ ) [mm]	Extraction speed [cm/min]	Rotation speed [rpm]	Concr ete/Wa ter	Flow [l/min]
<b>MONO-FLUID</b>	mortar	400-550	1-2 x 2-5	15-100	5-15	1.0-1.5	70-600
<b>TWO-FLUIDS</b>	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
<b>THREE- FLUIDS</b>	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.

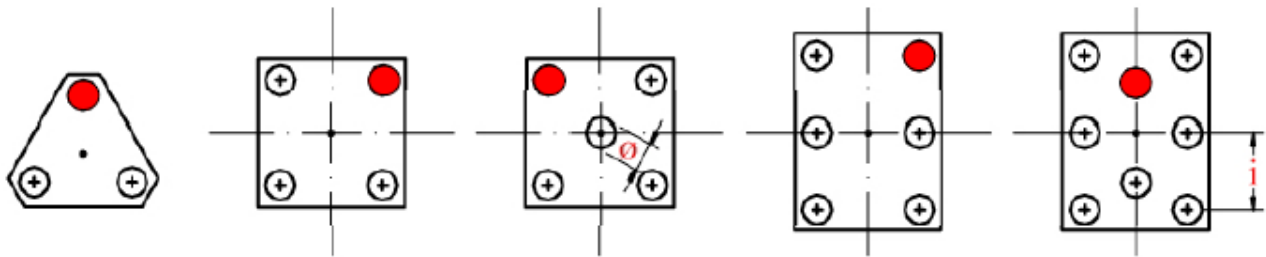
#### Example of application

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 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  $(2 \div 3)\phi$  – 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  $3\phi$ , we can have collapse of a single pile.

We can define an efficiency factor  $\gamma$  for the piles, which is the ratio between the actual pile system collapse load and the sum of the single pile collapse load.

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

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  $\gamma$ .

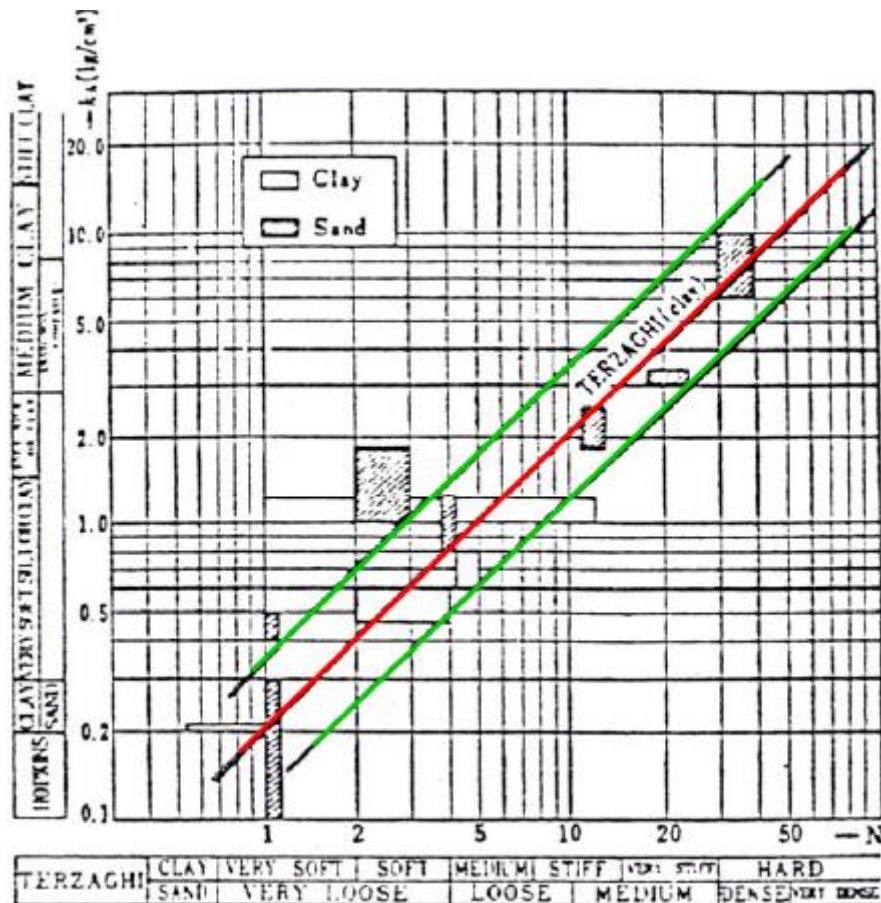
The typical situation corresponds to an interaxis  $i$  equal to  $3\phi$  and we can assume the following value.

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

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  $D$ .

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



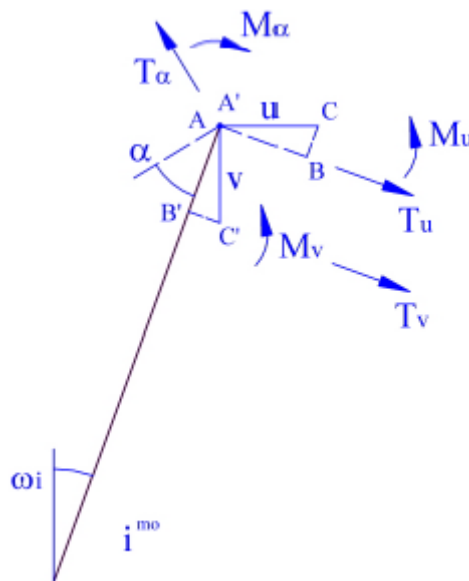
By using this diagram, we can estimate the Winkler modulus and then solve the problem of beam on an elastic soil, getting the internal actions inside the pile combined with the vertical load.

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.

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

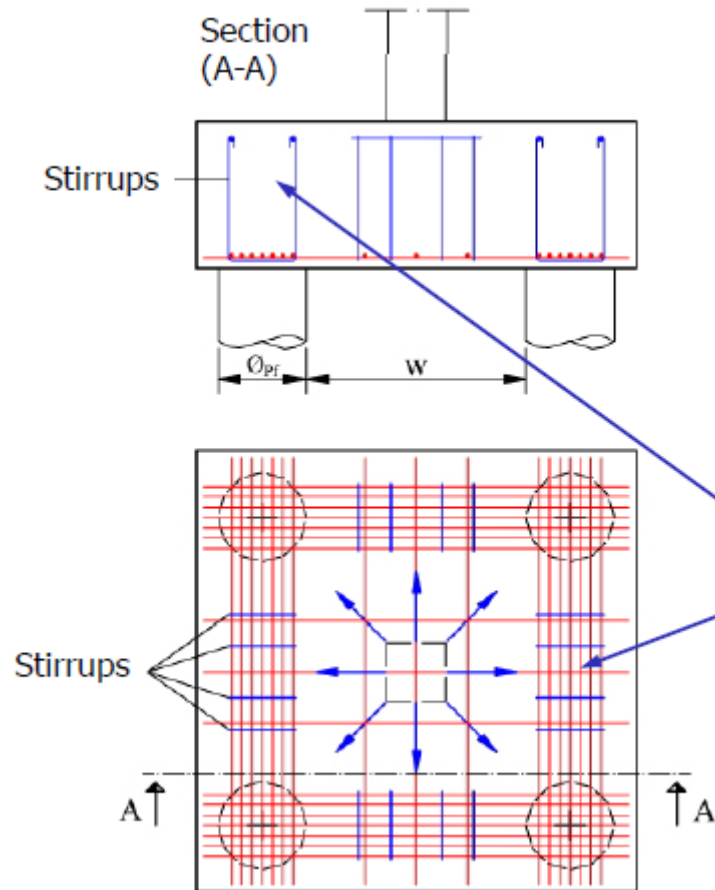
$$X_i = -P_i \sin \omega_i + H_i \cos \omega_i$$

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.

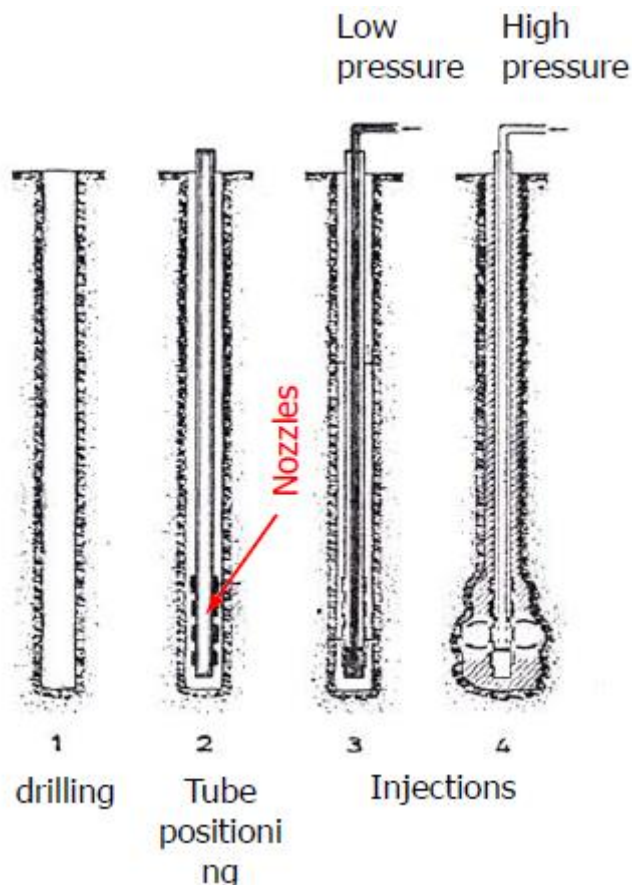
No normal piles are used, since their realization requires big machines, space and huge quantity of bentonite, in case of bored piles – it maintains the geometry of the hole during excavation. In other words, their realization would be complicated.

