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CONCEPTUAL DESIGN: FROM ABSTRACT REASONING TO CONSISTENT DETAILS

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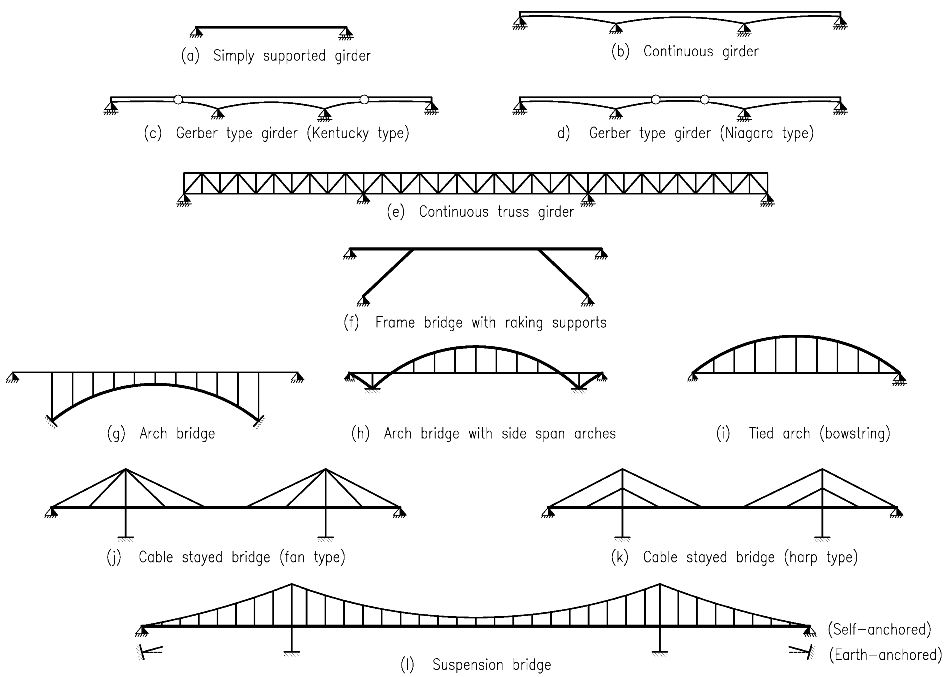
ABOUT ORIGINALITY

Structure is any assembly of materials which is intended to sustain loads (J. E. Gordon).

Civil Engineering structures are usually referred to typological schemes (beams, frames, trusses, arches, etc.) already geared, for shape and performance, to certain uses.



INTRODUCTORY CONSIDERATIONS

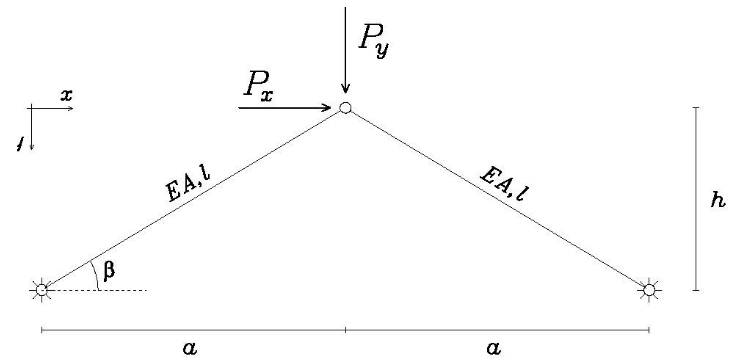
- The use of materials (steel, concrete aluminium, . . .) with well defined and guaranteed properties and the capacity of the numerical analyses have allowed to explore every possible implication of these classes of structures.
- It is therefore difficult to formulate structures with real elements of originality.
- Original structural proposals arise generally in architecture, where originality helps to enhance and make unique formal aspects.
- Original forms can be recognized in the design of industrial plants, where the need to adapt the structure to the configuration of the equipment and competitiveness linked at minimum cost (minimum weight of steel used), led to formulate really special solutions.

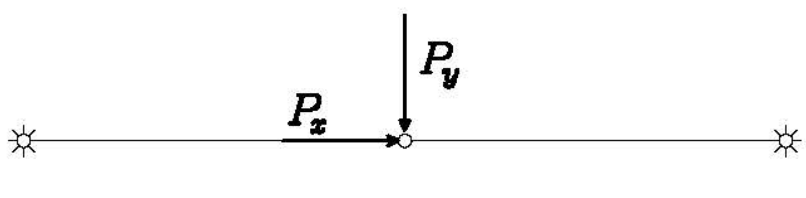
POSSIBILITIES DERIVED FROM IMPOSED CONSTRAINTS

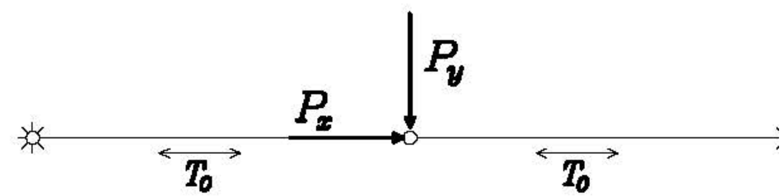
- New structural types may be formulated by working with imposed constraints.
- Spontaneous is the reference to prestressed concrete structures. In such case, the constraint is the usual prestressing action, which modify the original regime, through the antagonistic action of a cable system which, section by section, counterbalance self-weight and applied loads effects.
- But constraints may be applied to steel elements too, even if with different criteria, both in the role exercised by constraint, both in form with which it is applied. In this case wires, strands or ropes are arranged in the plane or in the space so that their action contribute in defining the shape of the structure and, conditioning the members to work mainly in tension or compression, allows better performances and a better use of materials used.

CONSTRAINTS AND CABLES

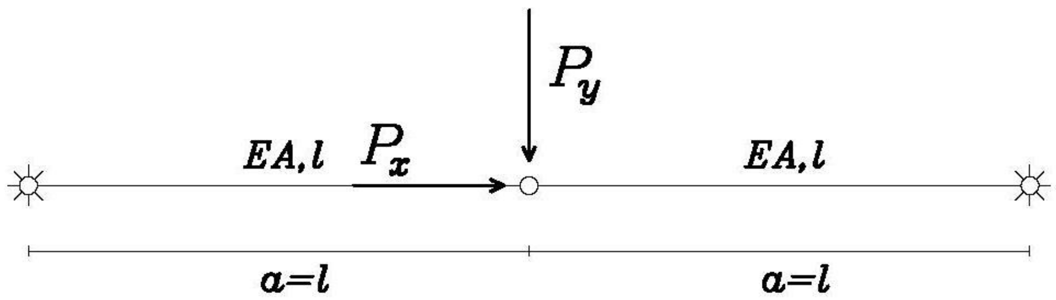
FROM VON MISES TRUSS TO . . . CHAINS





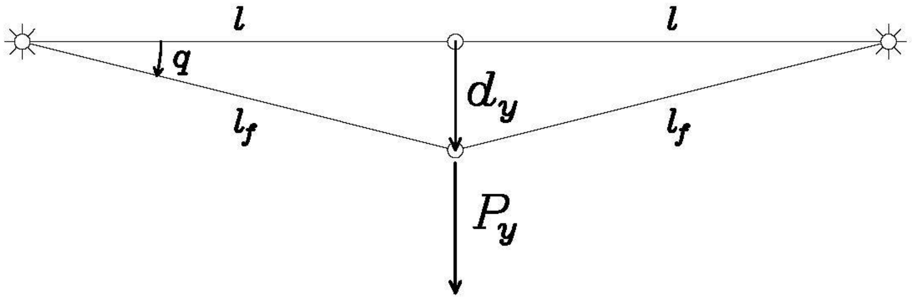


TWO BARS WITH THREE ALIGNED HINGES



$$k_x = 2\frac{EA}{l^3}a^2, \qquad k_y = 0 \qquad \mathbf{K} = \begin{bmatrix} k_x & 0 \\ 0 & k_y \end{bmatrix} = \begin{bmatrix} 2\frac{EA}{l^3}a^2 & 0 \\ 0 & 0 \end{bmatrix}$$

TWO BARS WITH THREE ALIGNED HINGES



$$e = l_f - l$$

$$l_f = \sqrt{d_y^2 + l^2} = l\sqrt{1 + \frac{d_y^2}{l^2}}$$

Mc Laurin Series . . .

$$l_f = l\left[1 + \frac{1}{2}\left(\frac{d_y}{l}\right)^2 - \frac{1}{8}\left(\frac{d_y}{l}\right)^4\right]$$

1th term . . . $e = l - l = 0$

2th term . . . $l_f = l\left[1 + \frac{1}{2}\left(\frac{d_y}{l}\right)^2\right] \Rightarrow e = l_f - l = l + \frac{1}{2}l\left(\frac{d_y}{l}\right)^2 - l = \frac{1}{2}\left(\frac{d_y^2}{l}\right)$

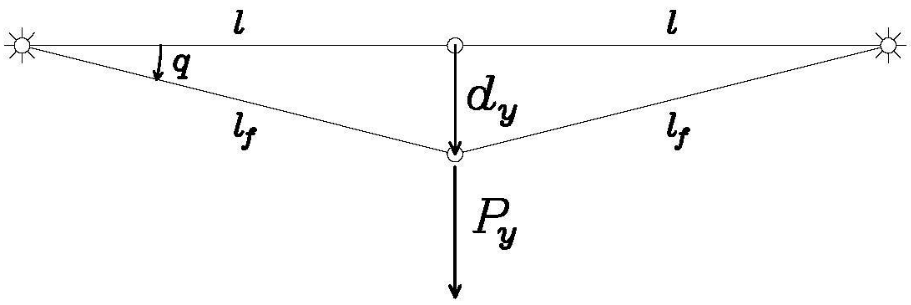
TWO BARS WITH THREE ALIGNED HINGES

$$e = l_f - l = l + \frac{1}{2} l \left(\frac{d_y}{l}\right)^2 - l = \frac{1}{2}\left(\frac{d_y^2}{l}\right)$$

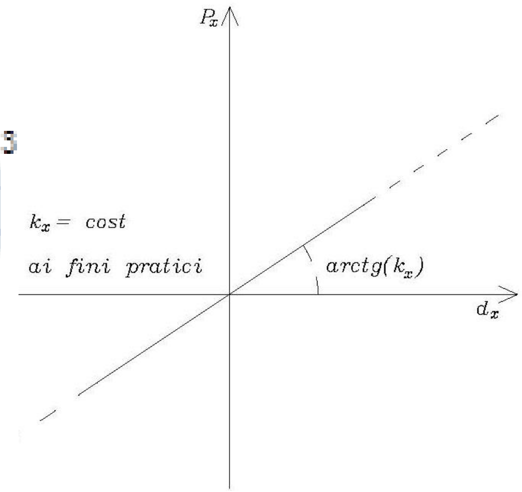
$$\varepsilon = \frac{e}{l} = \frac{l_f - l}{l} = \frac{1}{2}\left(\frac{d_y}{l}\right)^2$$

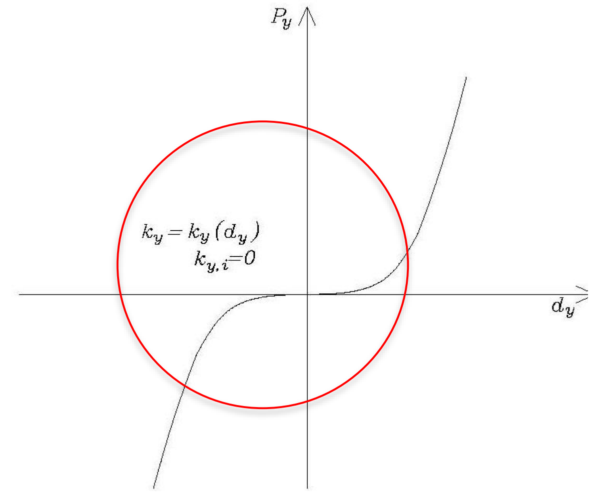
$$T = EA\varepsilon = \frac{1}{2} EA \left(\frac{d_y}{l}\right)^2$$

$$P_y = 2T \sin q = 2T \frac{d_y}{l} = EA \left(\frac{d_y}{l}\right)^3$$



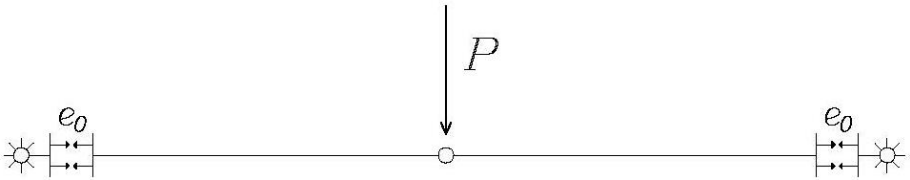
Vertical Stiffness as function of the vertical displacement

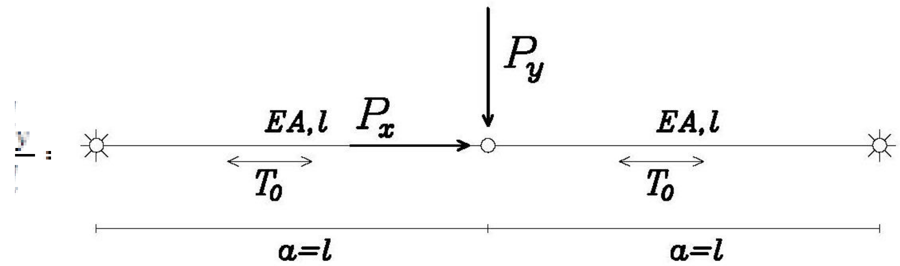




Horizontal stiffness

TWO BARS WITH THREE ALIGNED HINGES AND PRETENSION



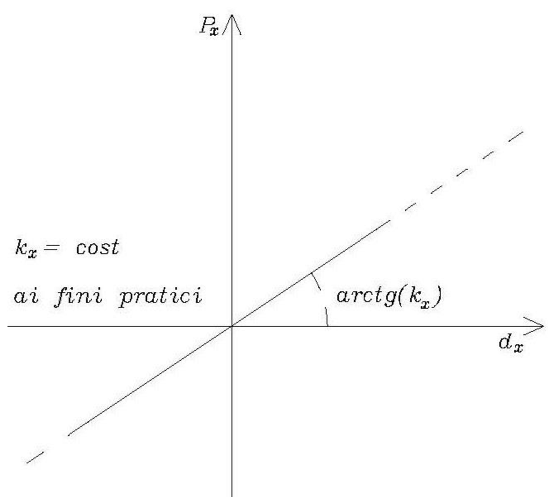


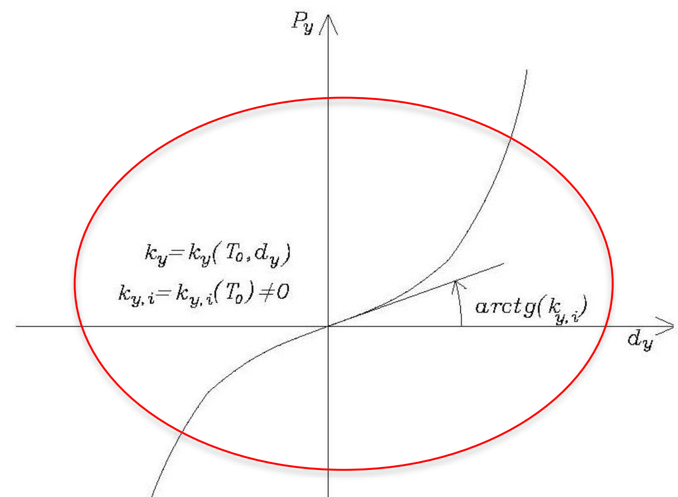
$$e_0 = T_0 \frac{l}{EA}$$

$$T = EA\frac{e}{l} = EA\frac{1}{l}\left(T_0 \frac{l}{EA} + \frac{1}{2}\frac{d_y^2}{l}\right) = T_0 + \frac{1}{2}EA\left(\frac{d_y}{l}\right)^2$$

$$P_y = 2T\sin q = 2T\frac{d_y}{l} = 2T_0\left(\frac{d_y}{l}\right) + EA\left(\frac{d_y}{l}\right)^3$$

$$k_{y,i} = 2\frac{T_0}{l}$$





2 BARS WITH 3 ALIGNED HINGES AND PRETENSION – STABILITY CHECK

STABILITY CHECK

External work

$$L = \frac{1}{2}\mathbf{d}^T\mathbf{f} = \frac{1}{2}\mathbf{d}^T\mathbf{K}\mathbf{d}$$

Stiffness Matrix

$$\mathbf{K} = \mathbf{K}_E + \mathbf{K}_G = 2\frac{EA}{l^3}\begin{bmatrix} \alpha^2 & 0 \\ 0 & 0 \end{bmatrix} + \begin{bmatrix} 0 & 0 \\ 0 & \frac{2T_0}{l} \end{bmatrix}$$

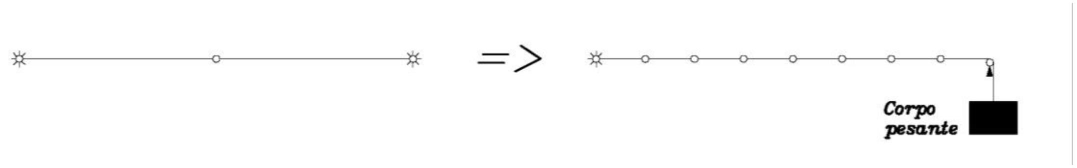
Ext. Work done by loads

$$L = \frac{1}{2}\begin{bmatrix} 0 & 1 \end{bmatrix}\begin{bmatrix} 2\frac{EA}{l^3}\alpha^2 & 0 \\ 0 & 0 \end{bmatrix}\begin{bmatrix} 0 \\ 1 \end{bmatrix} + \frac{1}{2}\begin{bmatrix} 0 & 1 \end{bmatrix}\begin{bmatrix} 0 & 0 \\ 0 & \frac{2T_0}{l} \end{bmatrix}\begin{bmatrix} 0 \\ 1 \end{bmatrix} = 0 + \frac{T_0}{l}$$

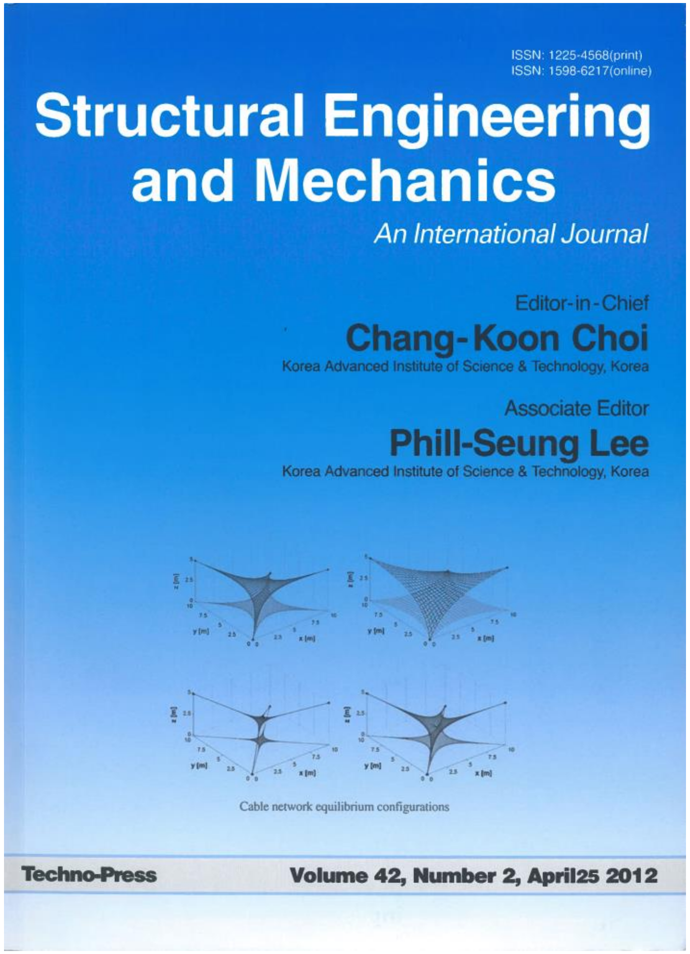
Stability Check:

The work is positive and the mechanism is stable if T_0 is positive and put the bars in tension.