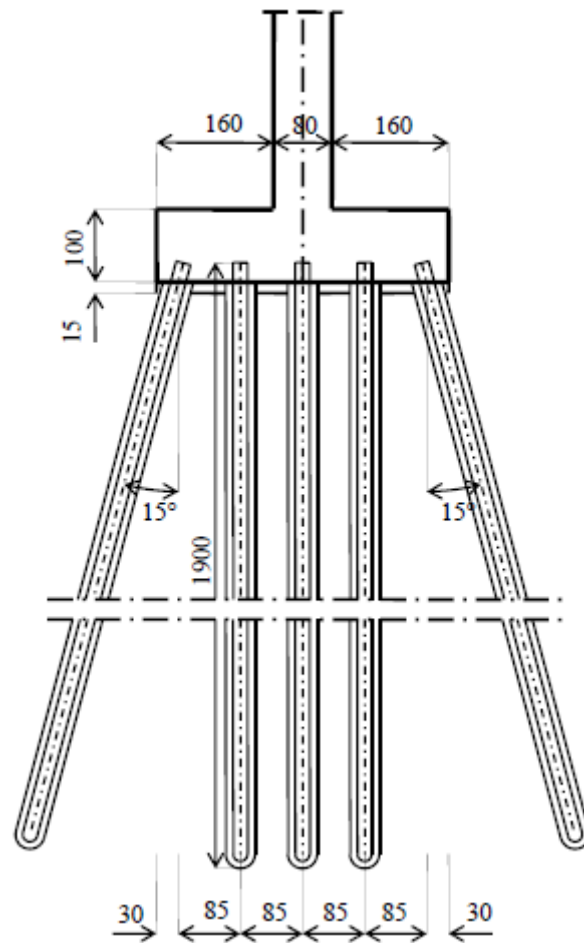
Moreover, micropiles are used in case of improvement of existing structures because their realization does not requires huge machinery and it interacts in a small way with surrounding elements.

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 –  $30 \div 40$  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  $4 \div 5$  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

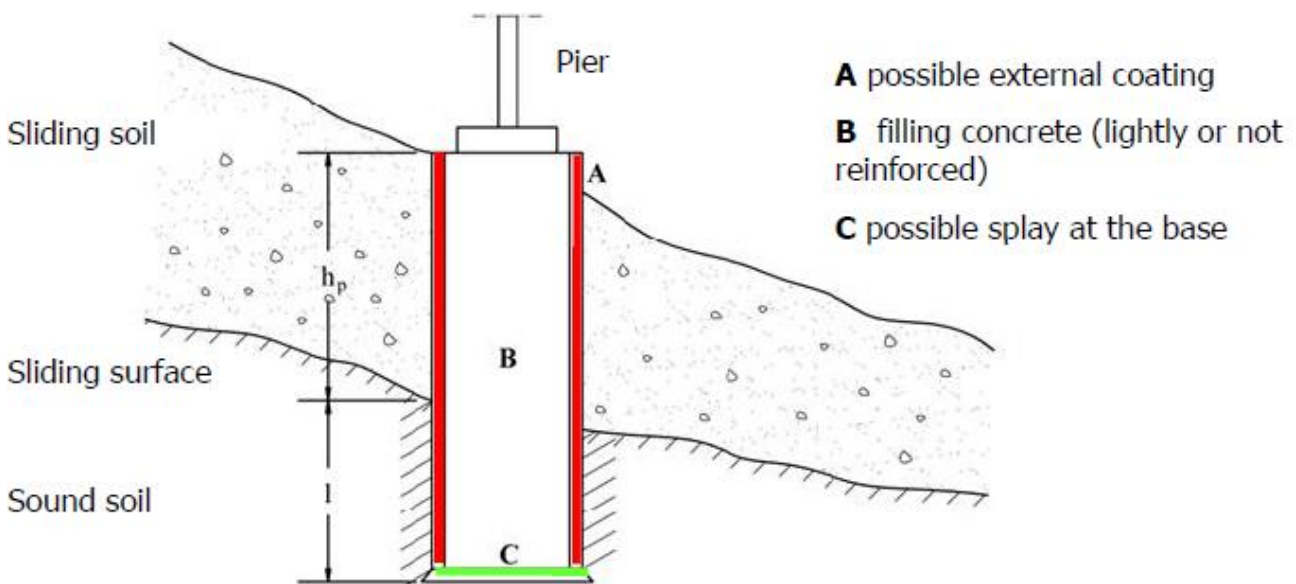
Shaft foundations are an expensive solution used for bridge foundation in presence of large vertical and horizontal actions, due to different reasons.

- 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



Hopping  
before  
spritzbeton



Underpinning  
detail



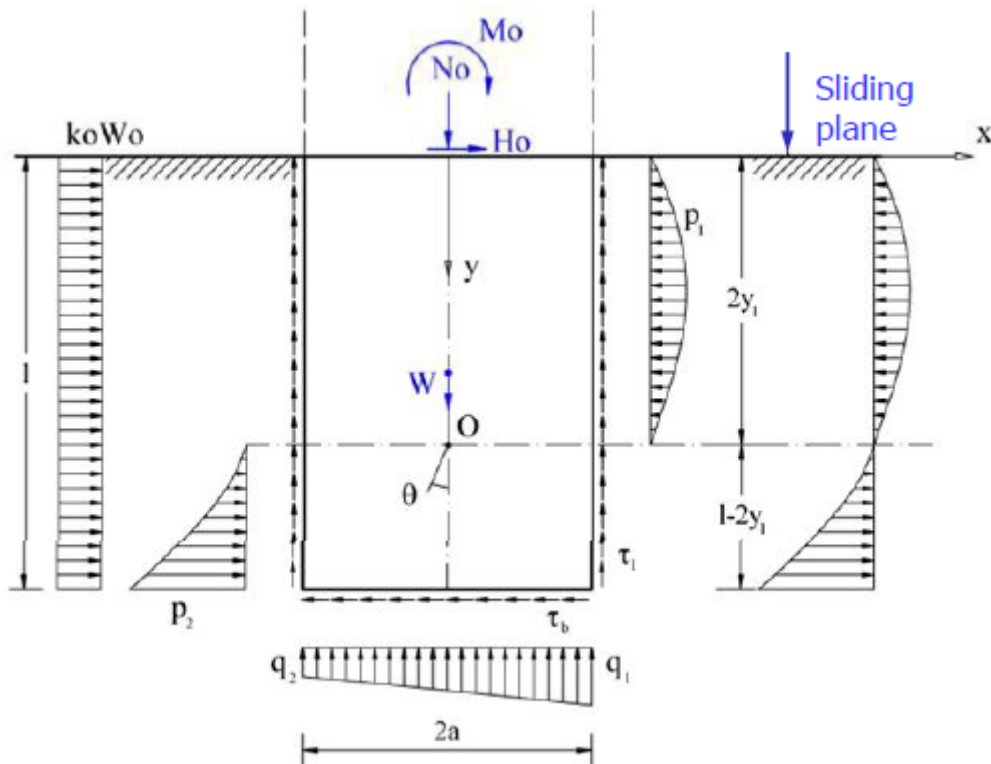
Digging at  
-23m



Cleaning up after emptying the flooded pit



Shaft bottom



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  $K_0 W_0$ , 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  $\tau_l$  on the lateral surface of shaft.
- Tangential stresses  $\tau_b$  on the base of shaft.

Generally, the evaluation of pressures  $p$ ,  $q$ ,  $\tau_l$  and  $\tau_b$  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

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

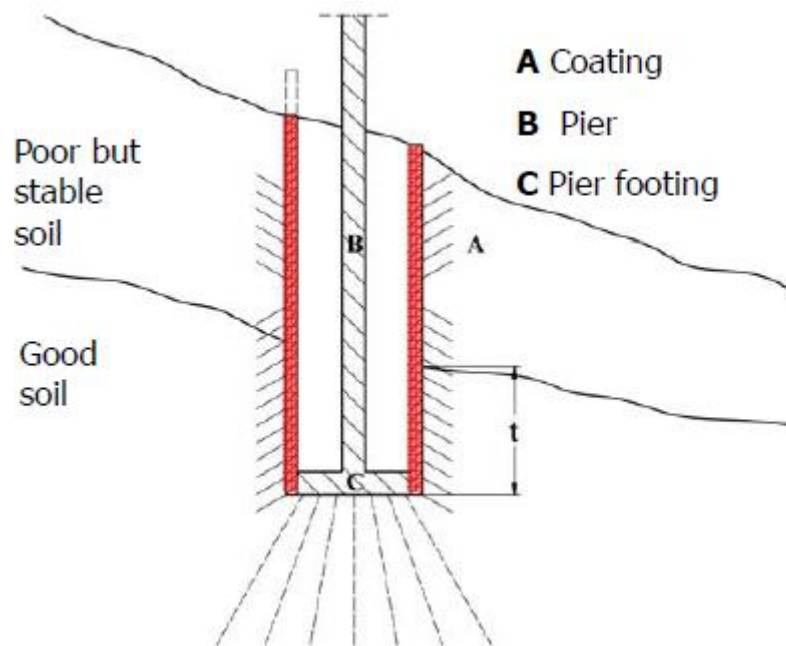
On the shaft side, typically shear and compressive-bending verifications must be satisfied. Moreover, maximum stresses on the ground  $q_1$  must be compatible with terrain compressive strength at depth  $l$ .

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.

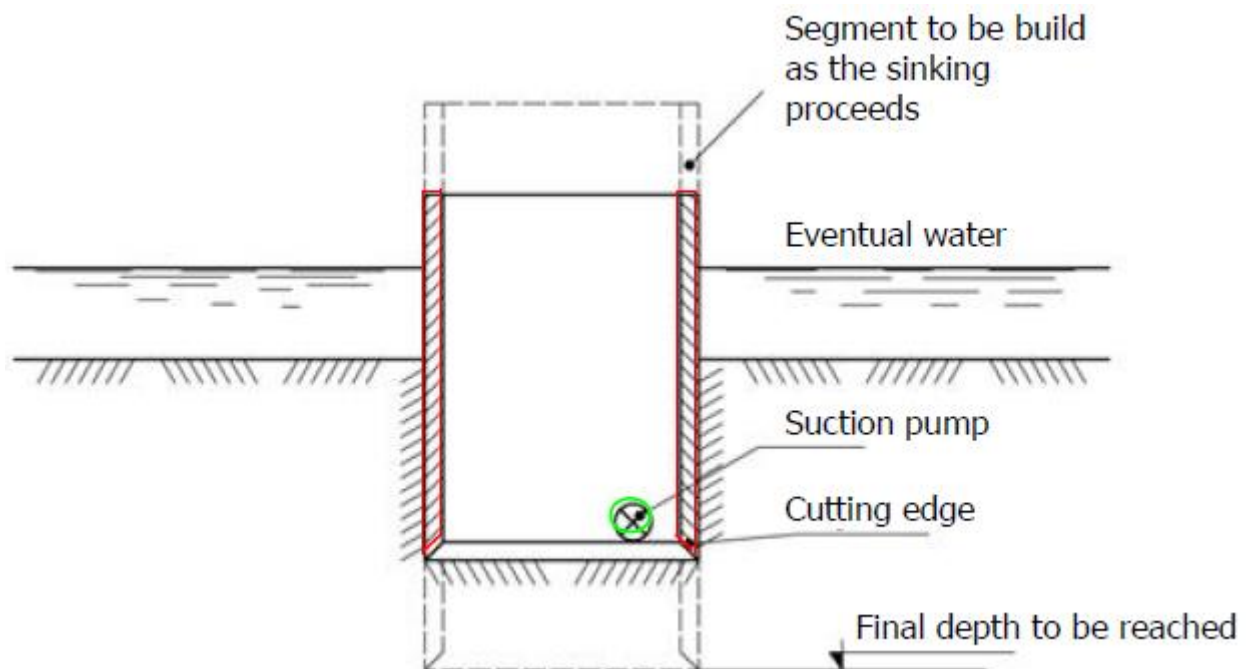
## CAISSON FOUNDATIONS

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.



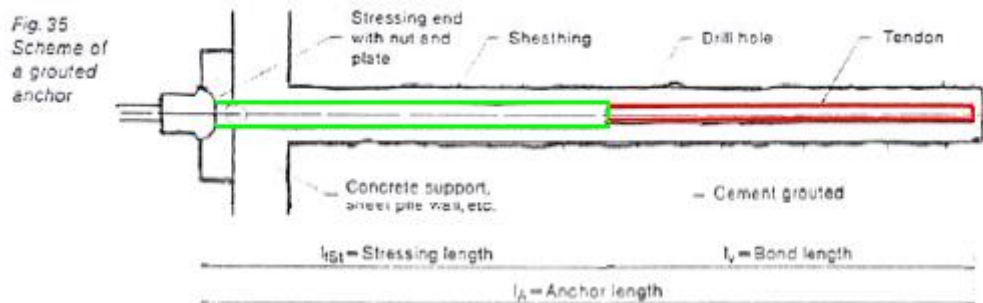
An important problem is that, in presence of water around the foundation, sometimes hydrostatic head may be too high and the ground too porous, so that bottom pumps can not keep the excavation dry or even lead to the failure of the ground due to the pressure exerted by the water – it is a terrible failure 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.

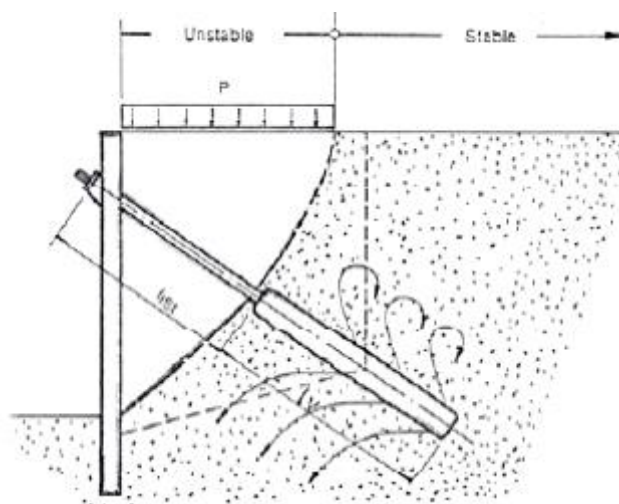


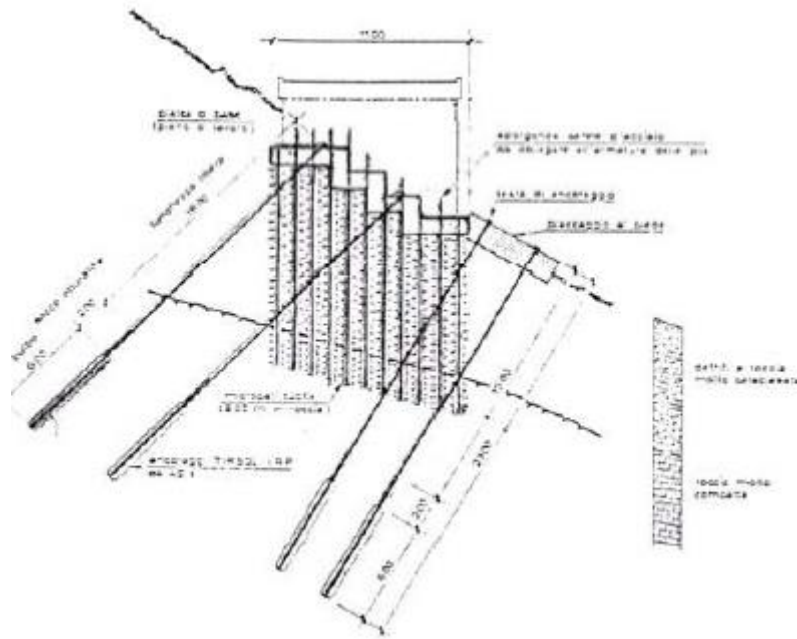
We follow a certain execution procedure for their realization.

- 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.





- 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  $\gamma_I$ , corresponding to a variation of the reference period  $T_0$  and, by consequence, a variation of the seismic action.

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

## 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.

$$V_{s,30} > 800 \text{ m/s}$$

- 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.

$$V_{s,30} = 360 \div 800 \text{ m/s}$$

$$N_{SPT} > 50$$

$$c_U > 250 \text{ kPa}$$

## 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.

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

$$T_B \leq T < T_C \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5$$

$$T_C \leq T < T_D \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \left( \frac{T_C}{T} \right)$$

$$T_D \leq T \quad S_e(T) = a_g \cdot S \cdot \eta \cdot 2,5 \cdot \left( \frac{T_C T_D}{T^2} \right)$$

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.

$$\begin{aligned}
 0 \leq T < T_B & \quad S_{ve}(T) = 0,9 a_g \cdot S \cdot \left( 1 + \frac{T}{T_B} \cdot (\eta \cdot 3,0 - 1) \right) \\
 T_B \leq T < T_C & \quad S_{ve}(T) = 0,9 a_g \cdot S \cdot \eta \cdot 3,0 \\
 T_C \leq T < T_D & \quad S_{ve}(T) = 0,9 a_g \cdot S \cdot \eta \cdot 3,0 \left( \frac{T_C}{T} \right) \\
 T_D \leq T & \quad S_{ve}(T) = 0,9 a_g \cdot S \cdot \eta \cdot 3,0 \cdot \left( \frac{T_C T_D}{T^2} \right)
 \end{aligned}$$

Ground category	S	T <sub>B</sub>	T <sub>C</sub>	T <sub>D</sub>
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

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

Ground category	T <sub>E</sub>	T <sub>F</sub>
A	4,5	10,0
B	5,0	10,0
C,D,E	6,0	10,0

$$T < T_E$$

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

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

$d_g$  = 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  $500 \text{ m} \times 500 \text{ m}$ . indeed, if the velocity of propagation is  $300 \text{ m/s}$ , 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

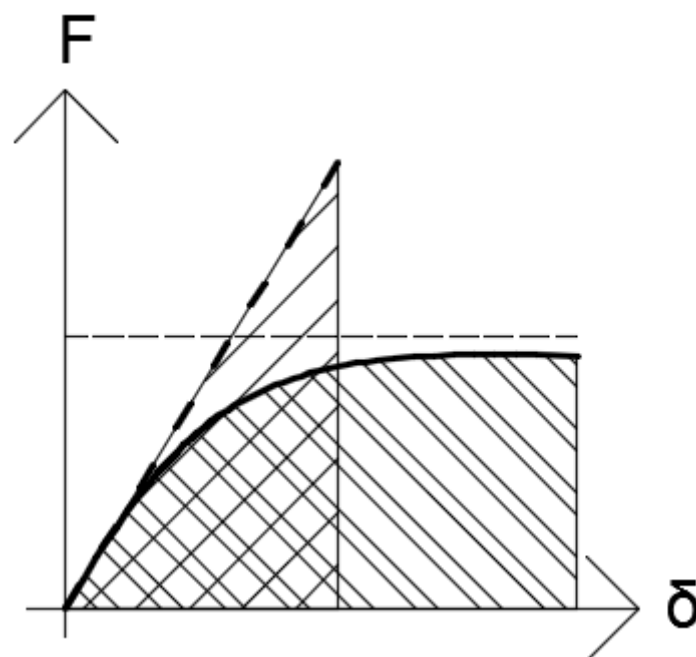
The direct application of the spectrum of elastic response in the computation of internal actions in the structures brings huge forces, since we are asking a structure to assume elastic behaviour during the seismic action. Of course, this kind of request is very expensive or even impossible to realize.

For this reason, we introduce the design spectrum for U.L.S., 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 the 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  $60 \div 70$  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.

$$\bar{A} > 0,9\overline{A_{ref}}, \quad 0,2T_1 \leq T \leq 2T_1$$

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 spatial variability of displacements.

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  $\gamma_I$  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.

$$\gamma_I E + G_k + P_k + \psi_{0,\Delta T} \Delta T, \quad \psi_{0,\Delta T} = 0,4$$

## 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  $\alpha_s$ , 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  $\lambda$ , to be applied to the structural factor.

$$\lambda = \begin{cases} 1,0 & \text{if } \alpha_s \geq 3 \\ \sqrt{\frac{\alpha_s}{3}} & \text{if } 1,0 \leq \alpha_s < 3 \end{cases}$$

- 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  $15 \div 20^\circ$ .
- Curved bridges.