By increasing the number of bars, we tend to the behaviour of a chain



CABLE NETS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Cable network equilibrium configurations
Techno-Press
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AN OUTLINE OF THE FORCE DENSITY METHOD (FDM) SCHEK, 1974)

We consider a generic net, with: n free nodes; n_f fixed nodes; $n_s = n + n_f$ total number of nodes connected by m cable elements.

Hypotheses

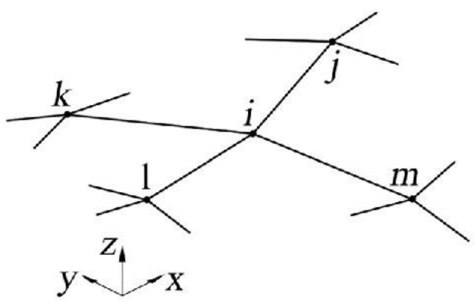
a) the net is made of straight cable elements, connected at the nodes

b) the net connectivity is known and its geometry is defined by the nodal coordinates

c) the cable elements are weightless

d) the net is subjected to concentrated forces, applied at the nodes.

With reference to the i^{th} node of Figure, the equilibrium equations in the x, y, z, directions are respectively



$$
\begin{aligned}
T_{ij}\frac{x_j - x_i}{L_{ij}} + T_{ik}\frac{x_k - x_i}{L_{ik}} + T_{il}\frac{x_l - x_i}{L_{il}} + T_{m}\frac{x_m - x_i}{L_{im}} + F_{xi} &= 0 \\
T_{ij}\frac{y_j - y_i}{L_{ij}} + T_{ik}\frac{y_k - y_i}{L_{ik}} + T_{il}\frac{y_l - y_i}{L_{il}} + T_{m}\frac{y_m - y_i}{L_{im}} + F_{yi} &= 0 \\
T_{ij}\frac{z_j - z_i}{L_{ij}} + T_{ik}\frac{z_k - z_i}{L_{ik}} + T_{il}\frac{z_l - z_i}{L_{il}} + T_{m}\frac{z_m - z_i}{L_{im}} + F_{zi} &= 0
\end{aligned}
\qquad (1)
$$

Where T_{ij} is the tensile force and L_{ij} is the length of the cable element between the nodes i and j.

AN OUTLINE OF THE FORCE DENSITY METHOD (FDM) SCHEK, 1974)

By introducing the following vectors and matrices, the previous Equation can be set into a matrix form:

- $\mathbf{x}_S$, $\mathbf{y}_S$, $\mathbf{z}_S$, $[n_s \times 1]$, coordinates of the nodes. By numbering the set of the fixed nodes after that of the free ones, the three vectors are partitioned into the following subvectors: $\mathbf{x}$, $\mathbf{y}$, $\mathbf{z}$, $[n \times 1]$, coordinates of the free nodes; $\mathbf{x}_f$, $\mathbf{y}_f$, $\mathbf{z}_f$, $[n \times 1]$, coordinates of the fixed nodes;
- $\mathbf{f}_x$, $\mathbf{f}_y$, $\mathbf{f}_z$, $[n \times 1]$, nodal forces;
- $\mathbf{l}$, $[m \times 1]$ length of the elements. $\mathbf{L} = diag(\mathbf{l})$;
- $\mathbf{t}$, $[m \times 1]$, tensile forces in the elements.
- connectivity matrix $\mathbf{C}_S$, having dimensions $[m \times n_s]$, whose terms are

$$c_S(e) = \begin{cases} +1 & \text{if} \quad i = 1 \\ -1 & \text{if} \quad i = 2 \\ 0 & \text{in other cases} \end{cases} \tag{2}$$

The difference between the couples of coordinates in the three directions *x*, *y*, *z* are

$$\mathbf{u} = \mathbf{C}_s\mathbf{x}_s, \qquad \mathbf{v} = \mathbf{C}_s\mathbf{y}_s, \qquad \mathbf{w} = \mathbf{C}_s\mathbf{z}_s \tag{3}$$

AN OUTLINE OF THE FORCE DENSITY METHOD (FDM) SCHEK, 1974)

The difference between the couples of coordinates in the three directions *x*, *y*, *z* are

$$\mathbf{u} = \mathbf{C}_s \mathbf{x}_s, \qquad \mathbf{v} = \mathbf{C}_s \mathbf{y}_s, \qquad \mathbf{w} = \mathbf{C}_s \mathbf{z}_s \tag{3}$$

In this equation, by partitioning the matrix $\mathbf{C}_s$ we can put in evidence separately the coordinates of the free nodes and those of the fixed nodes, as follows

$$\mathbf{u} = \mathbf{C}_s \mathbf{x}_s = \mathbf{C}\mathbf{x} + \mathbf{C}_f \mathbf{x}_f, \qquad \mathbf{v} = \mathbf{C}_s \mathbf{y}_s = \mathbf{C}\mathbf{y} + \mathbf{C}_f \mathbf{y}_f, \qquad \mathbf{w} = \mathbf{C}_s \mathbf{z}_s = \mathbf{C}\mathbf{z} + \mathbf{C}_f \mathbf{z}_f \tag{4}$$

By introducing the diagonal matrices $\mathbf{U} = diag(\mathbf{u})$, $\mathbf{V} = diag(\mathbf{v})$, $\mathbf{W} = diag(\mathbf{w})$, $\mathbf{L} = diag(\mathbf{l})$ the equilibrium equations are expressed by the system

$$\begin{cases} \mathbf{C}^T \mathbf{U} \mathbf{L}^{-1} \mathbf{t} = \mathbf{f}_x \\ \mathbf{C}^T \mathbf{V} \mathbf{L}^{-1} \mathbf{t} = \mathbf{f}_y \\ \mathbf{C}^T \mathbf{W} \mathbf{L}^{-1} \mathbf{t} = \mathbf{f}_z \end{cases} \tag{5}$$

AN OUTLINE OF THE FORCE DENSITY METHOD (FDM) SCHEK, 1974)

Now, if we introduce the concept of force density $q = T/L$, in matrix form we obtain

$$\mathbf{q} = \mathbf{L}^{-1}\mathbf{t} \tag{6}$$

Through this transformation, the equations of the system (5) become linear and uncoupled in the three cartesian directions

$$\mathbf{C}^T\mathbf{U}\mathbf{q} = \mathbf{f}_x, \qquad \mathbf{C}^T\mathbf{V}\mathbf{q} = \mathbf{f}_y, \qquad \mathbf{C}^T\mathbf{W}\mathbf{q} = \mathbf{f}_z \tag{7}$$

By introducing the diagonal matrix $\mathbf{Q} = diag(\mathbf{q})$, the following identities hold

$$\mathbf{U}\mathbf{q} = \mathbf{Q}\mathbf{u}, \qquad \mathbf{V}\mathbf{q} = \mathbf{Q}\mathbf{v}, \qquad \mathbf{W}\mathbf{q} = \mathbf{Q}\mathbf{w} \tag{8}$$

Calling $\mathbf{D} = \mathbf{C}^T\mathbf{Q}\mathbf{C}$ and $\mathbf{D}_f = \mathbf{C}^T\mathbf{Q}\mathbf{C}_f$, and substituting Eq. (4) and Eq. (8) into Eq. (7), we obtain the following relationships

$$\mathbf{D}\mathbf{x} = \mathbf{f}_x - \mathbf{D}_f\mathbf{x}_f, \qquad \mathbf{D}\mathbf{y} = \mathbf{f}_y - \mathbf{D}_f\mathbf{y}_f, \qquad \mathbf{D}\mathbf{z} = \mathbf{f}_z - \mathbf{D}_f\mathbf{z}_f \tag{9}$$

whose solution is

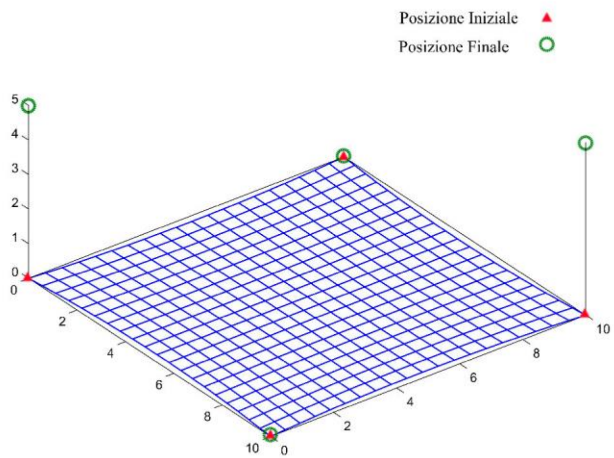
$$\mathbf{x} = \mathbf{D}^{-1}\left(\mathbf{f}_x - \mathbf{D}_f\mathbf{x}_f\right), \qquad \mathbf{y} = \mathbf{D}^{-1}\left(\mathbf{f}_y - \mathbf{D}_f\mathbf{y}_f\right), \qquad \mathbf{z} = \mathbf{D}^{-1}\left(\mathbf{f}_z - \mathbf{D}_f\mathbf{z}_f\right) \tag{10}$$

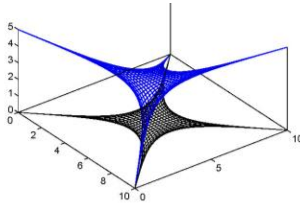
Being $\mathbf{Q}$ diagonal and $\left(\mathbf{Q}^T = \mathbf{Q}\right)$, the matrix $\mathbf{D}$ is symmetric and, for pretensioned nets, positive defined.

Given a net topology and assumed a vector $\mathbf{q}$ of force densities, Eq. (10) allows us to find the unique equilibrium configuration of the system.

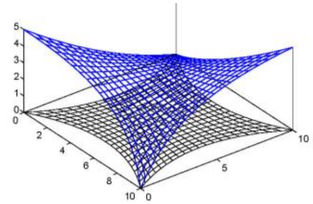
CABLE NETS

INFLUENCE OF THE FORCE DENSITY CHOICES ON THE NET CONFIGURATION

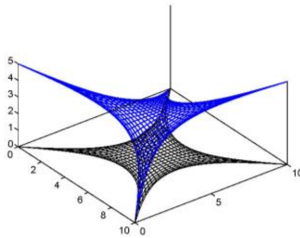




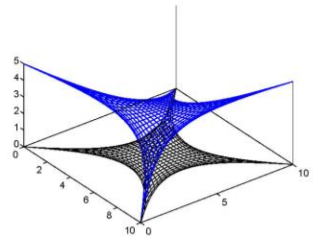
(a) Elementi di bordo: $q = 1$, elementi interni: $q = 1$. Rapporto 1:1.



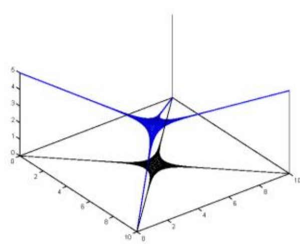
(b) Elementi di bordo: $q = 10$, elementi interni: $q = 1$. Rapporto 10:1.



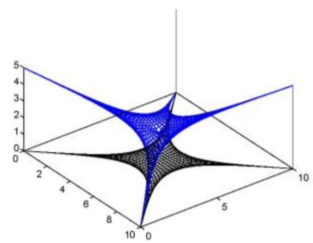
(c) Elementi di bordo: $q = 10$, elementi interni: $q = 5$. Rapporto 2:1.



(d) Elementi di bordo: $q = 20$, elementi interni: $q = 10$. Rapporto 2:1.



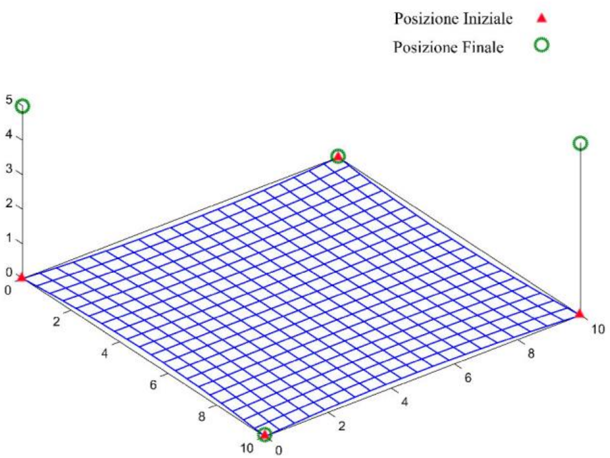
(e) Elementi di bordo: $q = 1$, elementi interni: $q = 5$. Rapporto 1:5.

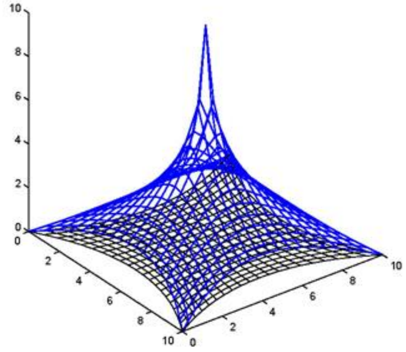


(f) Elementi di bordo: $q = 1$, elementi interni: $q = 1$. Rapporto 1:1. Carico concentrato in centro $f_z = -1$.

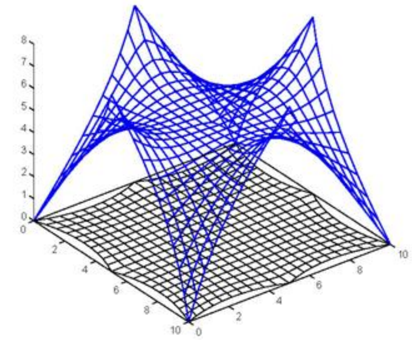
CABLE NETS

INFLUENCE OF THE FORCE DENSITY CHOICES ON THE NET CONFIGURATION

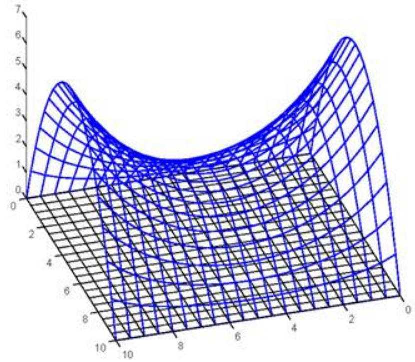




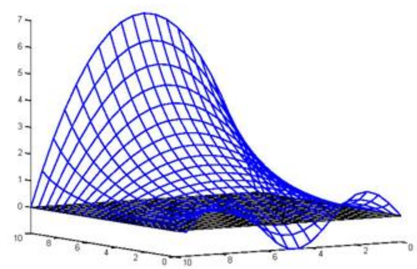
(a) Elementi di bordo: $q = 1$, elementi interni: $q = 1$.



(b) Rete di cavi 441 nodi, 840 elementi.

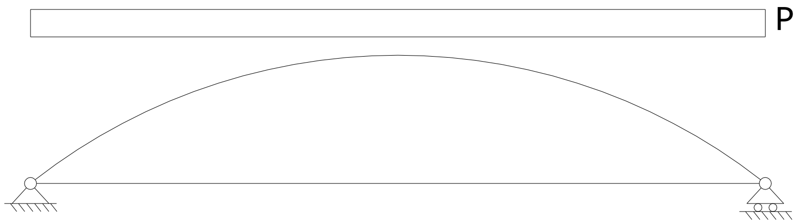


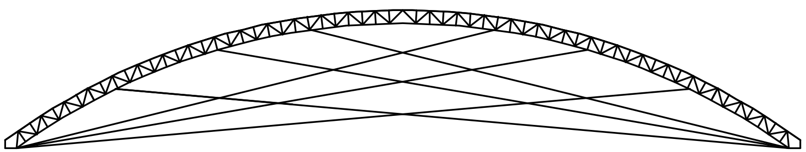
(c) Rete vincolata su quattro lati per formare una sella.

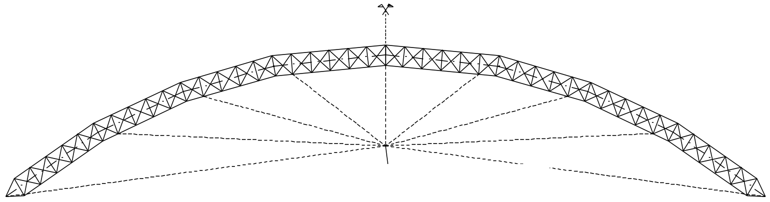


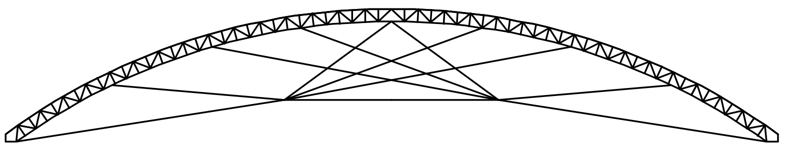
(d) Rete vincolata su quattro lati, uno parabolico, uno sinusoidale, gli altri due fissi nel piano xy.