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

$$q = 1$$

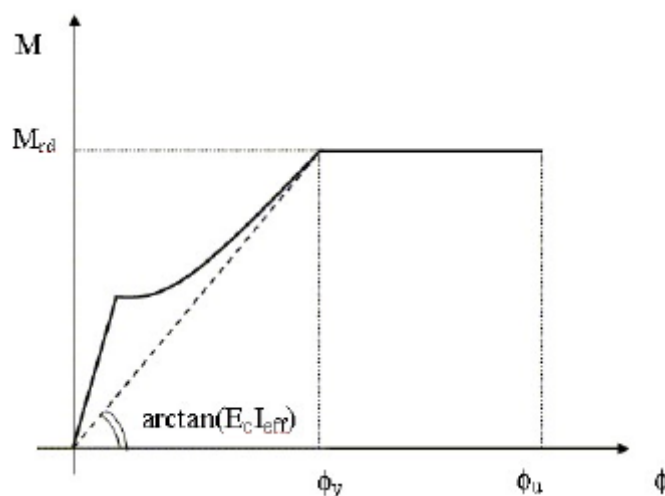
The reason is that these bridges do not have the condition of formation of plastic hinges and energy dissipation.

## MODELLING FOR LINEAR ANALYSIS

How can we model the bridge for the linear analysis?

As regards rigidity modelling, 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 – represented 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.

$$e_{max} < 0,05l_{bridge}$$

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.

$$M = \begin{cases} M_{deck} + \sum M_{i,upper\ half} \\ M_{deck} + M_{i,upper\ half} \end{cases}$$

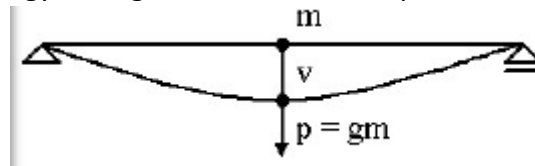
$S_d$  is the value of response spectrum in correspondence of the reference period  $T_1$ .

$$T_1 = 2\pi \sqrt{\frac{M}{K}}$$

$K$  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.

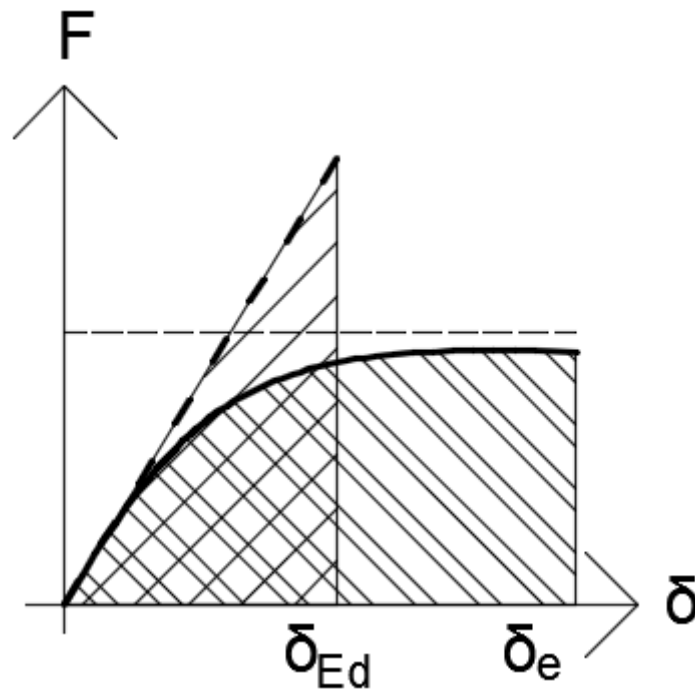
$$E_k = \frac{1}{2} m \dot{v}^2(t)$$

Assuming sinusoidal oscillation, the kinetic energy will be

$$E_k = \frac{1}{2} m v_0^2(t) \omega^2 \cos^2(\omega t)$$

The potential energy depends on the position, i.e. the displacement.

$$E_k = \frac{1}{2} p v(t) = \frac{1}{2} p v_0 \sin(\omega t), \quad p = mg$$



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

- Nonlinear dynamic analysis

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.

$$\sum (E_{piers,bottom,nonlin} + E_{abut,nonlin}) > 0,8 \sum (E_{piers,bottom,lin} + E_{abut,lin})$$

- Nonlinear static analysis (Push-over)

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.

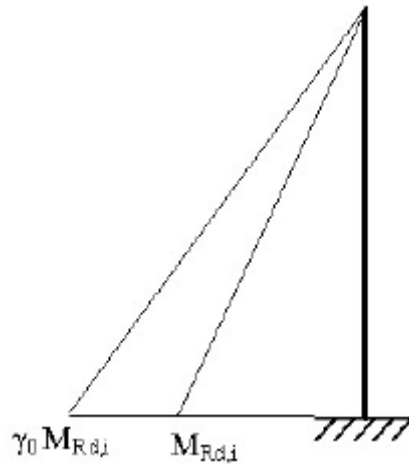
For instance, we assign a horizontal force at the top of the pier until a certain displacement at the top.

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



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 confinement of concrete by reinforcement.

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.

$$\omega_{wd,r} = \begin{cases} 0,33 \frac{A_c}{A_{cc}} \eta_k - 0,07 \geq \begin{cases} 0,18 & \text{ductile behaviour} \\ 0,12 & \text{limited ductile behaviour} \end{cases} \\ \frac{A_{sw}}{sb} \frac{f_{yd}}{f_{cd}}, & s \leq \begin{cases} 6\phi_{long} \\ \frac{1}{5} \times (\text{minimum confinement dimension}) \end{cases} \end{cases}$$

$A_c$  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

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

$Q$  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.

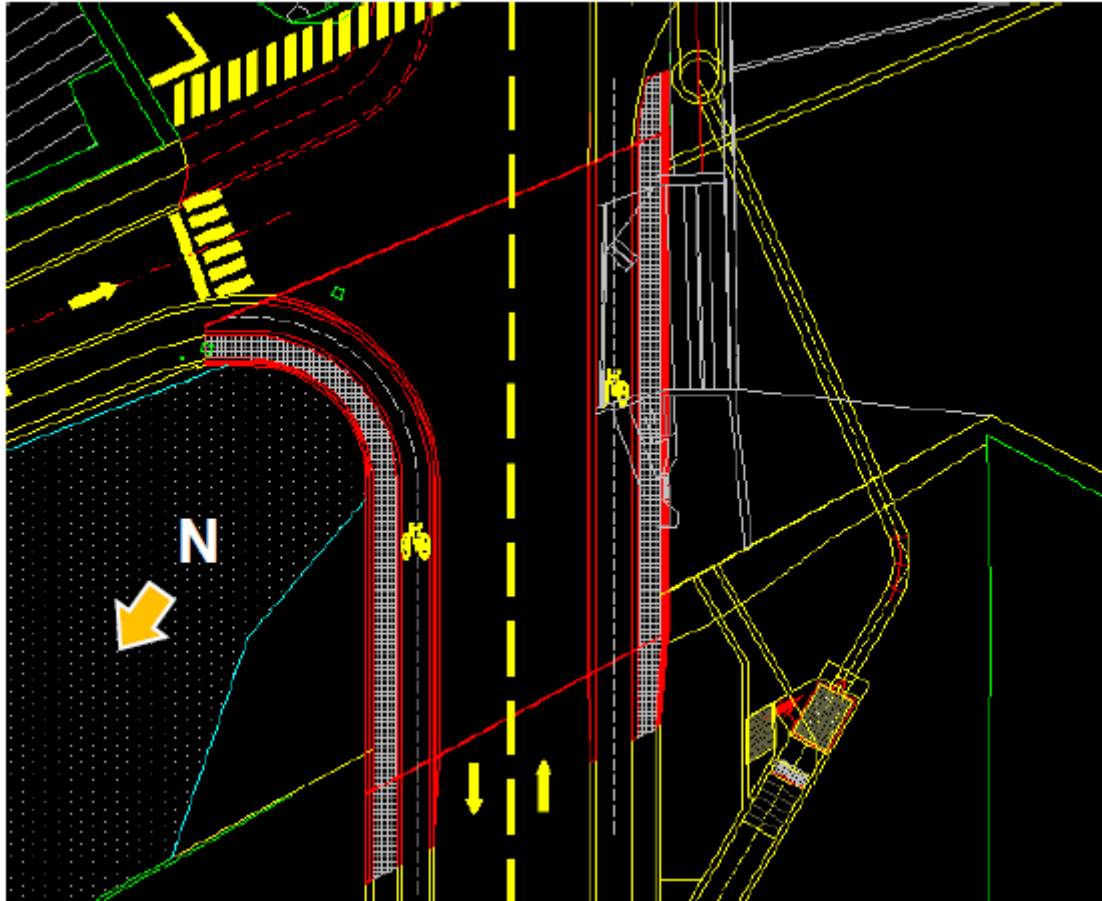
The seismic forces and friction forces of bearings should be protected by a capacity design, with a factor 1,3.

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

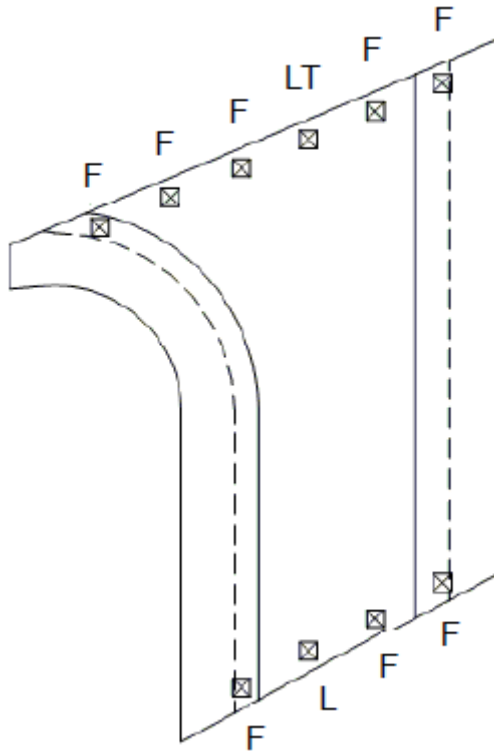
### GEOMETRY

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 another 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