CONSTRAINTS AND COMBINED SYSTEMS *(CABLES AND BARS OR BEAM ELEMENTS)*









IMPROVING THE ARCH BEHAVIOUR THROUGH CONSTRAINTS

A correctly designed arch is an highly efficient structural system:

- All section tend to be fully compressed
- The structural volume tends to be a minimum

To exploit these characteristics, the ends of the arch have to be properly restrained by suitable vertical and horizontal reactions.

Inefficient end supports caused the lost of the majority of the arch structures of the past.

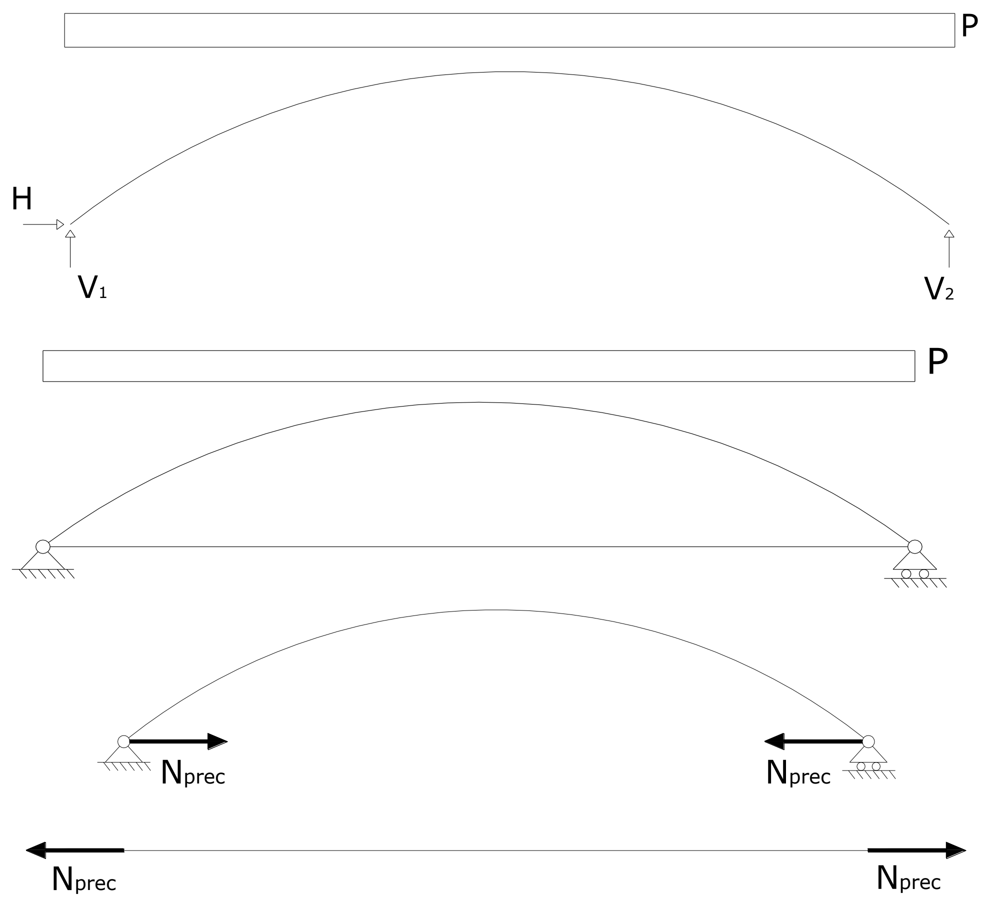
An evolution of the arch concept arose from the possibility to avoid the need of an external horizontal thrust, by introducing a tie along the chord.

ARCH AND TIED ARCH

1. ARCH RESTRAINED BY THE FOUNDATION SYSTEM.

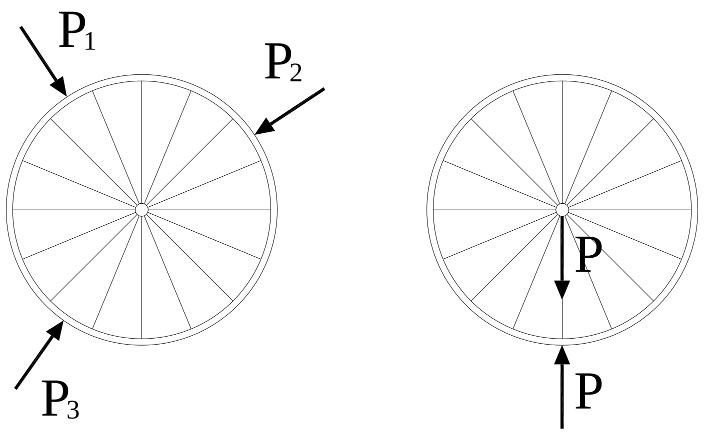
2. IN THE TIED ARCH, THE HORIZONTAL THRUST IS CARRIED OUT BY THE TIE.

3. THE PRETENSIONING ALLOWS TO CONTROL THE STRESSES IN THE ARCH AND THE RELATIVE DISPLACEMENT BETWEEN THE TWO ENDS



EXAMPLE OF AN EFFICIENT COMBINED SYSTEM

In a tied-arch bridge, the tie is the deck, which has a relevant flexural stiffness. This suggested several improved schemes called *combined arch - beam systems* (Nielsen, Langer, etc.).



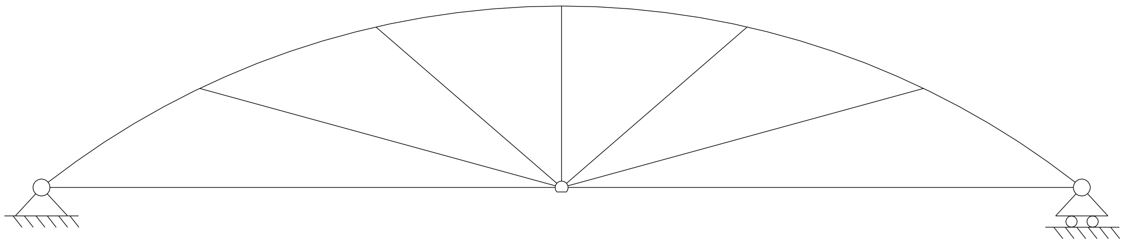
A very *efficient combined system* is the bicycle wheel: a ring, reinforced by a large number of pre tensioned spokes, which behaves like a beam on elastic foundation.

The external loading is balanced by the resultant of the spoke forces, which is transmitted through the axle.

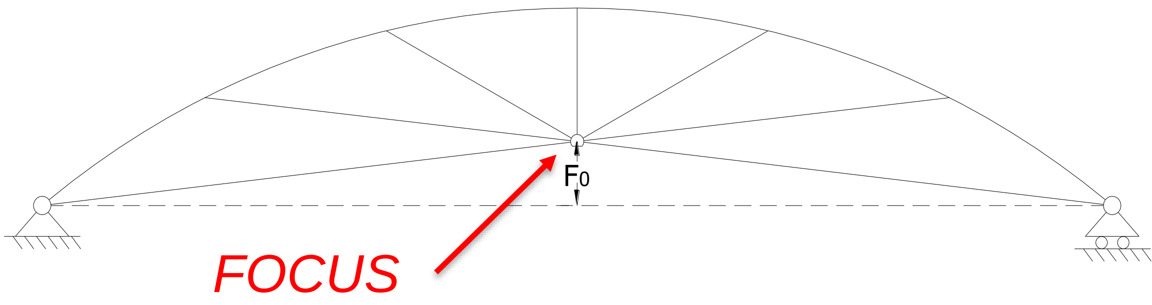
FROM THE BICYCLE WHEEL TO A CABLE STAYED ARCH

. . . And if we extend this concept to an arch ?

The simple addition of stays don't improve the structural performances.

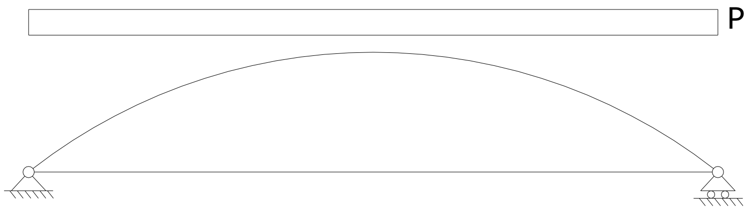


A significant evolution of the static behaviour can be achieved by moving the "focus" position.

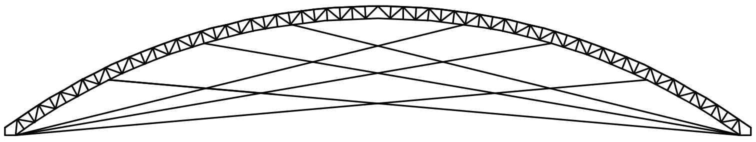


CABLE STAYED ARCHES

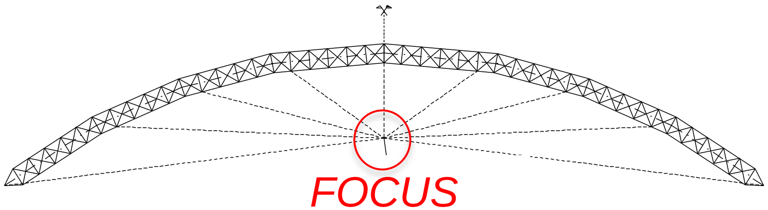
The tied arch the simplest form of cable stayed arch.



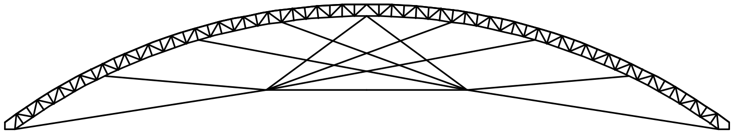
The ends are connected to others points along the arch through two fans of stays.



The ends and others points of the arch are connected to a central pin (“focus”)



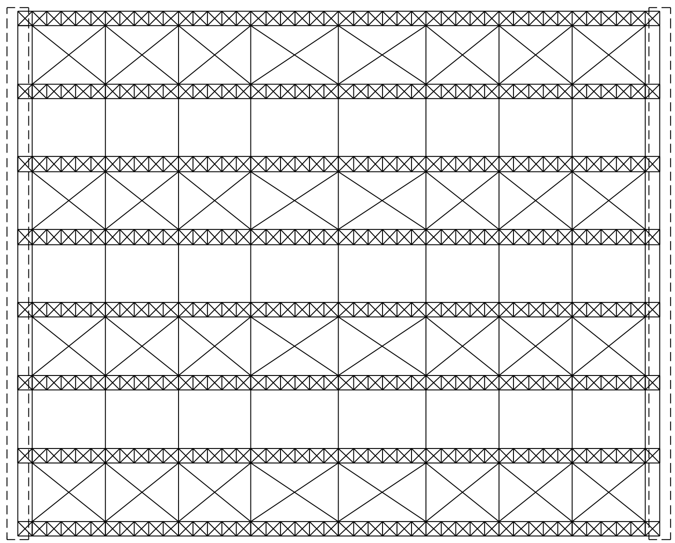
Other mixed stay configurations can be proposed (From Belenya, Moscow, 1977)

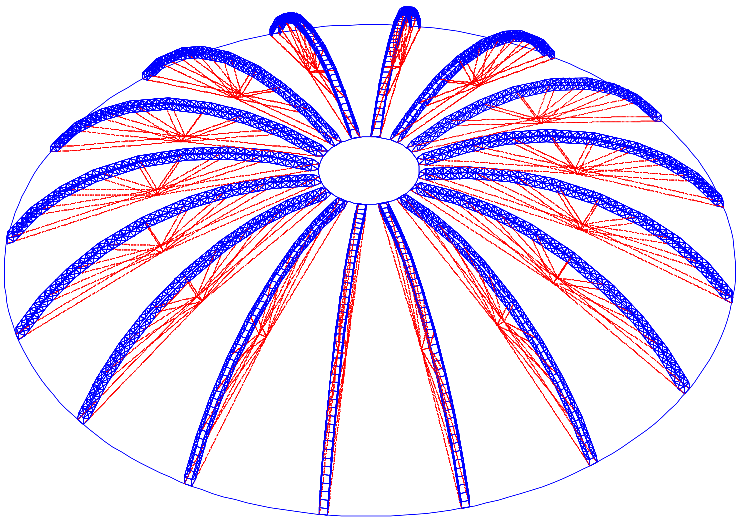


CABLE STAYED ARCHES

In all cases, the contribution of the stays (additional internal elastic restraints and stay tensioning) suggests new ways to condition the arch behaviour, to increase its stiffness and the level its critical loads.

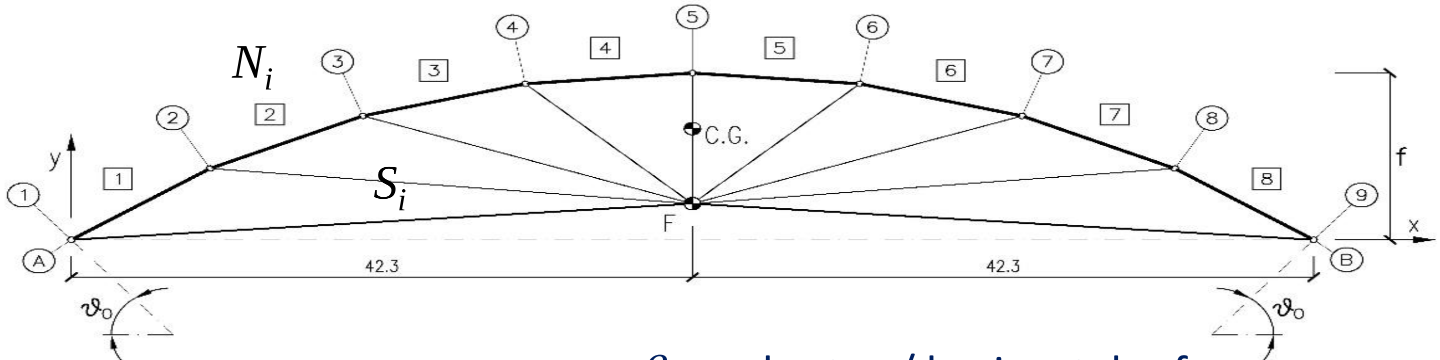
Assembly of stayed arches can be adopted in large span roofs or in domes.





CABLE STAYED ARCHES

How does a cable stayed arch work ?



α_i angle arch segment / horizontal ref.

β_i angle stay / horizontal ref.

Equilibrium Equation at node 1

$$-N_1 \cdot cos\,\alpha_1 + S_1\, cos\,\beta_1 = 0$$

$$-N_1 \cdot sin\,\alpha_1 - S_1\, sin\,\beta_1 + V_H - F_1 = 0$$

Equilibrium Equation at node ith
(recursive formula)

$$N_{i-1} \cdot cos\,\alpha_{i-1} - N_i\, cos\,\alpha_i + S_i\, cos\,\beta_i = 0$$

$$N_{i-1} \cdot sin\,\alpha_{i-1} - N_i\, sin\,\alpha_i - S_i\, sin\,\beta_i - F_i = 0$$

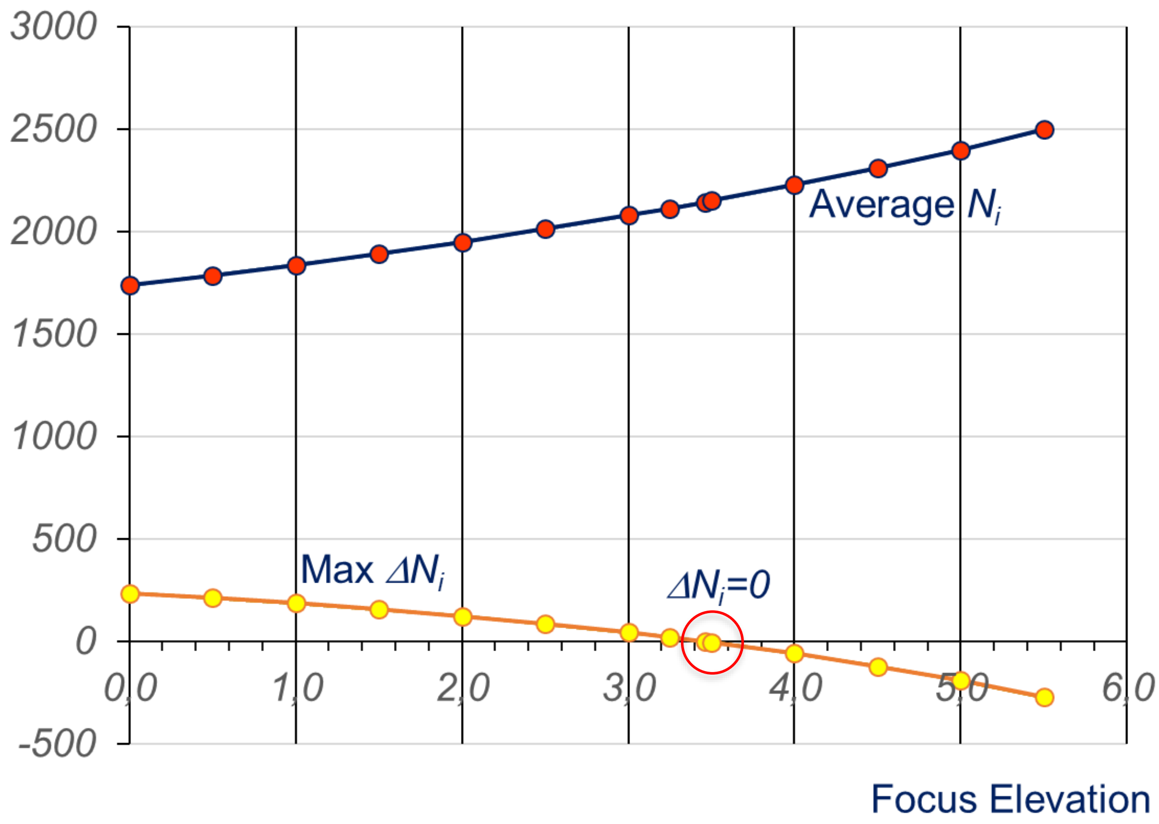
CABLE STAYED ARCHES

Varying the focus position, the average axial force N_i and the difference between max and min axial forces ΔN_i vary as in the diagram.

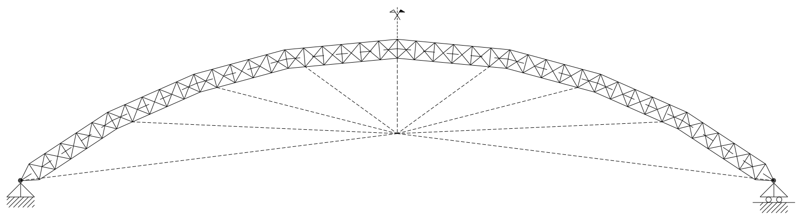
For the focus elevation

$y_F = 3{,}46$ m

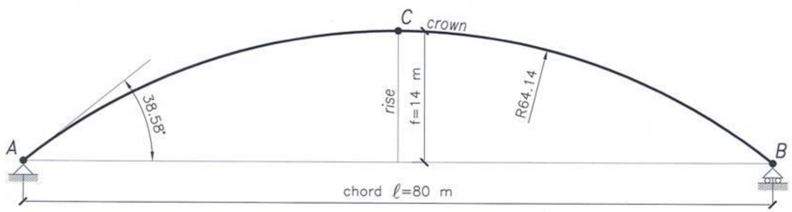
such difference is $\Delta N_i=0$



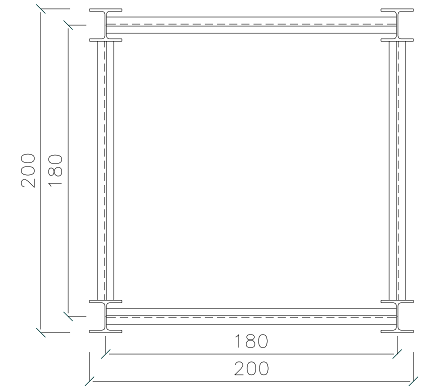
AN ENGINEERING APPLICATION



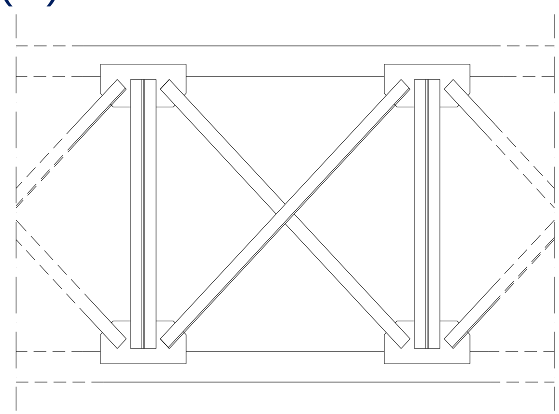
(a) View of the truss cable stayed arch.



(b) Radius=63.14m; Span=80m; Rise=14m



(c) Cross Section: 4xHE200B



(d) Bracing:

- Vertical L 70x7 Diagonal L 90x9

<u>LOADING CONDITIONS</u>:

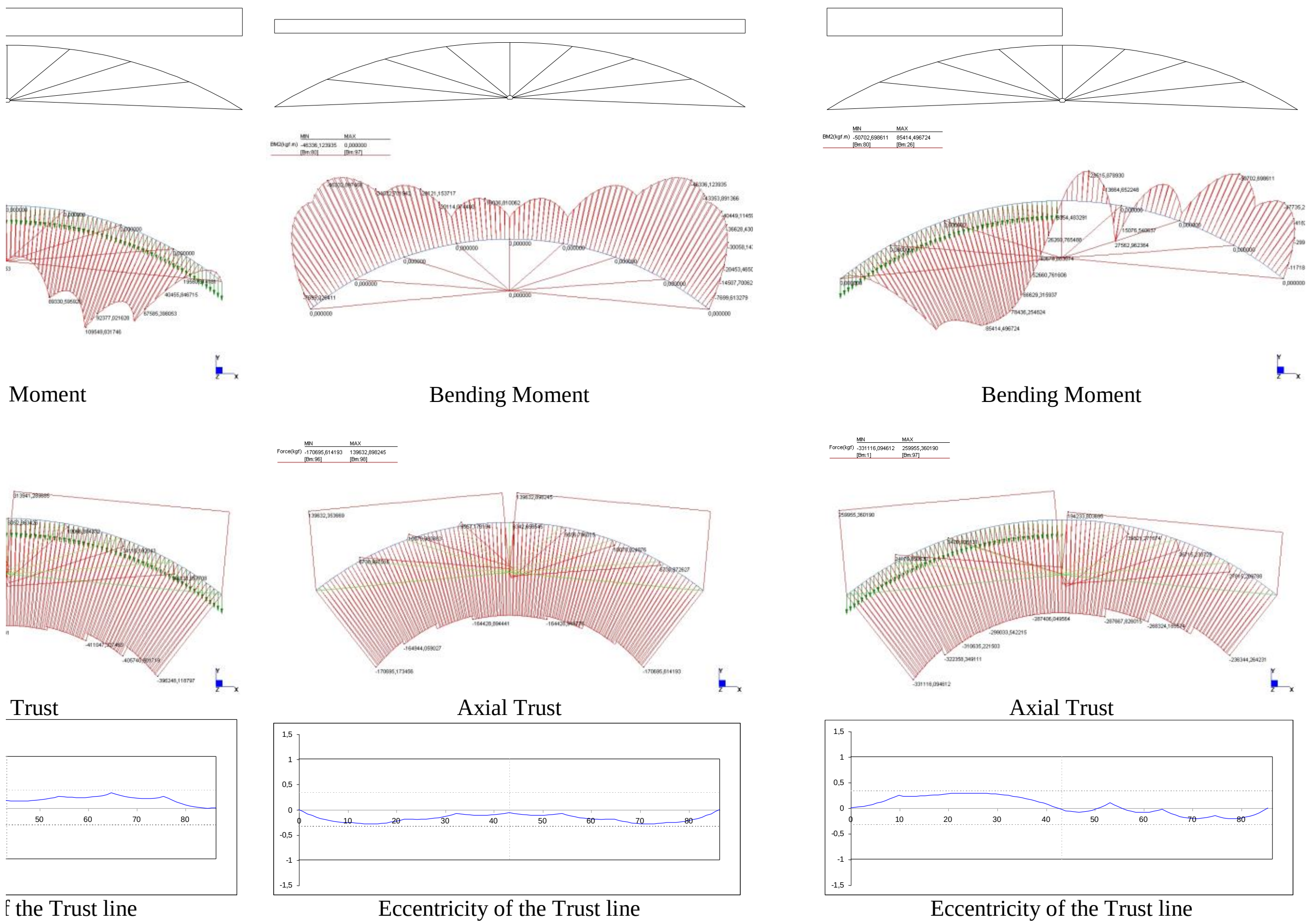
1 - Dead Load + 50% Live Load (Ref. for Pretension)

2 - Dead Load + 100% Live Load

3 - Dead Load + 0% Live Load

4 - Dead Load + 100% Live Load on Half Span

AN ENGINEERING APPLICATION



AN ENGINEERING APPLICATION

Comparison among the characteristic parameters of the Cable Stayed Arch and those of no pre-tensioned and pre-tensioned tied arches (L.C. 4)

	N_{max} *[kN]*	M_{max} *[kNm]*	e_{max} *[cm]*	k_{min}
a) Arch with horiz. non tensioned tie	2039	3940	193	11,6
b) Arch with horizontal tensioned tie	2117	3267	154	10.8
c) Cable Stayed Arch	3311	854	28	37

AN ENGINEERING APPLICATION

"THE FOCUS "

A VIEW DURING THE ERECTION STAGE

AN ENGINEERING APPLICATION

A VIEW DURING THE ERECTION STAGE

TIME DEPENDENT EFFECTS DUE TO APPLIED CONSTRAINTS

CREEP AND SHRINKAGE PHENOMENA IN REINFORCED AND PRESTRESSED CONCRETE STRUCTURES . . .

- MAY HAVE LITTLE INFLUENCE ON S.L.S. AND U.L.S.

OR

- MAY PLAY A CRUCIAL ROLE FOR THE REQUIRED PERFORMANCES A THE BEGINNING AND AT THE END OF THE EXPECTED SERVICE LIFE.

IN THE CASE OF BRIDGES, PAST EXPERIENCES SHOWED US SIGNIFICANT DRAWBACKS WITH REGARD TO SERVICEABILITY WITH . . .

- REDUCTION OF DURABILITY AND RIDE COMFORT
- EXCESSIVE VERTICAL DISPLACEMENTS
- SLOPE DISCONTINUITIES
- LONGITUDINAL SHORTENING.

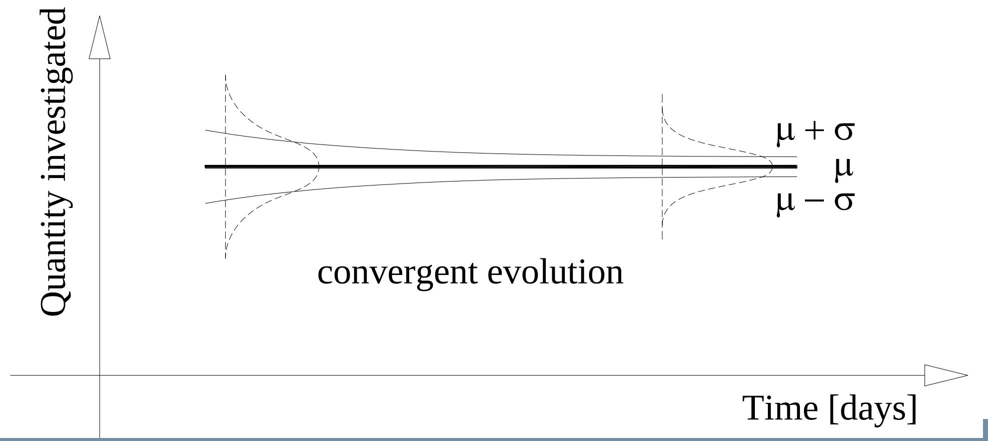
- THE CONVENTIONAL ASSESSEMENT OF THE TIME DEPENDENT BEHAVIOUR IS BASED ON DETERMINISTIC HYPOTHESES.
- GEOMETRY, MATERIAL CHARACTERISTICS, TENSIONING FORCES ARE KNOWN AND CERTAIN DATA.
- IN TRUTH, THESE DATA INVOLVES MANY UNCERTAINTIES:
- CREEP MODELS, ENVIRONMENTAL PARAMETERS (T°, R.H.%), ACTUAL ON SITE TECHNOLOGIES, . . .
- THROUGH A PROBABILISTIC APPROACH THE EFFECTS OF LARGE EFFECTS OF LARGE VARATIONS OF THESE BASIC VARIABLES IS HIGHLIGHTED.

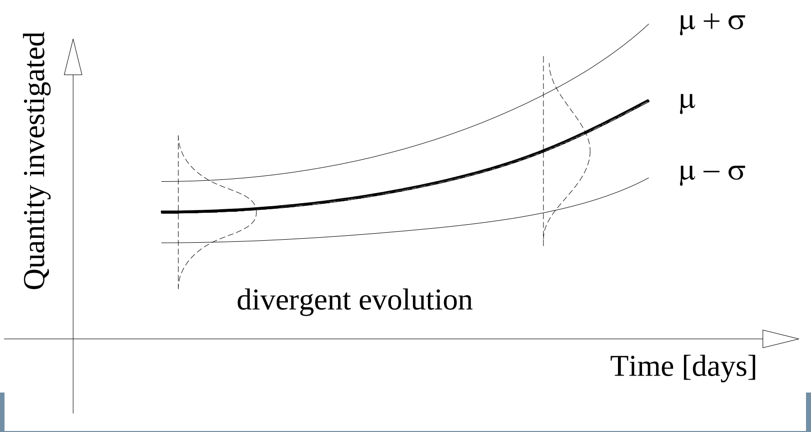
In such type of analyses, many uncertain quantities are involved. We consider

- environmental factors
- material characteristics
- tension in the cables

as uncertain parameters. These uncertainties are modelled through normal Gaussian distributions, by the mean value and by the standard deviation.

We want to assess the sensitivity over time of structural systems with respect to a certain level of the dispersion of the initial data.





Sensitivity is measured on quantities concerning the safety and/or the serviceability of the structure.

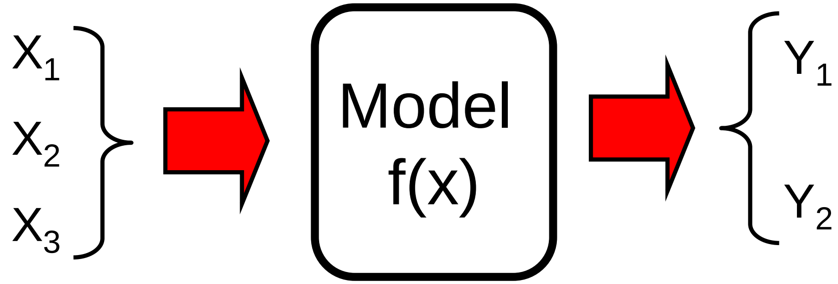
(a)

(b)

in a cable **stayed bridge** these quantities should be the **tension in the cables**
In a **cantilever bridge** it should be the **vertical displacement** of a particular point, assumed as indicator of an excessive deformed shape.

Note: (a) Palizzi Tranway Cable Stayed bridge, designed by Francesco Martinez y Cabrera, 1995.

Deterministic model



Through Monte Carlo method the **uncertainty propagation** is analyzed.

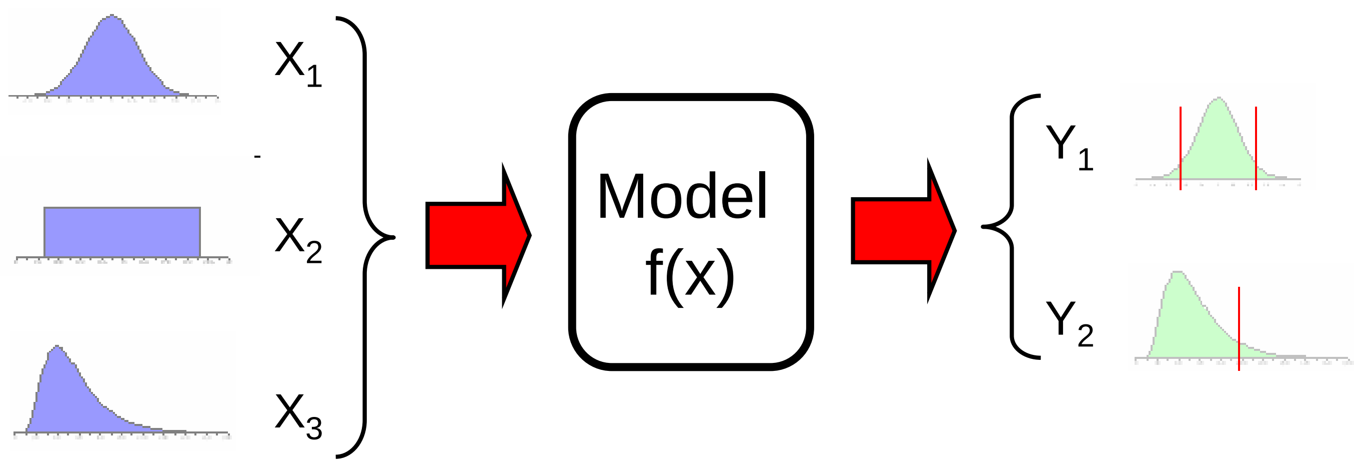
Step 1: Create a parametric model, $y = f(x_1, x_2, ..., x_q)$

Step 2: Generate a set of random inputs, $x_{i1}, x_{i2}, ..., x_{iq}$

Step 3: Evaluate the model and store the results as y_i

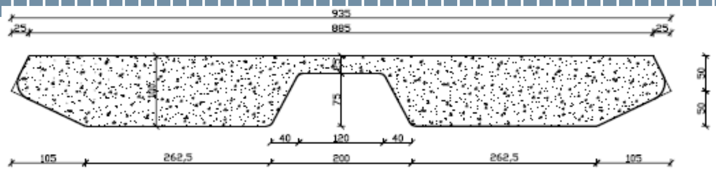
Step 4: Repeat steps 2 and 3 for $i = 1$ to n.

Step 5: Analyze the results through histograms, summary statistics, etc.

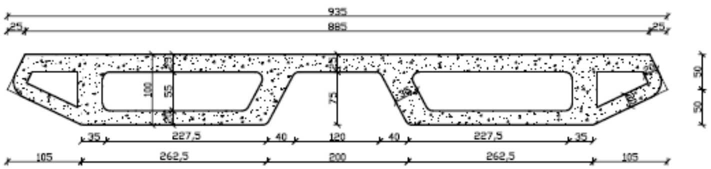


AN URBAN CABLE STAYED BRIDGE - PALIZZI BRIDGE – MARTINEZ Y C.