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

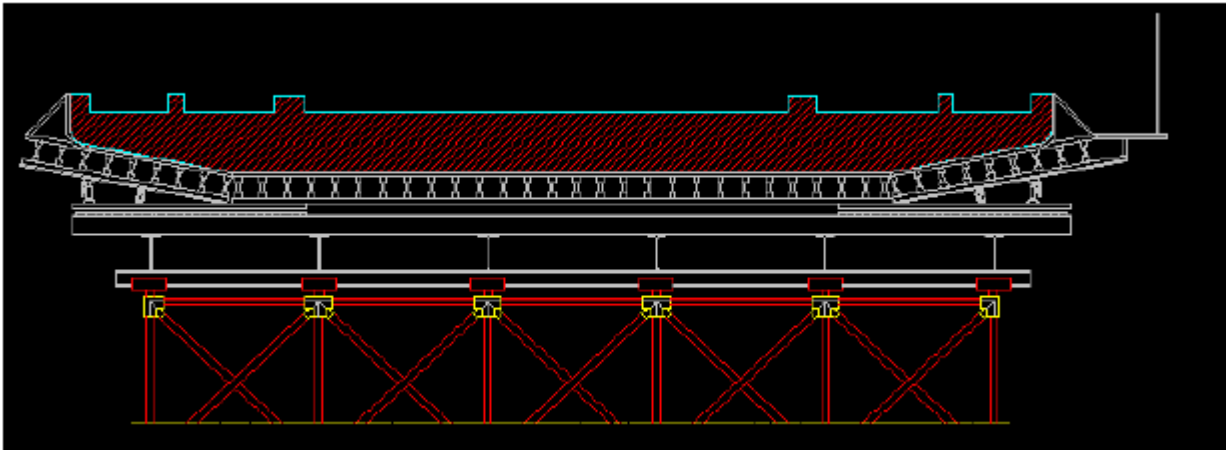
- Prestressing steel with 0,6'' strands – standard strands for prestressing. Their area is 1500 mm<sup>2</sup>, which is not the actual area but a given number, as a strand is not full section but has sparrels.

$$f_{pt,k} = 1860 \text{ MPa}$$

$$f_{pt1,k} = 1670 \text{ MPa}$$

$$E_s = 210000 \text{ MPa}$$

- 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



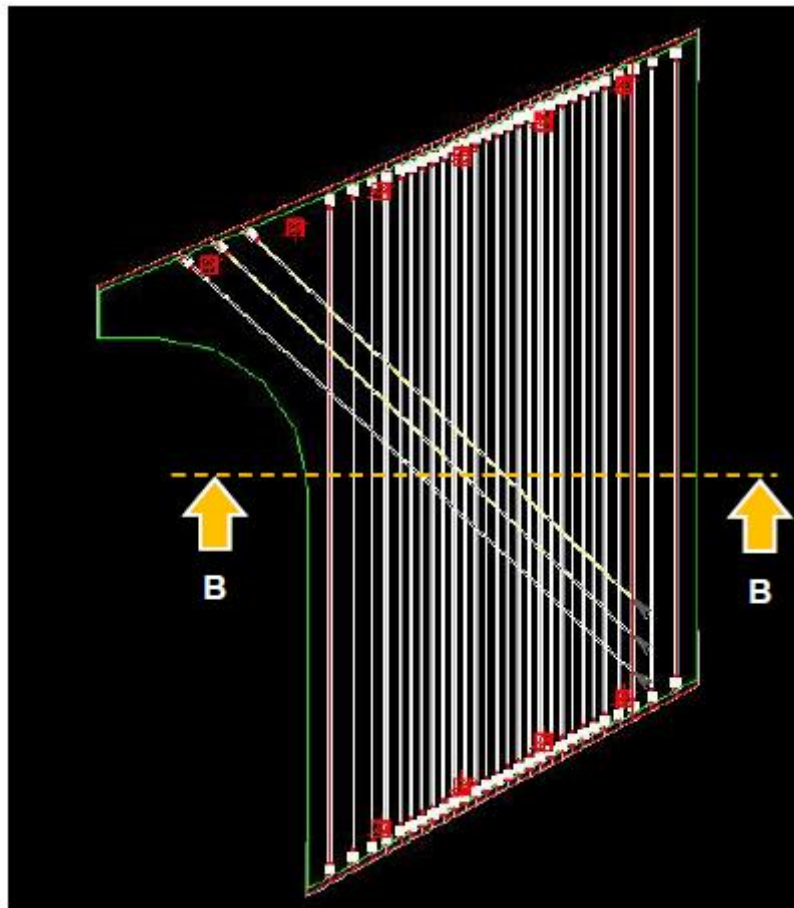
4. Once concrete has been hardened, they introduced prestressing in the deck slab, which creates a negative bending moment that lifted up the deck from the scaffolding and weight was closed on the bearings. At this point, they could dismantle the scaffolding. After the introduction of prestressing, they 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 placed 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.



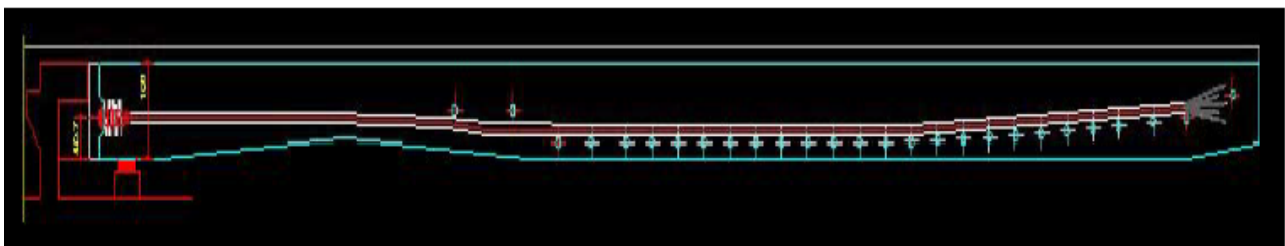
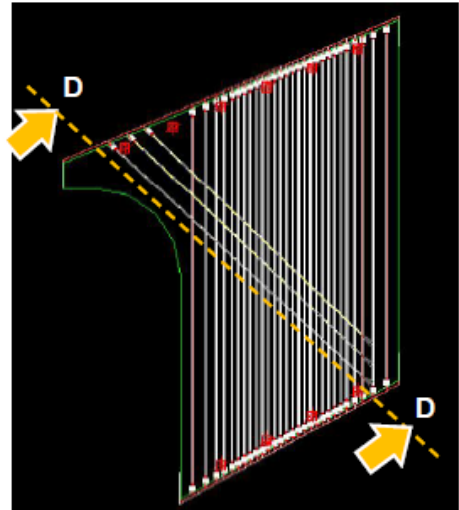
Looking at the cross section B-B, in correspondence of the mid-span in the direction orthogonal to the traffic, the tendons are placed at the bottom because positive bending moment is acting due to traffic loads and self-weight.

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







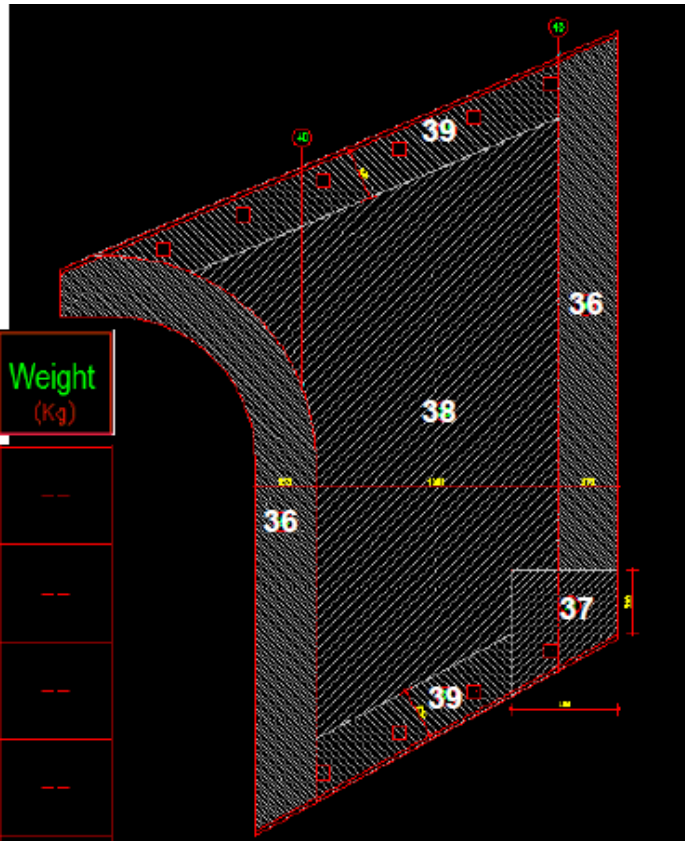
## ORDINARY REINFORCEMENT

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 of 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	$\phi$ (mm)	Shape	Length (cm)	N°	Weight (Kg)
36	14 40x40	 17 17 94=46	128 + 80	400 + 510	--
37	20 40x40	 27 27 94=46	148 + 100	138	--
38	14 40x60	 17 17 94	128	1140	--
39	16 40x40	 20 20 94	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-exhausting 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		80 + 1200	31	---
10	20 20		1200x2 + 607+777	82	---
12	20 20		632 + 1200	57	---
23a	20 20		367 + 1200	23	---
42	26 20		1200 + 810	5	---
42a	26 20		1200 + 917	5	---

Pos	φ (mm)	Shape	Length (cm)	N°	Weight (Kg)
44	20		144	111 x 50	---
45	26		164	1+1 x 50	---
46	20 20		144	5+5 x 50	---
47	20 20		180	(1+1) x 2	---
47a	20 20		185	1+1 x 8	---
48	20 20		800x2 + 700x2	6 x 2	---

DEAD ANCHORAGE DETAIL










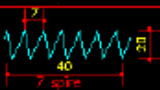
In this anchorage, each strand opens – candy-shaped strands – to give a better anchorage and the region presents small square stirrups in order to simulate the effect of the spiral. At the end of the duct, plastic foam is applied in order to avoid concrete going inside it during the casting.