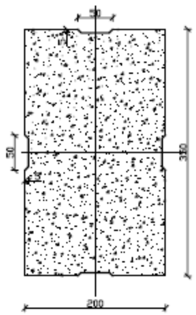
SPANS: 20+22+24/28.5-19.5-24-10 m

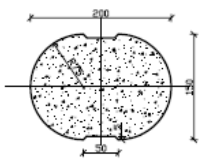


SEZ A
SEZIONE CORRENTE PIENA
Area: 74718.7 cm^2
Perimetro: 2017.0 cm
Momento d'inerzia Jc: 59940433.1 cm^4
Rapporto geometrico ho:740.889 mm



SEZ B
SEZIONE CORRENTE CELLULARE
Area: 46364.0 cm^2
Perimetro: 3510.0 cm
Momento d'inerzia Jc: 52101743.8 cm^4
Rapporto geometrico ho: 264.18 mm

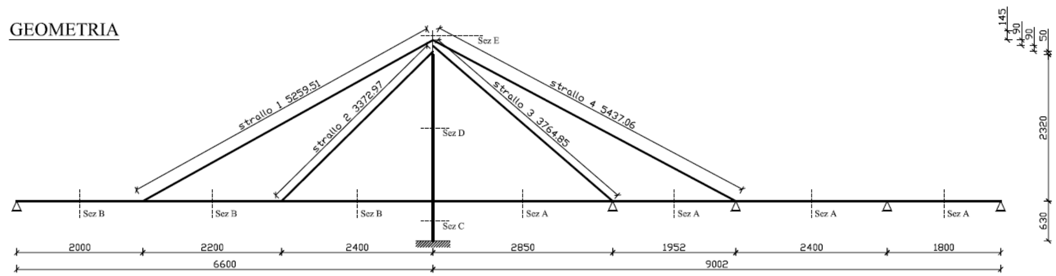




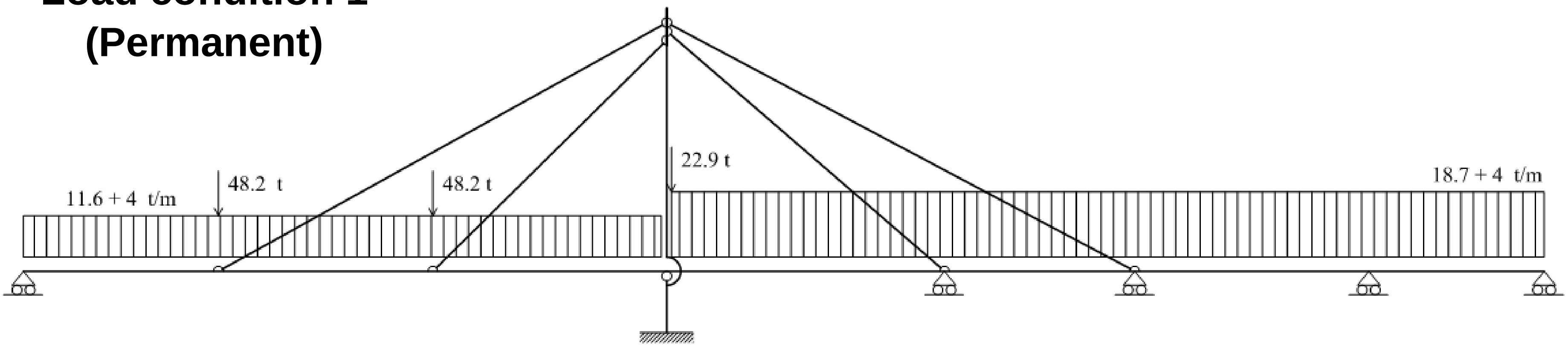
SEZ C
BASAMENTO ANTENNA
Area: 69100.0 cm^2
Perimetro: 1116.6 cm
Momento d'inerzia
Jc: 701100833.3 cm^4
Rapporto geometrico ho: 1237.7 mm

SEZ D
ELEVAZIONE ANTENNA
Area: 32221.5 cm^2
Perimetro: 679.5 cm
Momento d'inerzia
Jc: 137702260.5 cm^4
Rapporto geometrico ho: 948.38 mm

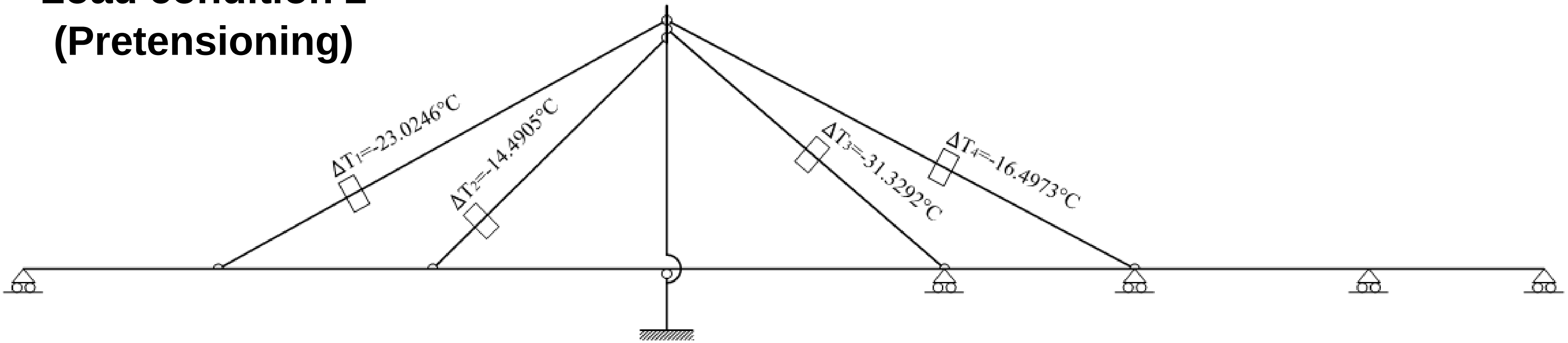
SEZ E
DISPOSITIVO DI
ANCORAGGIO
Area: 2400 cm^2
Momento d'inerzia
Js: 8000000 cm^4

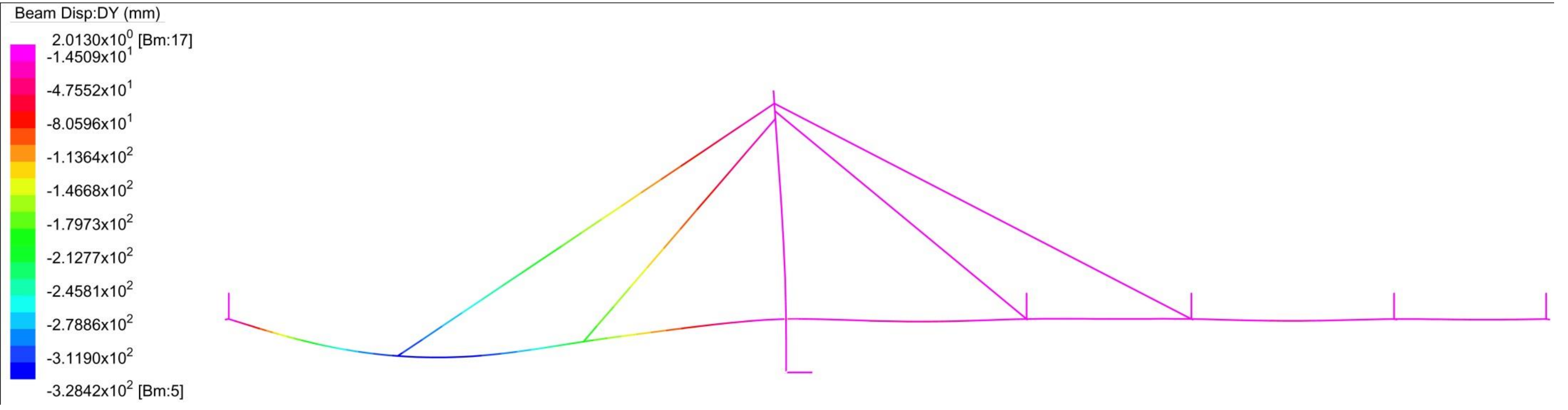


Load condition 1 (Permanent)

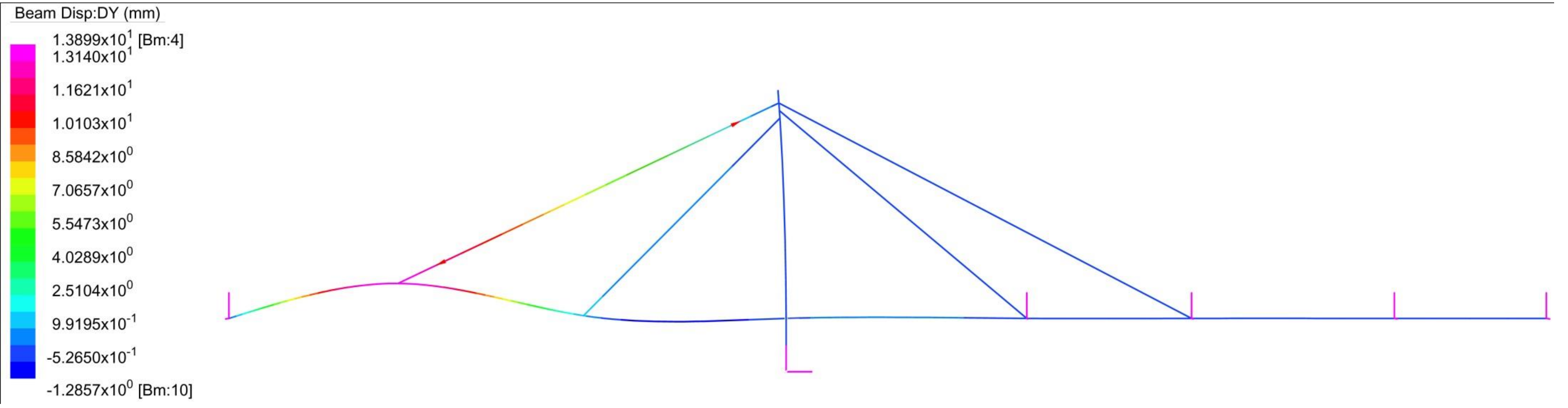


Load condition 2 (Pretensioning)

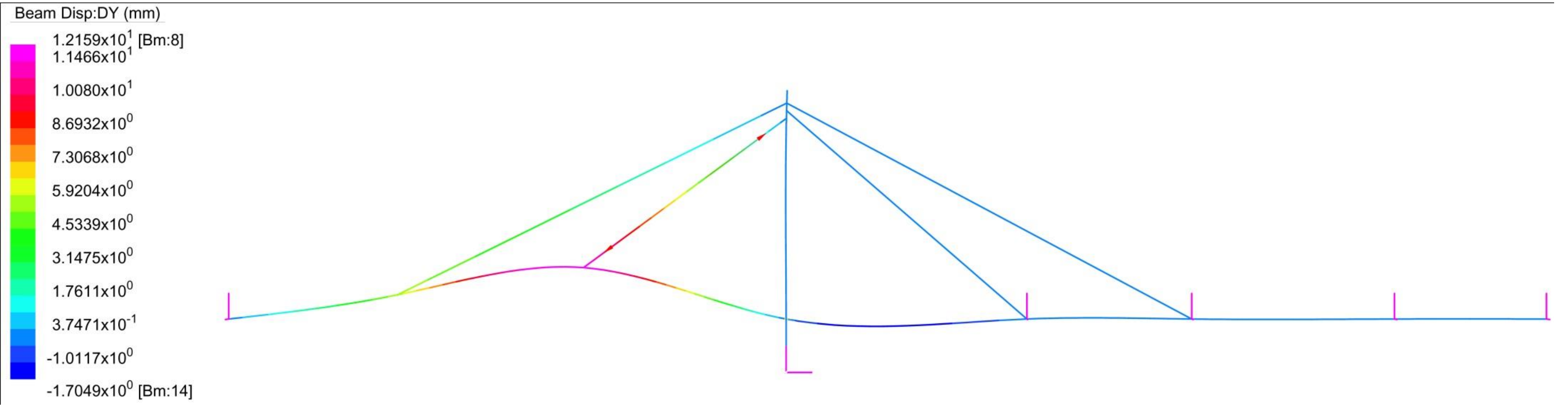




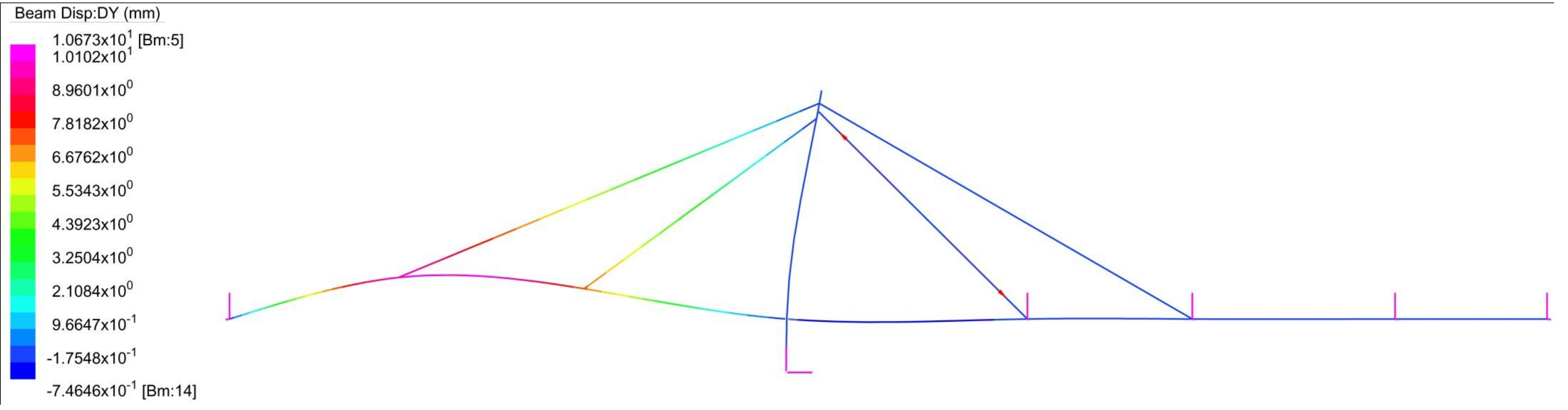
1. **Permanent load**
2. Pretensioning – Cable 1
3. Pretensioning – Cable 2
4. Pretensioning – Cable 3
5. Pretensioning – Cable 4
6. Permanent load + Pretensioning (1+2+3+4)



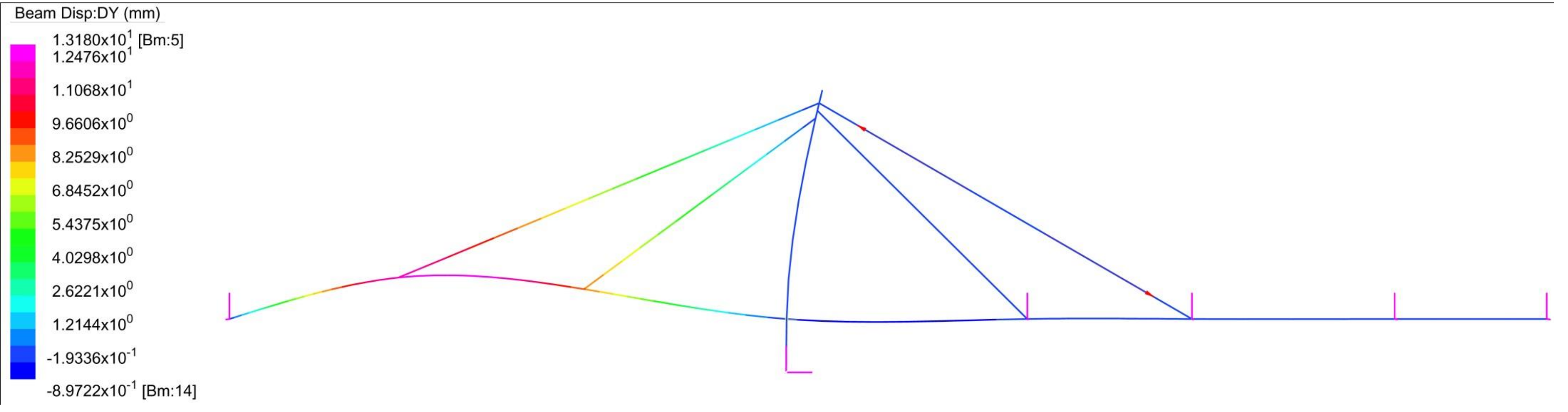
1. Permanent load
2. **Pretensioning – Cable 1**
3. Pretensioning – Cable 2
4. Pretensioning – Cable 3
5. Pretensioning – Cable 4
6. Permanent load + Pretensioning (1+2+3+4)



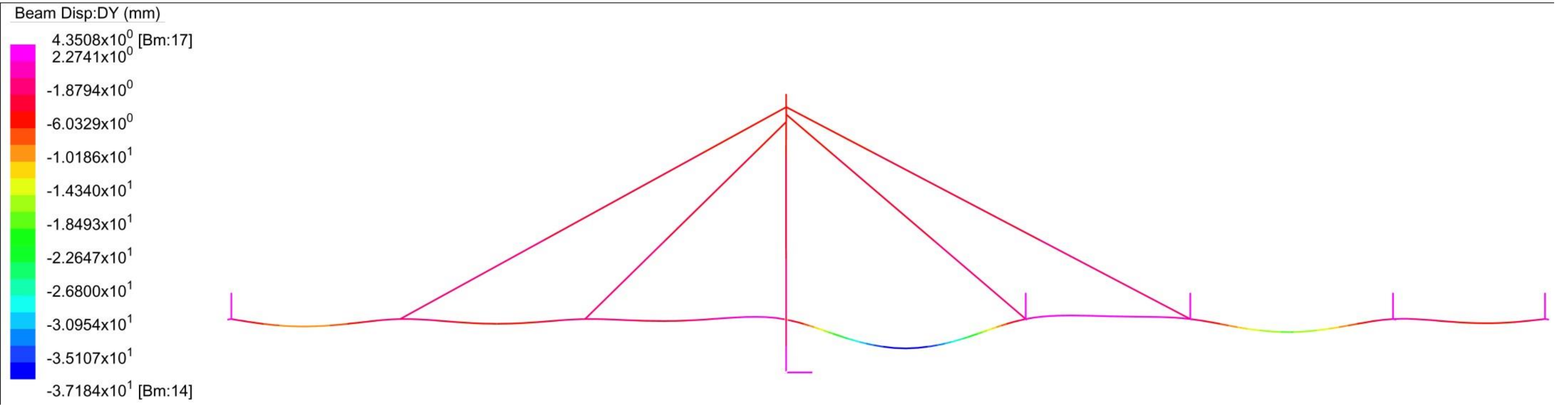
1. Permanent load
2. Pretensioning – Cable 1
3. **Pretensioning – Cable 2**
4. Pretensioning – Cable 3
5. Pretensioning – Cable 4
6. Permanent load + Pretensioning (1+2+3+4)



1. Permanent load
2. Pretensioning – Cable 1
3. Pretensioning – Cable 2
4. **Pretensioning – Cable 3**
5. Pretensioning – Cable 4
6. Permanent load + Pretensioning (1+2+3+4)

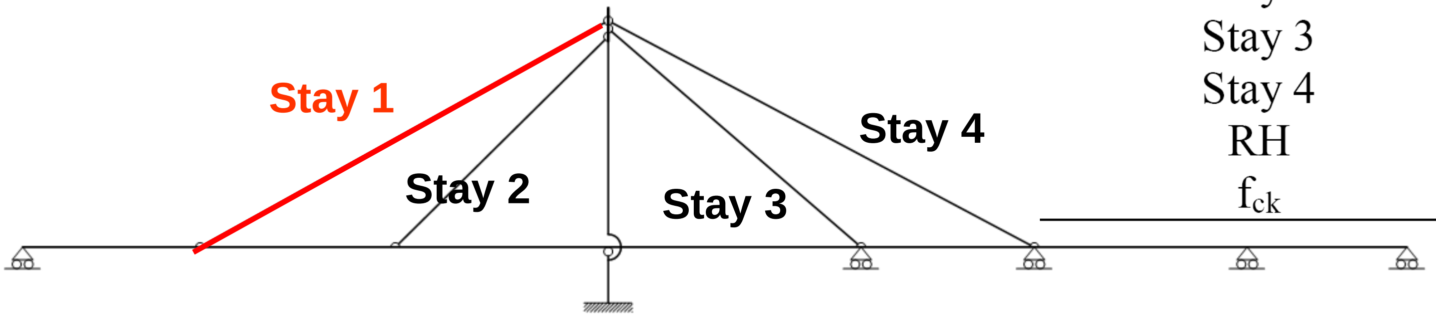


1. Permanent load
2. Pretensioning – Cable 1
3. Pretensioning – Cable 2
4. Pretensioning – Cable 3
5. **Pretensioning – Cable 4**
6. Permanent load + Pretensioning (1+2+3+4)

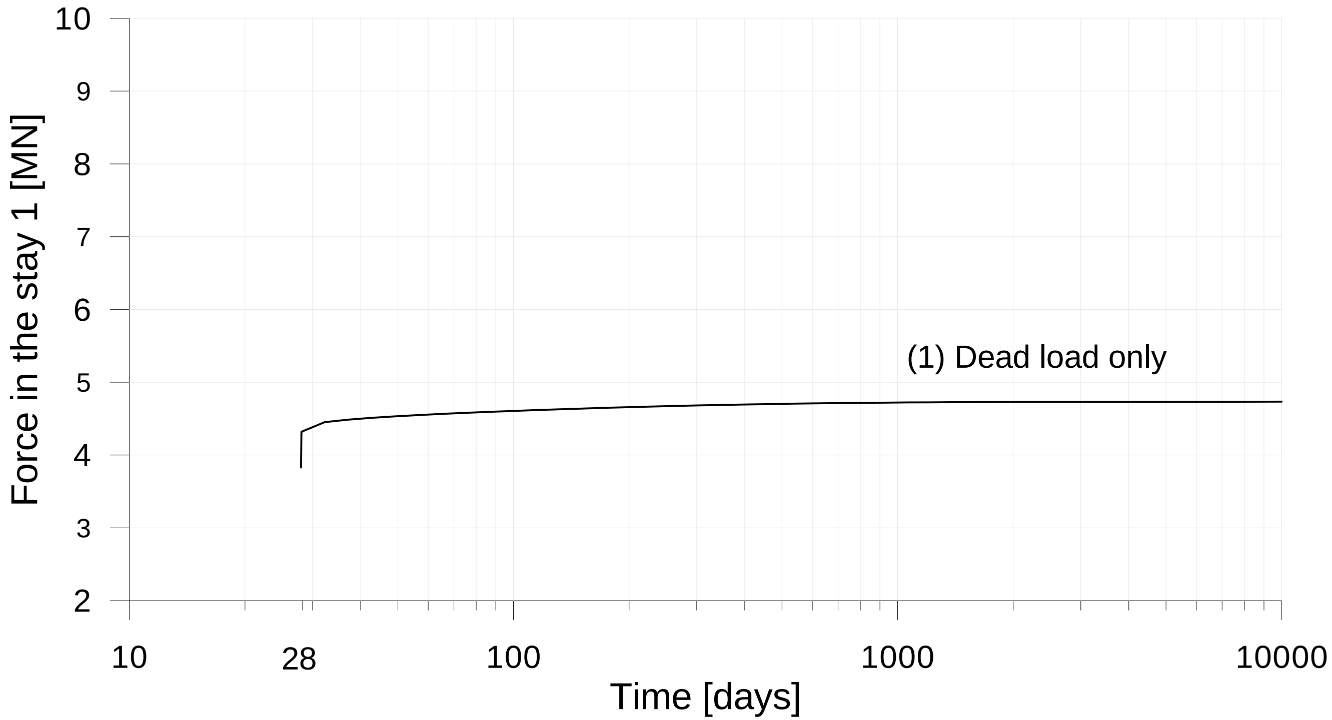


1. Permanent load
2. Pretensioning – Cable 1
3. Pretensioning – Cable 2
4. Pretensioning – Cable 3
5. Pretensioning – Cable 4
6. **Permanent load + Pretensioning (1+2+3+4)**

Element or parameter	μ (mean)
Stay 1	8675 kN
Stay 2	5241 kN
Stay 3	9170 kN
Stay 4	4924 kN



Element or parameter	μ (mean)
Stay 1	8675 kN
Stay 2	5241 kN
Stay 3	9170 kN
Stay 4	4924 kN
RH	65%
f_{ck}	48.1 MPa



Element or parameter	μ (mean)
Stay 1	8675 kN
Stay 2	5241 kN
Stay 3	9170 kN
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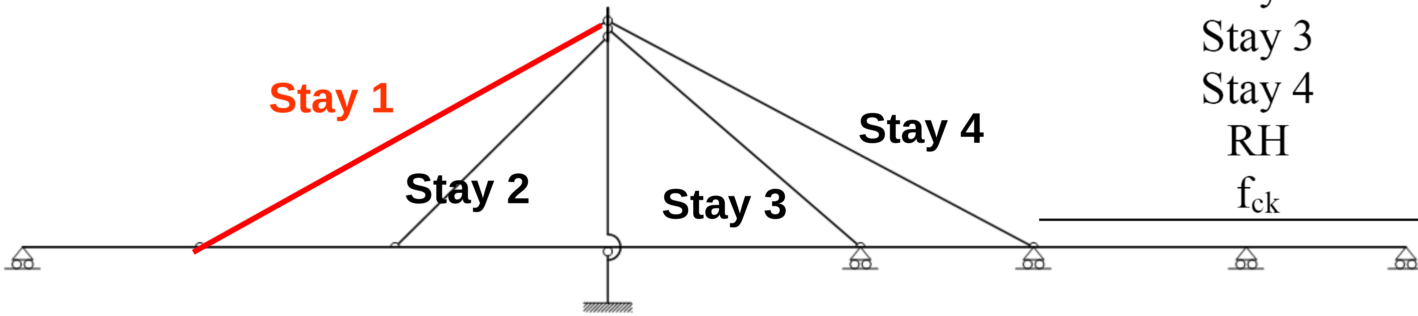
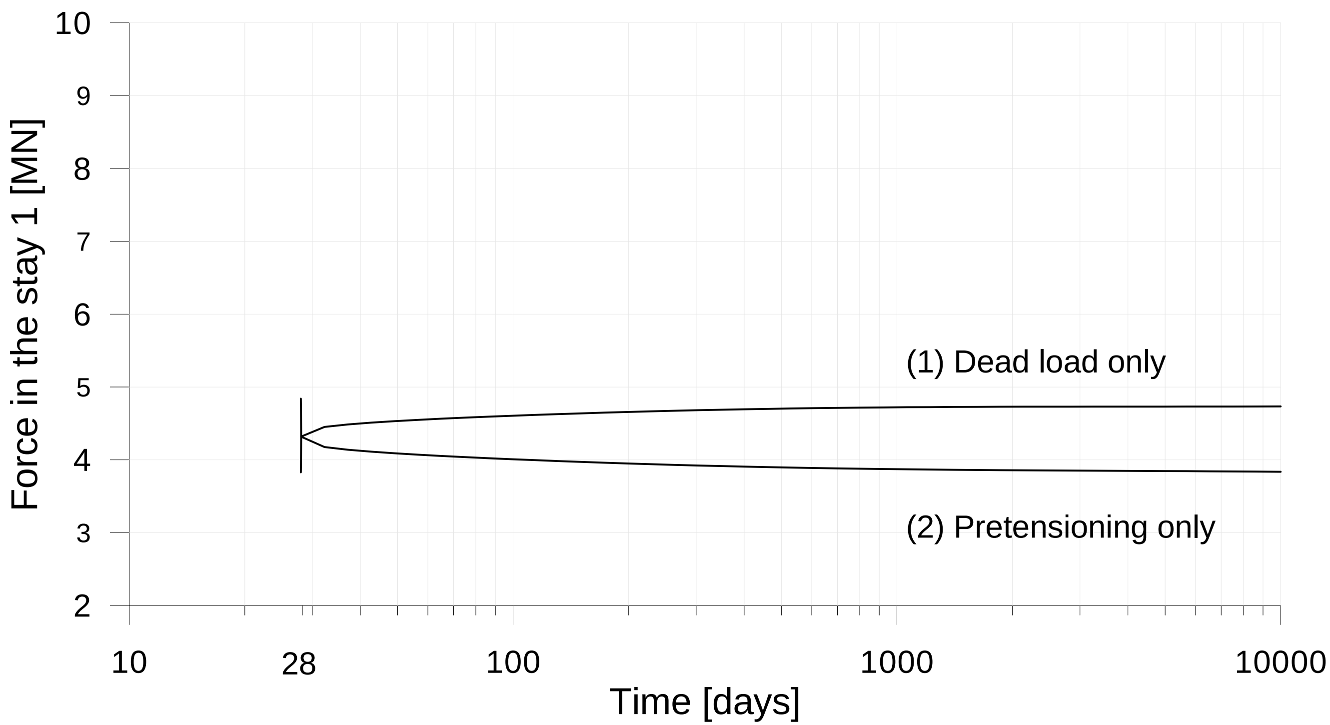
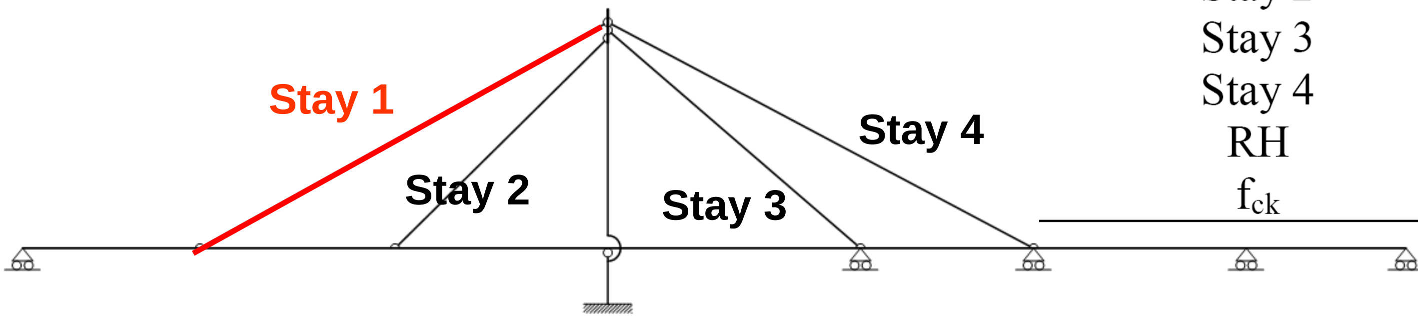
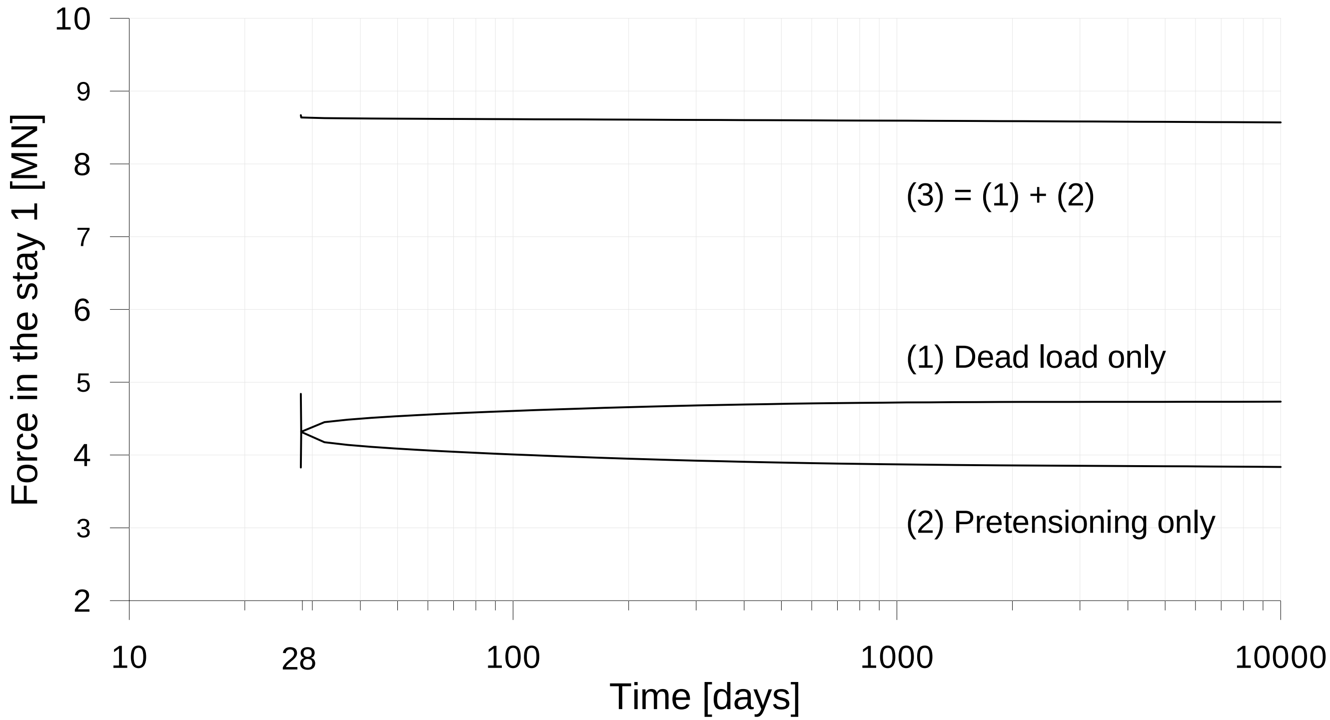


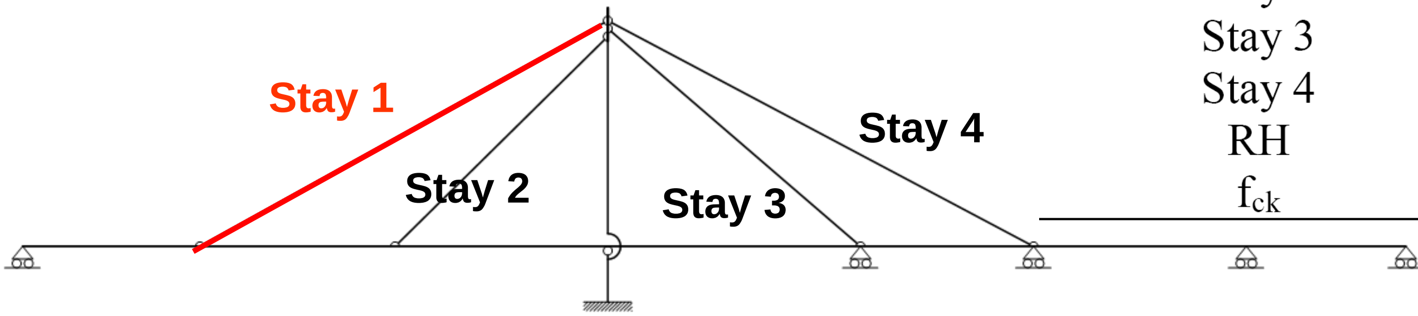


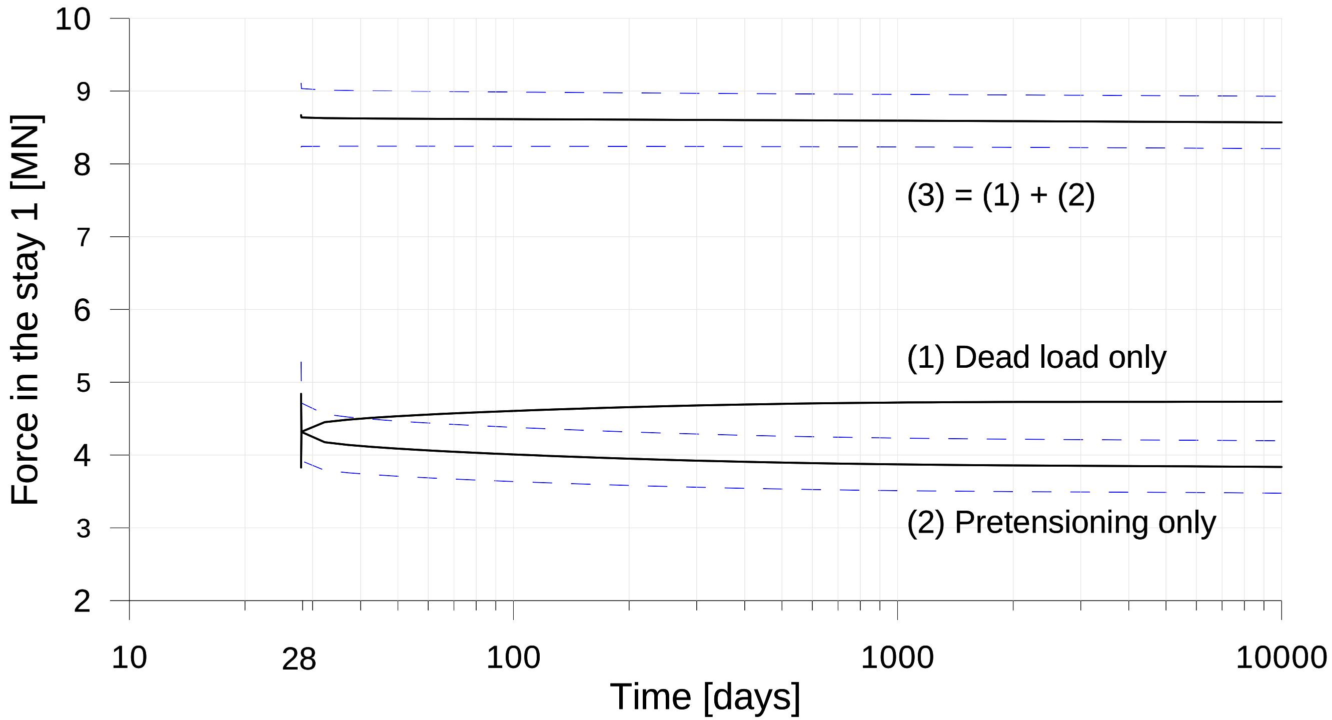
Element or parameter	μ (mean)
Stay 1	8675 kN
Stay 2	5241 kN
Stay 3	9170 kN
Stay 4	4924 kN
RH	65%
f_{ck}	48.1 MPa