Dead  
anchorage  
detail



# Dead anchorage detail

## Reinforcement description

Pos	$\phi$ (mm)	Shape	Length (cm)	N°	Weight (Kg)	Pos	$\phi$ (mm)	Shape	Length (cm)	N°	Weight (Kg)
49	20 / 20		360	1+1 x 3	---	53	20 / 20		120	3+3 x 3	---
50	20 / 20		400	1+1 x 3	---	54	20		144	1+1 x 3	---
51	20		100	2+2 x 3	---	55	26		164	1+1 x 3	---
52	10 / 20		90	3	---	56	20 / 20		144	5+5 x 3	---
52a	10 / 20		110	3	---	58	14		440	3	---

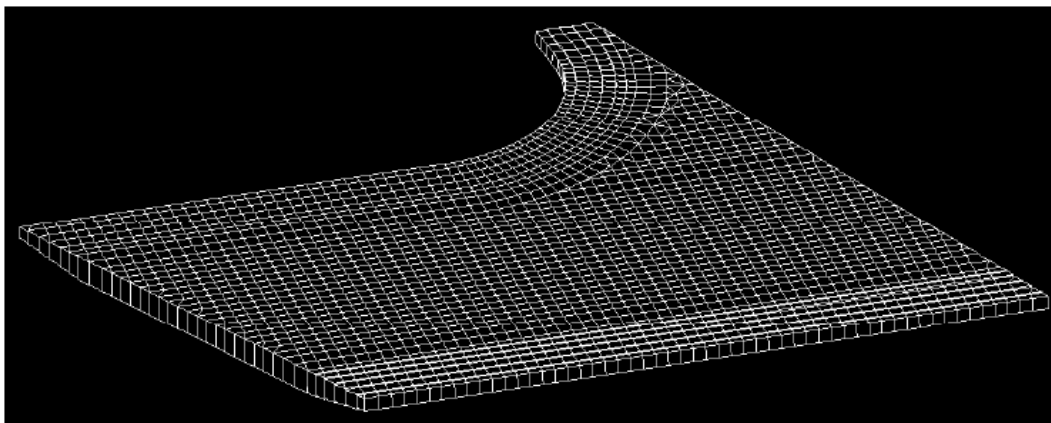
## Mesh

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.

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

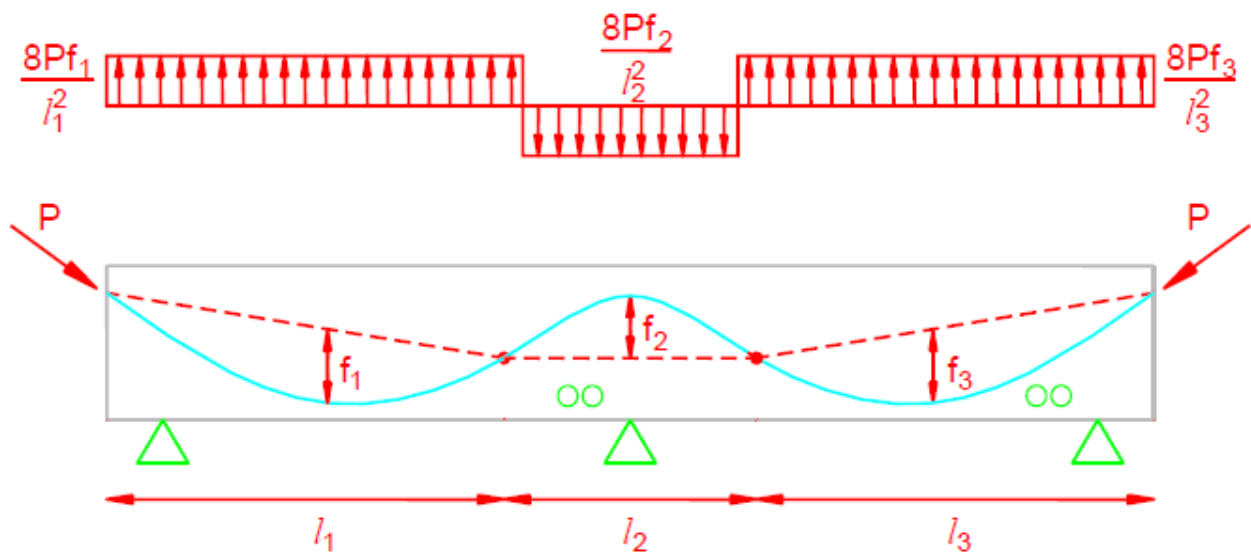
Assuming a parabolic path of the tendon, we get a closed-form solution of this force.

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

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.

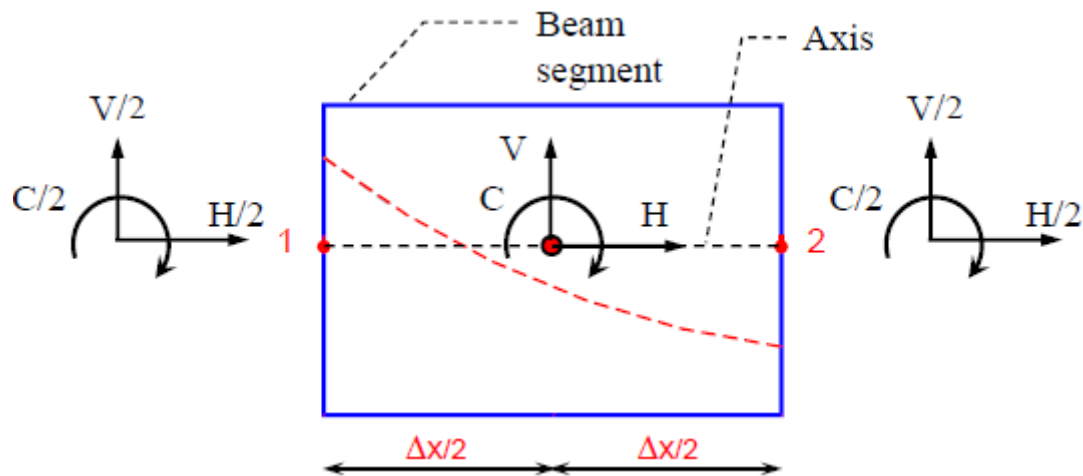
- All tendons follow a parabolic path, which is not the case of bridges.
- The value of the prestressing force  $P$  is constant along each tendon but it is not, due to friction.

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  $\Delta x$  – 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.

$$F_{2,x} = \frac{H}{2}$$

$$F_{2,y} = 0$$

$$F_{2,z} = \frac{V}{2}$$

$$M_{2,x} = 0$$

$$M_{2,y} = \frac{C}{2}$$

$$M_{2,z} = 0$$

$$F_{3,x} = \frac{H}{2}$$

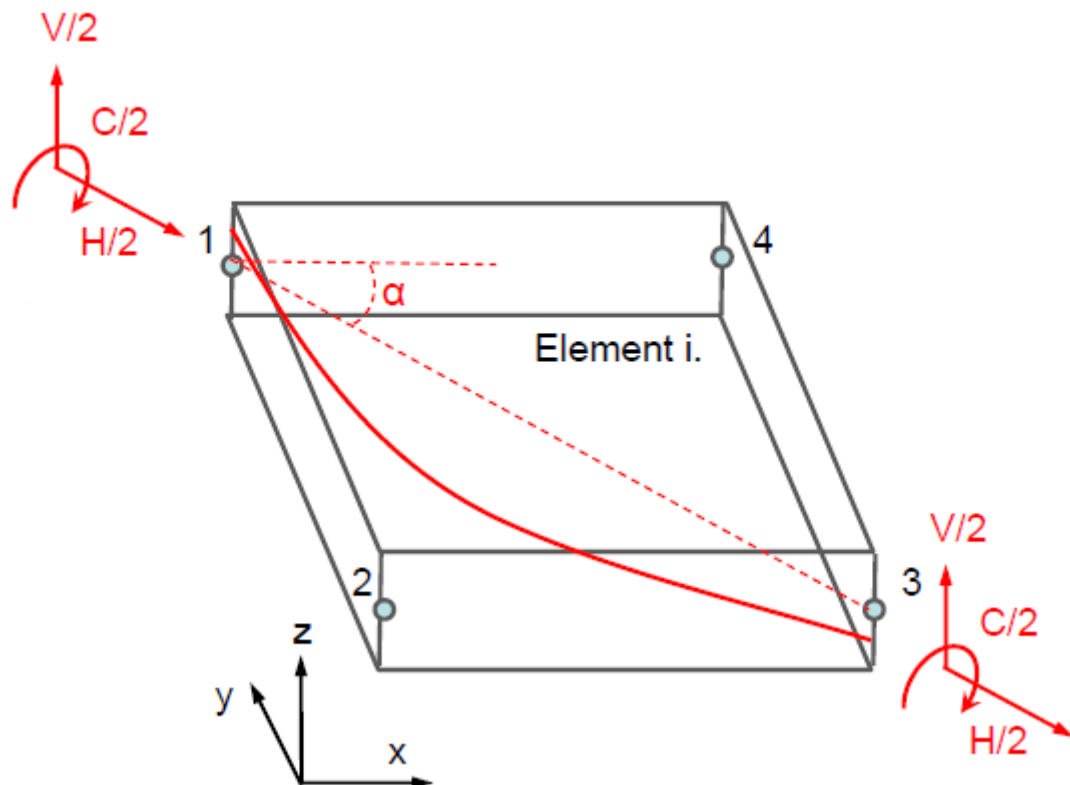
$$F_{3,y} = 0$$

$$F_{3,z} = \frac{V}{2}$$

$$M_{3,x} = 0$$

$$M_{3,y} = \frac{C}{2}$$

$$M_{3,z} = 0$$



- 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  $\alpha$  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.

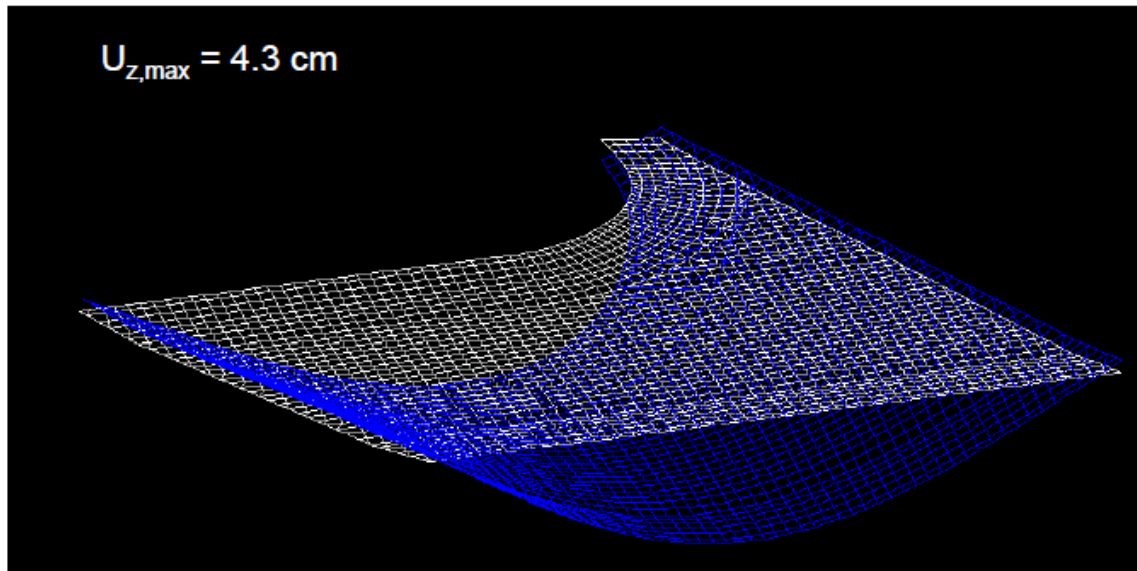
Finally, these components are transferred to nodes and, to do it, we can follow two ways.