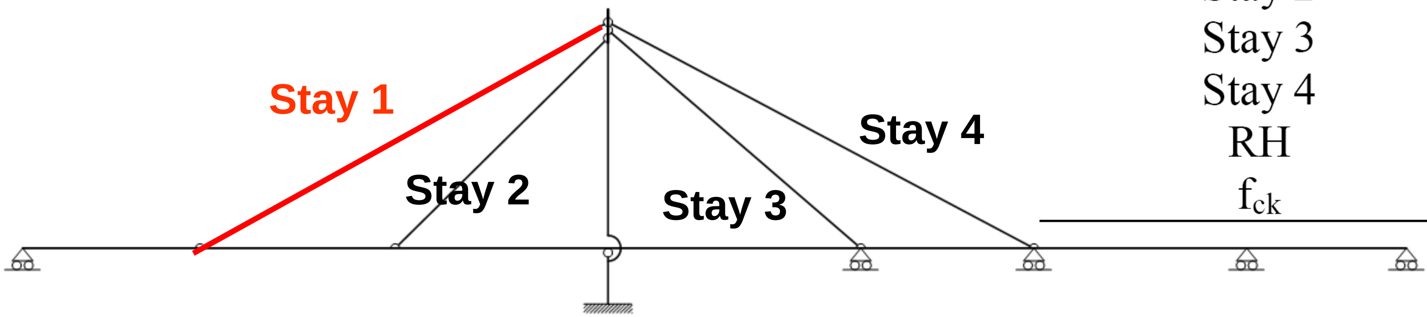


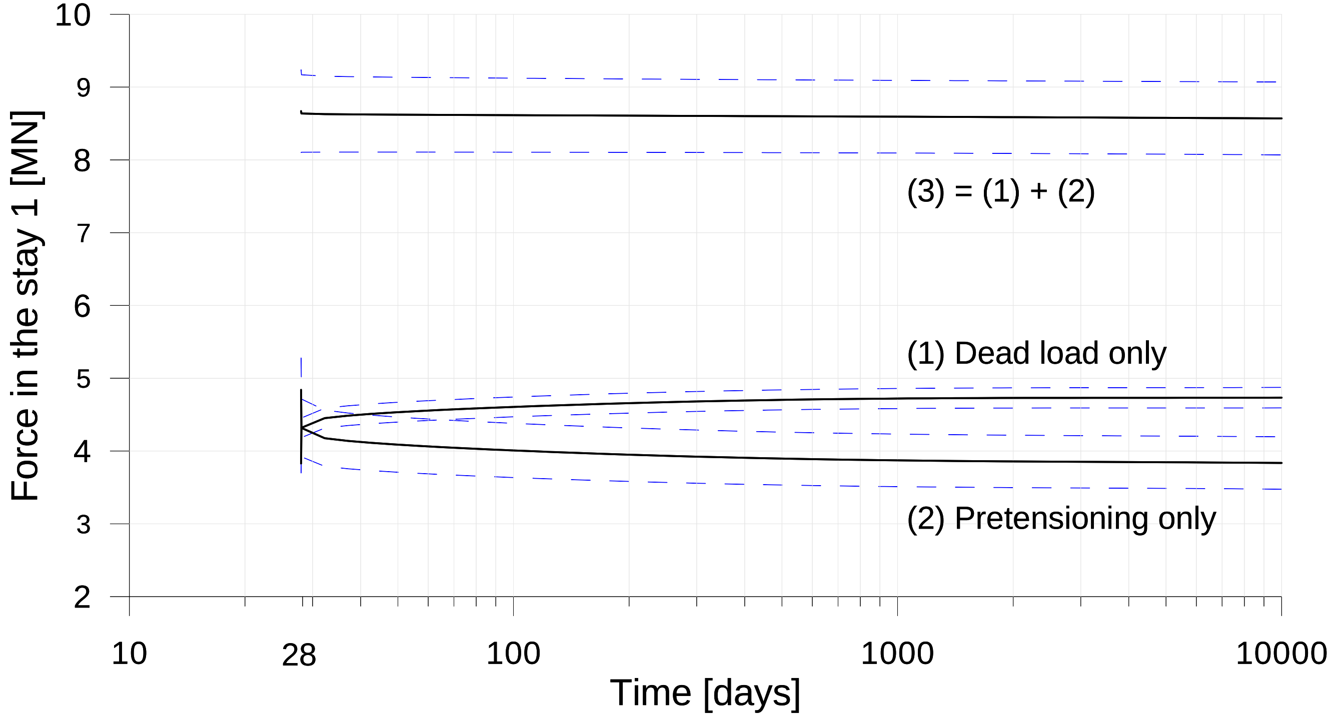
Element or parameter	μ (mean)	σ (s.d.)
Stay 1	8675 kN	10%
Stay 2	5241 kN	10%
Stay 3	9170 kN	10%
Stay 4	4924 kN	10%
RH	65%	0%
f_{ck}	48.1 MPa	0%





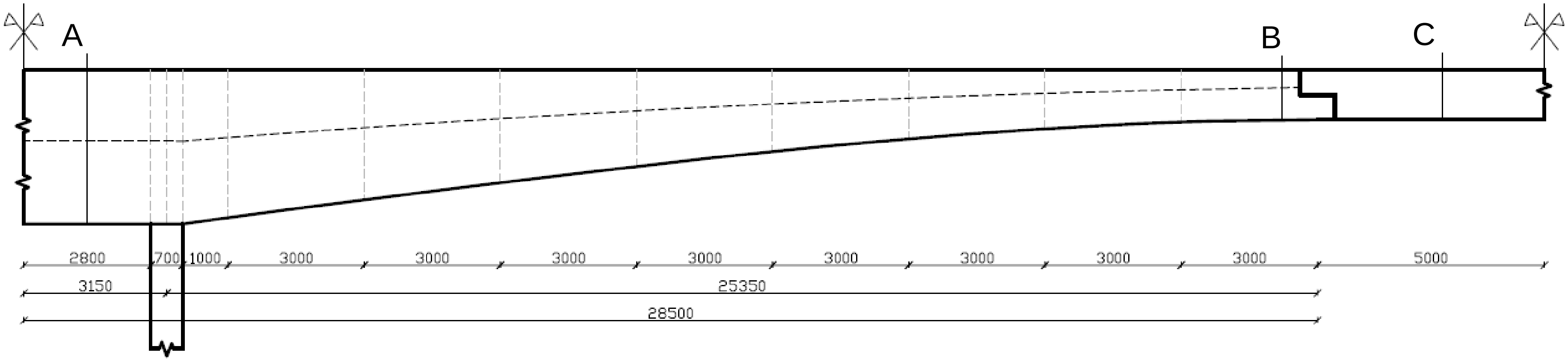
Element or parameter	μ (mean)	σ (s.d.)
Stay 1	8675 kN	10%
Stay 2	5241 kN	10%
Stay 3	9170 kN	10%
Stay 4	4924 kN	10%
RH	65%	20%
f_{ck}	48.1 MPa	20%



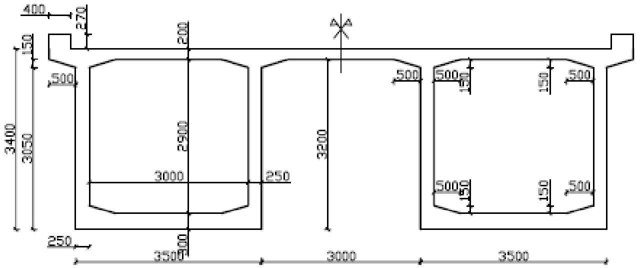


Bridge made of short cantilevers spanning 28.5 m and supporting a closing suspended girders, 10.0 m long. The free length of each cantilever is 25.0 m. The length between two piers of a typical reach is 67.0 m. blades

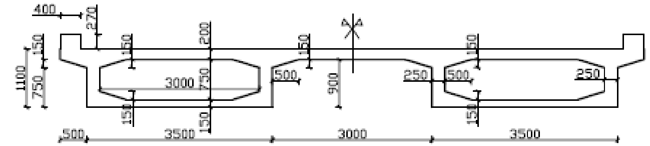
The transversal section is made of a double box girder 11.0 m wide. The piers are made of a couple of parallel reinforced concrete blades



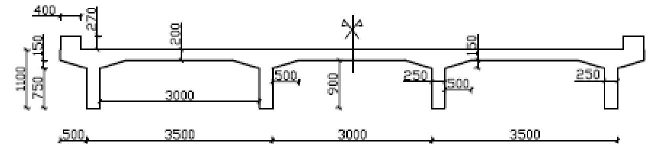
A

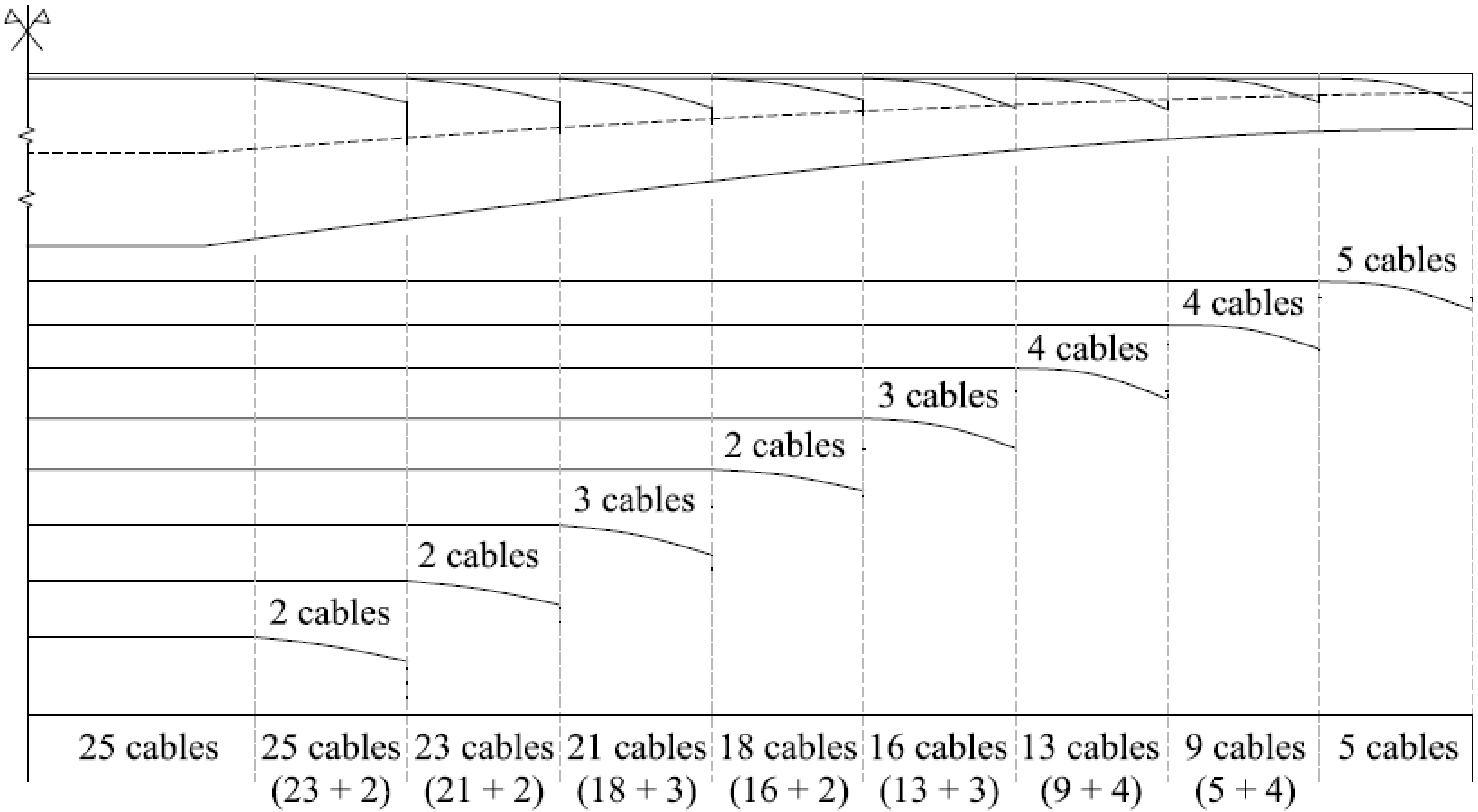


B



C

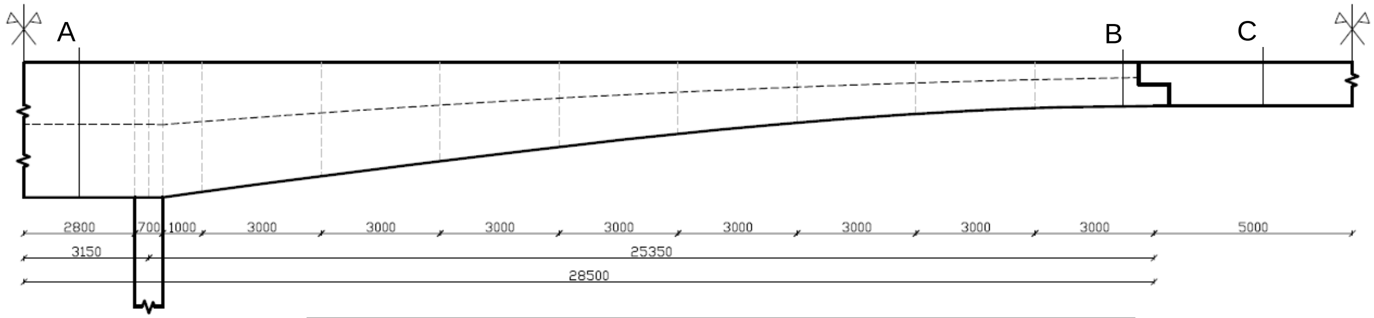




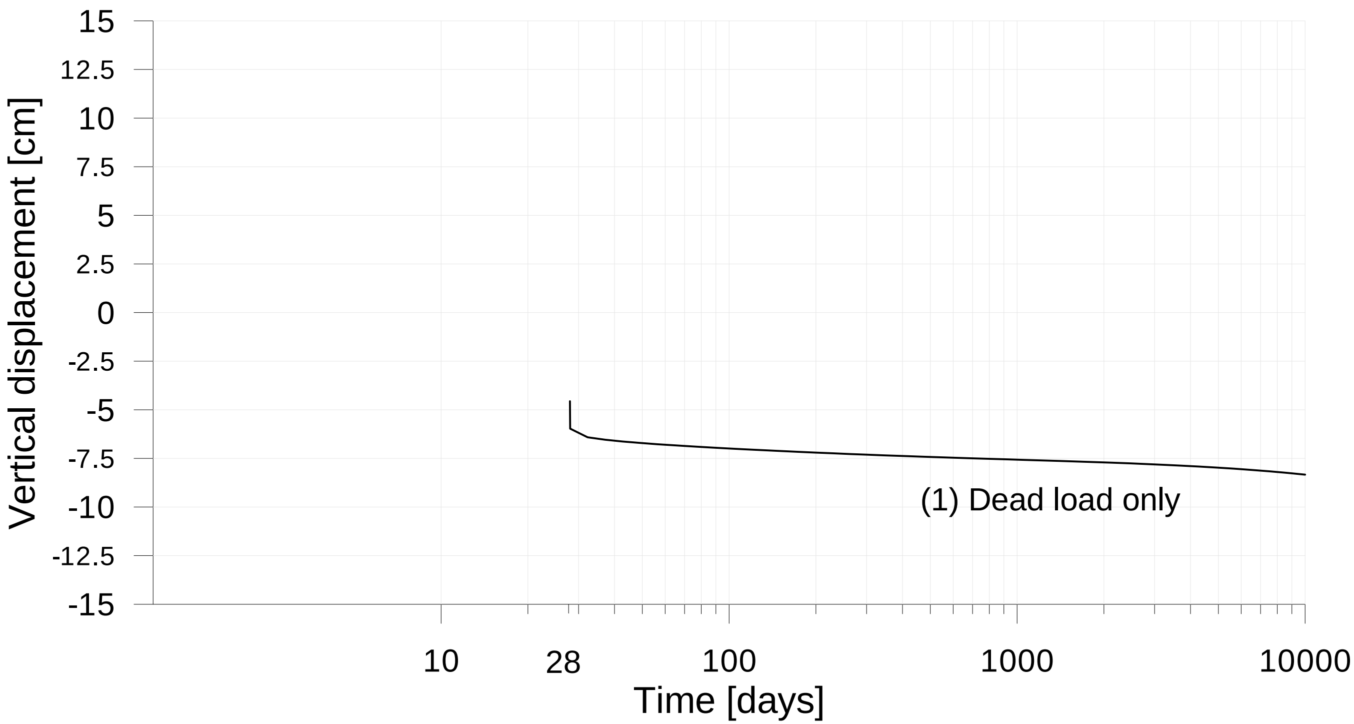
Considering the cantilever as an elastic beam, the prestressing intensity has been computed as the force which induces zero vertical displacements at the tip.