- 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.

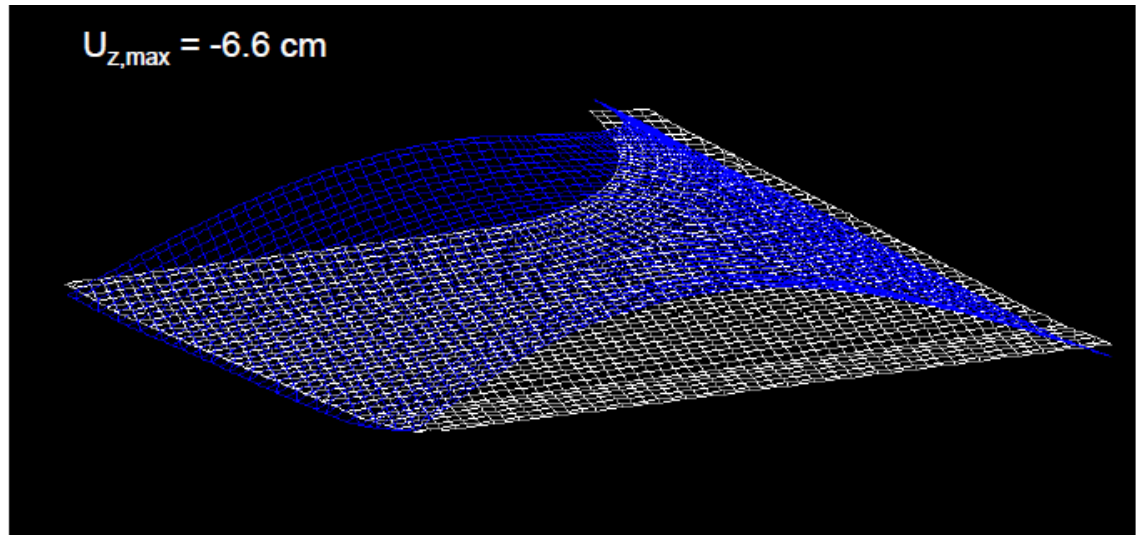
## ANALYSIS RESULTS

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

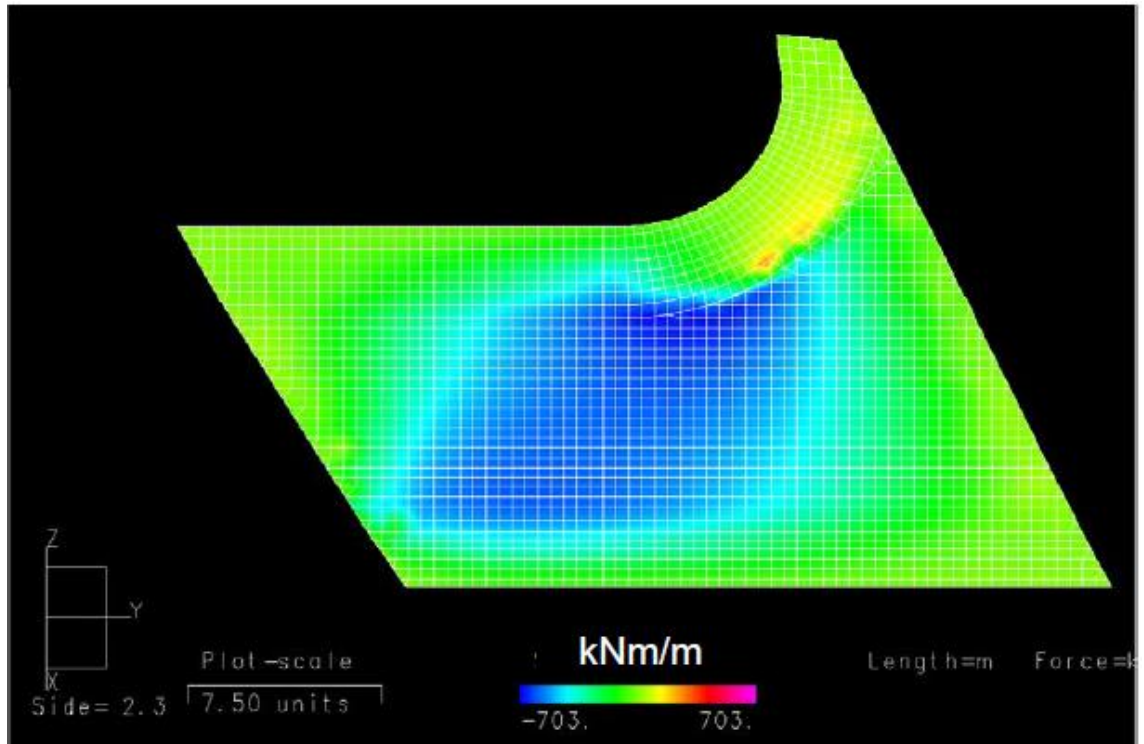
- The deformed shape of the deck due to dead weight shows a maximum vertical displacement of 4 cm with respect to 18 m of longitudinal dimension, which means a vertical displacement of 2 cm every 10 m.



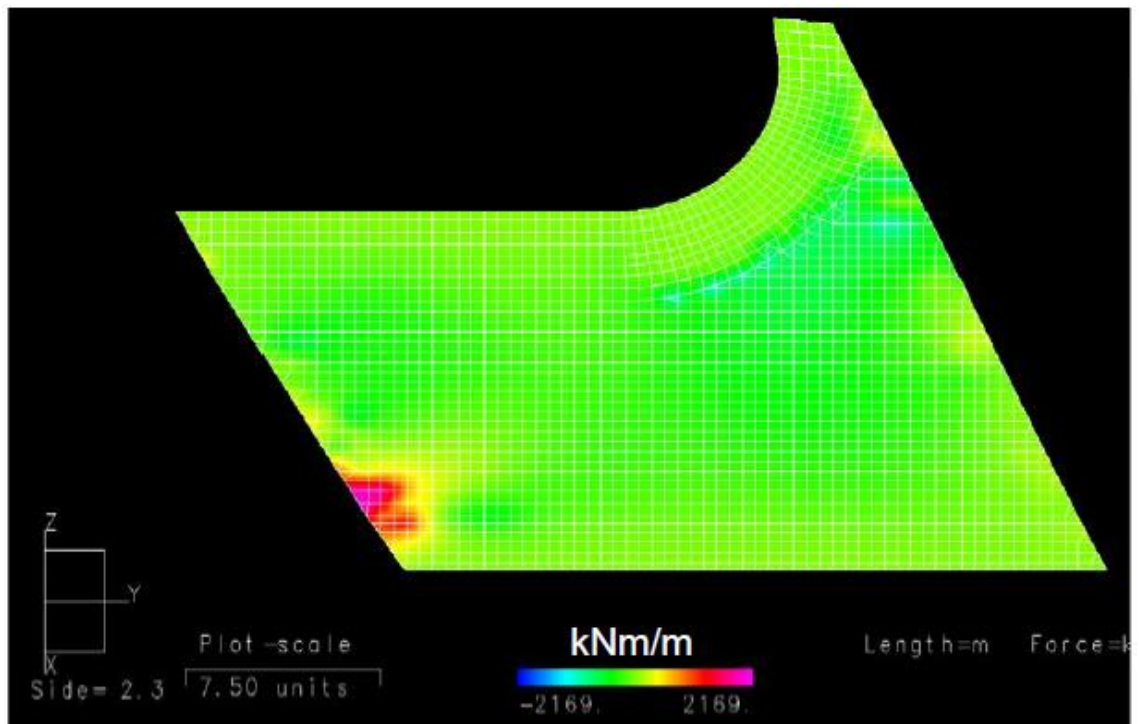
- The deformed shape of the deck due to prestressing is in the opposite direction with a maximum vertical displacement of 6,5 cm. indeed, generally prestressing – as an action – is bigger than self-weight, also 1,5 times bigger.



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

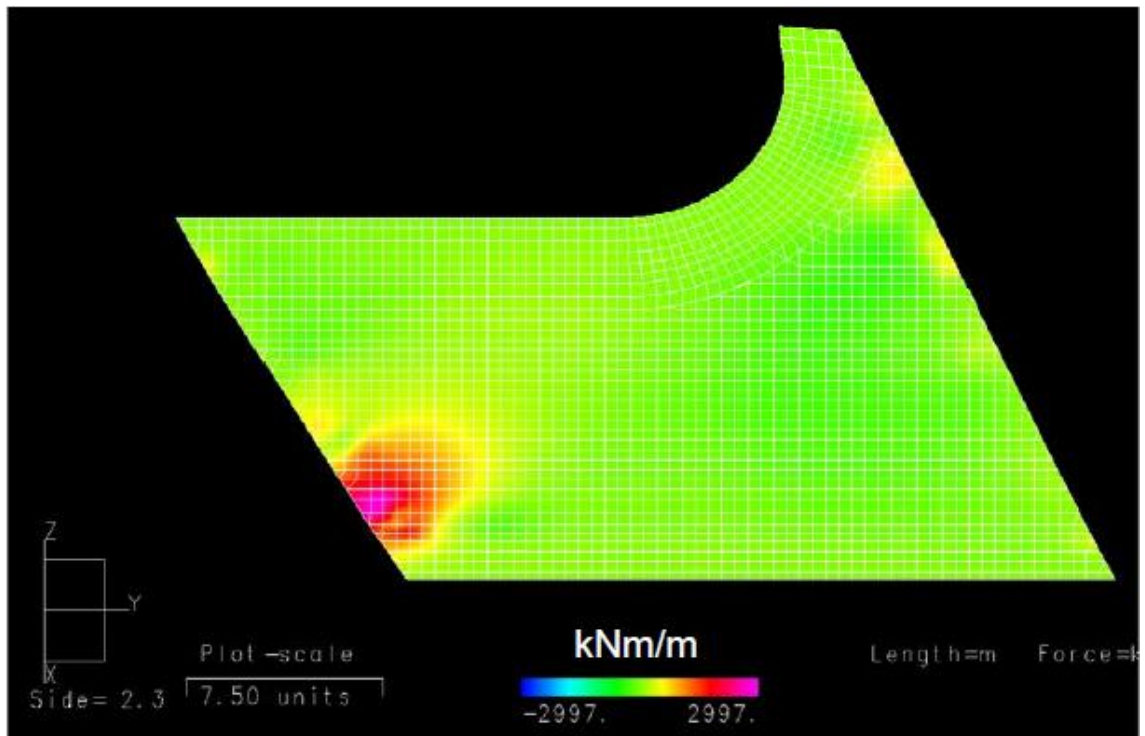


- Bending moment  $m_x$  in  $x$  direction due to prestressing, which is small as most of prestressing is aligned in the longitudinal direction and does not generate stresses in the transverse direction. There is just a concentration next to the corner due to the anchorage of skew tendons.

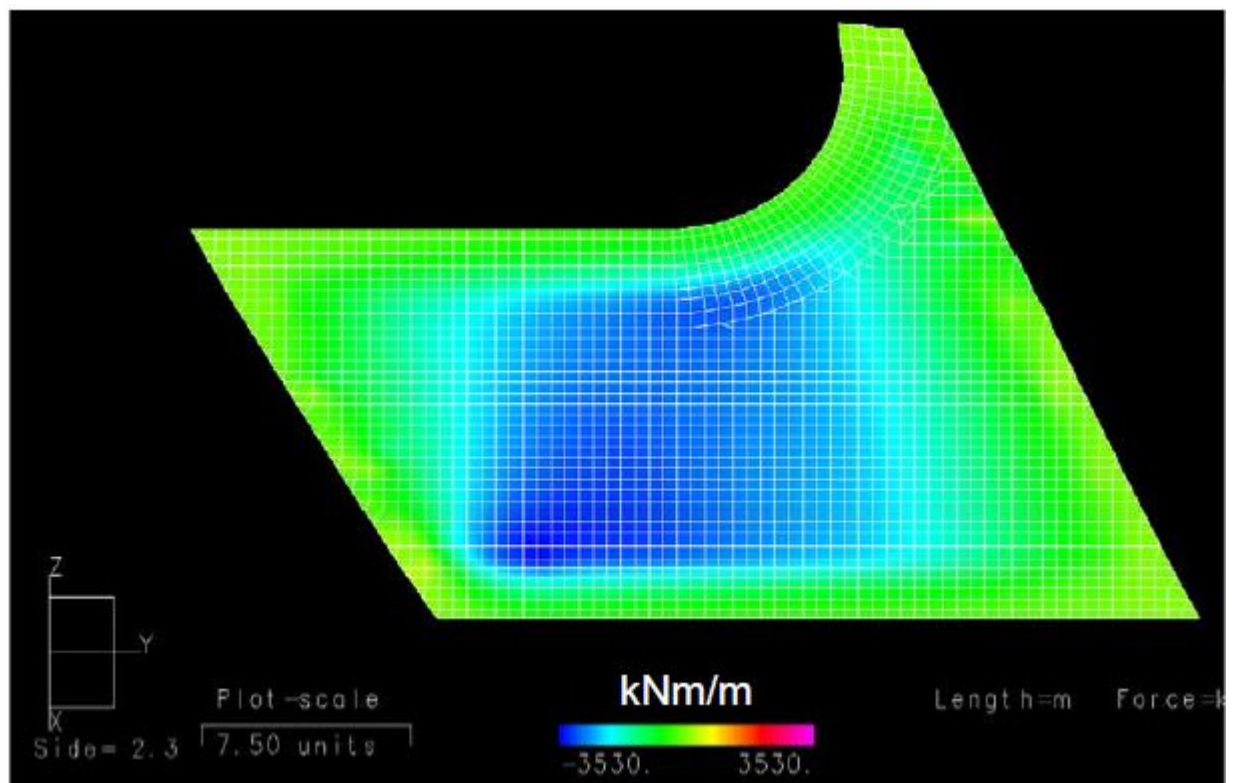


- Bending moment  $m_y$  in  $y$  direction due to prestressing, which presents a big negative value in order to counterbalance the positive bending moment, with a value of 3250 kNm/m with respect to 2000 kNm/m.

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.

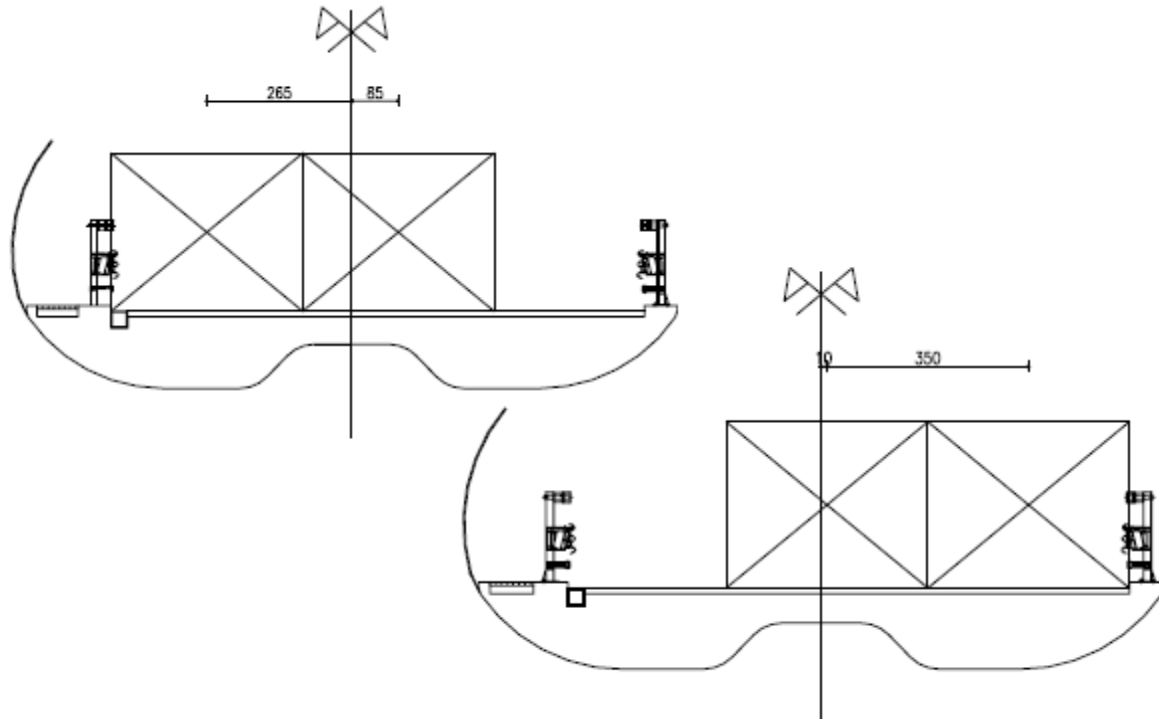


- Principal bending moment  $m_2$  at the end of service life is negative as principal stress  $\sigma_2$  is negative due to prestressing. It presents a decrease of 15% of its value due to prestressing losses.



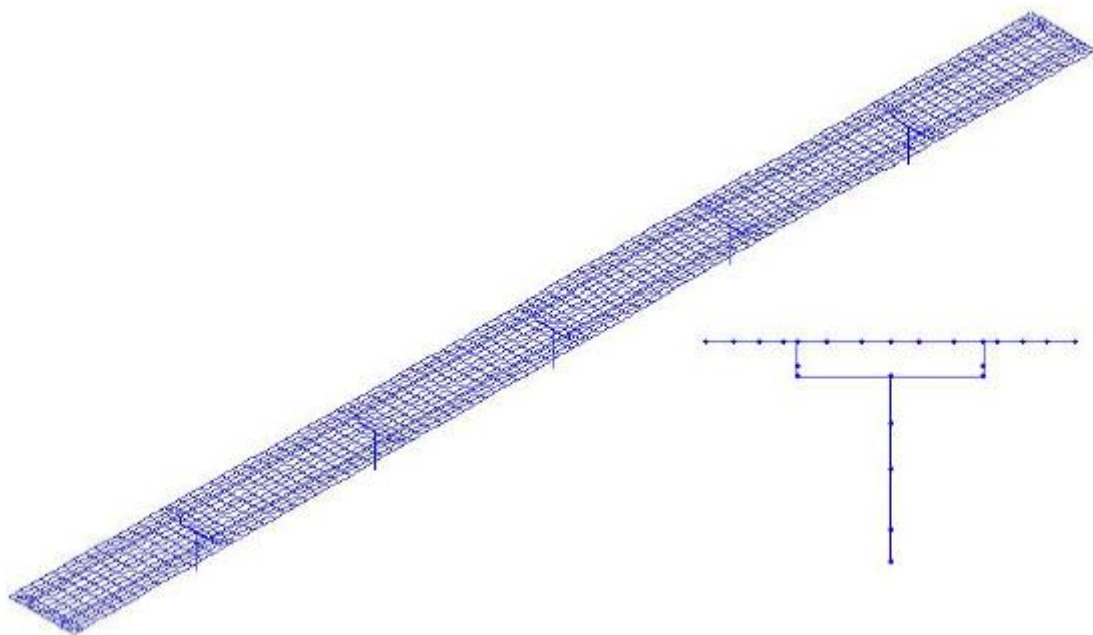
and, in this way, actual beams are loaded. Indeed, the software is able to run a moving load but it needs a real or virtual path, along which the movable load is moved.

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