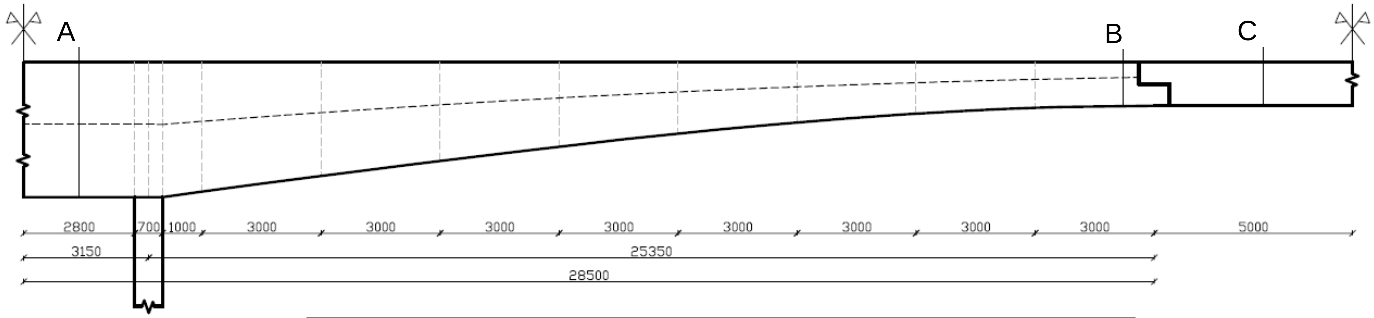
Two different ages of prestressing are considered:

1. prestressing at t_0 = 28 days;
2. prestressing at t_0 = 3.5 days;

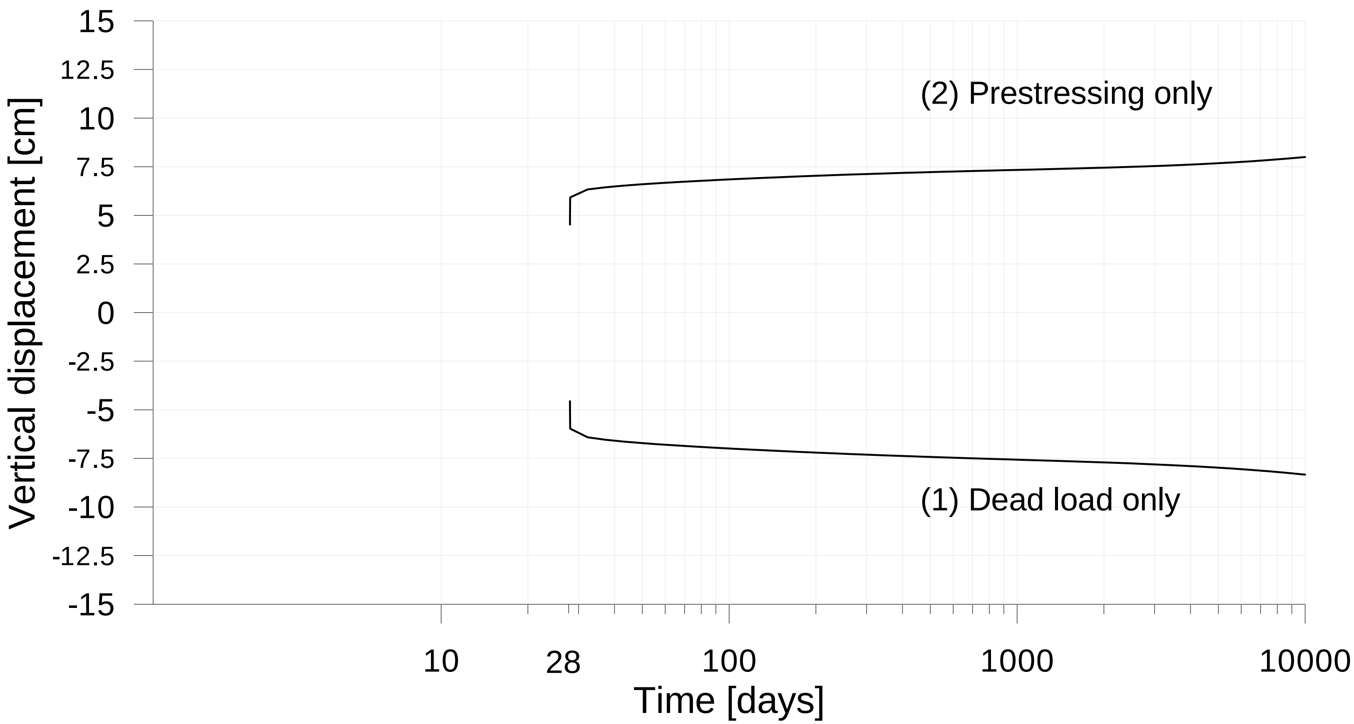


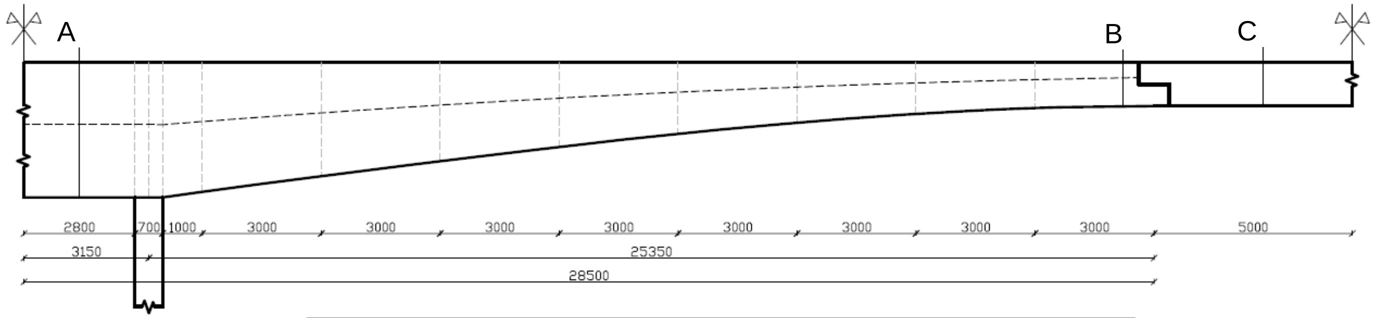
Element or parameter	μ (mean)	σ (s.d.)
Prestressing in the cables	388.6 kN	50%
f_{ck}	35 MPa	10%





Element or parameter	μ (mean)	σ (s.d.)
Prestressing in the cables	388.6 kN	50%
f_{ck}	35 MPa	10%





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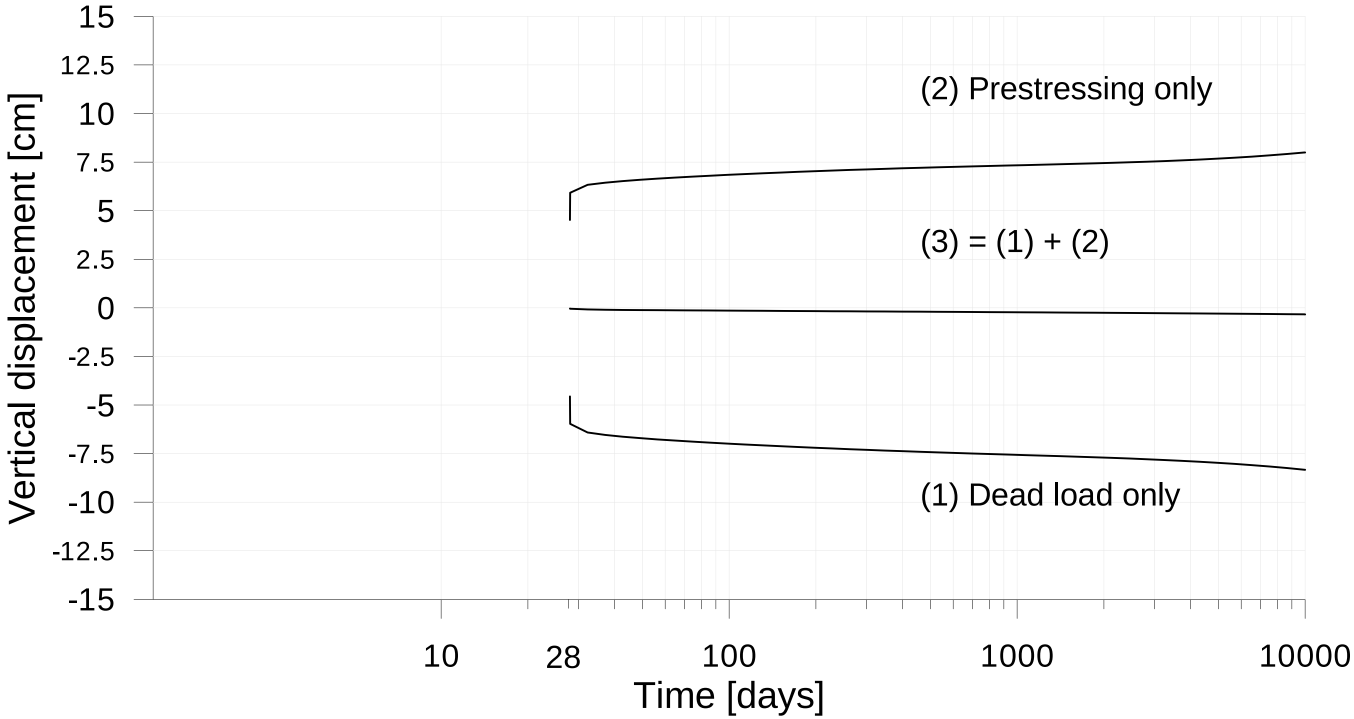

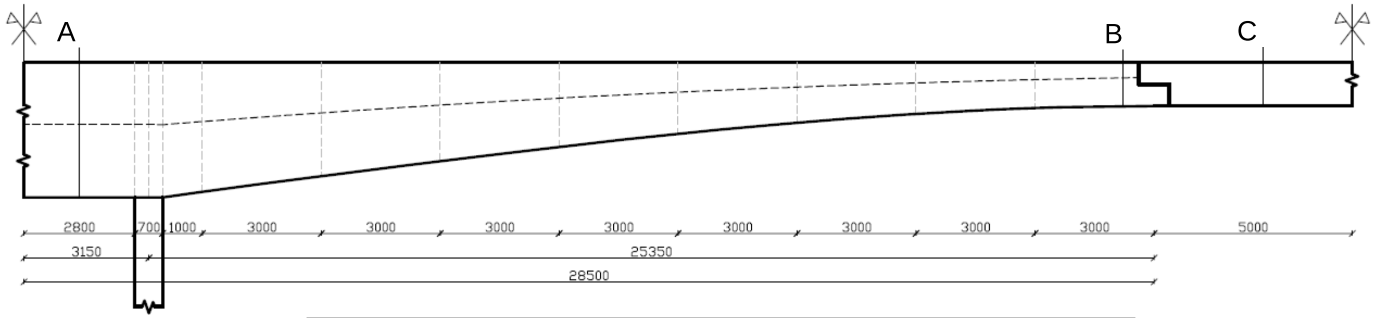

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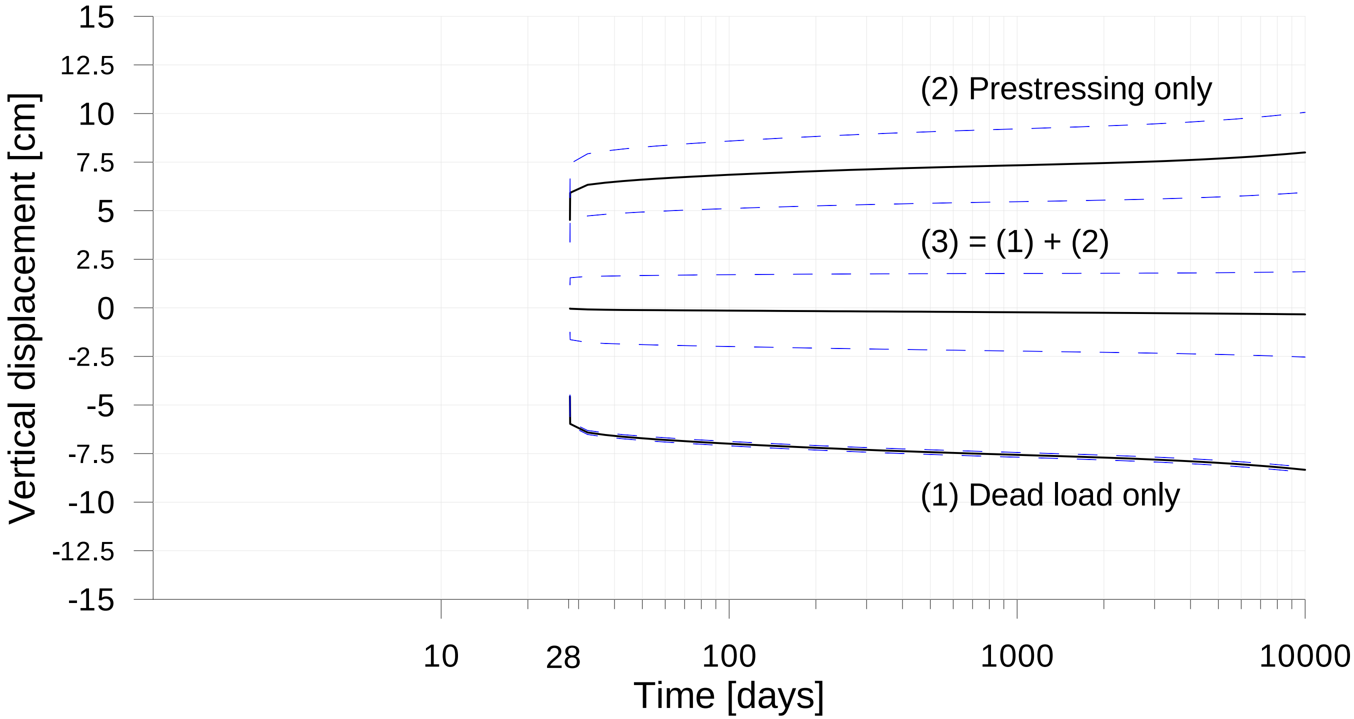

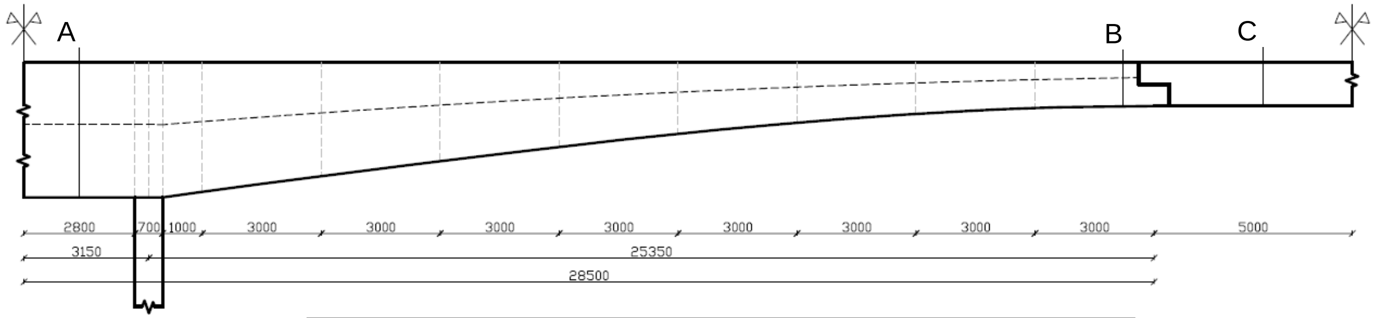

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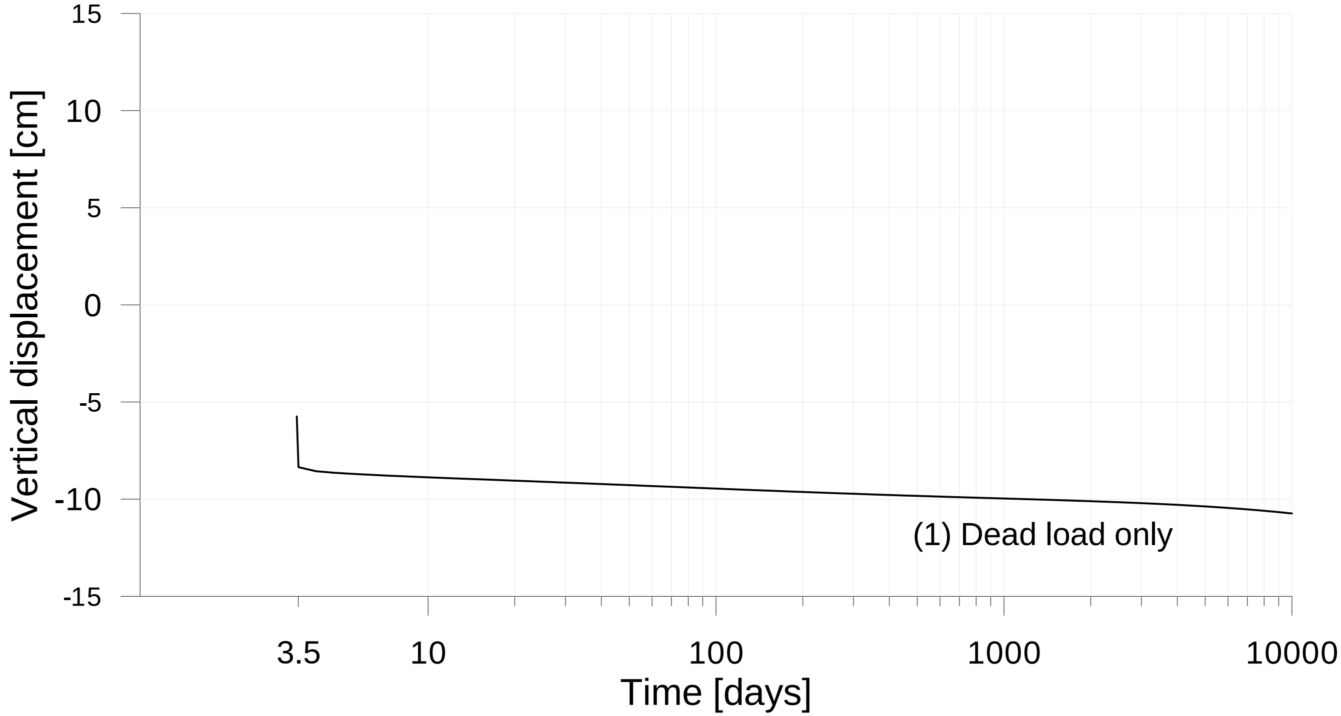

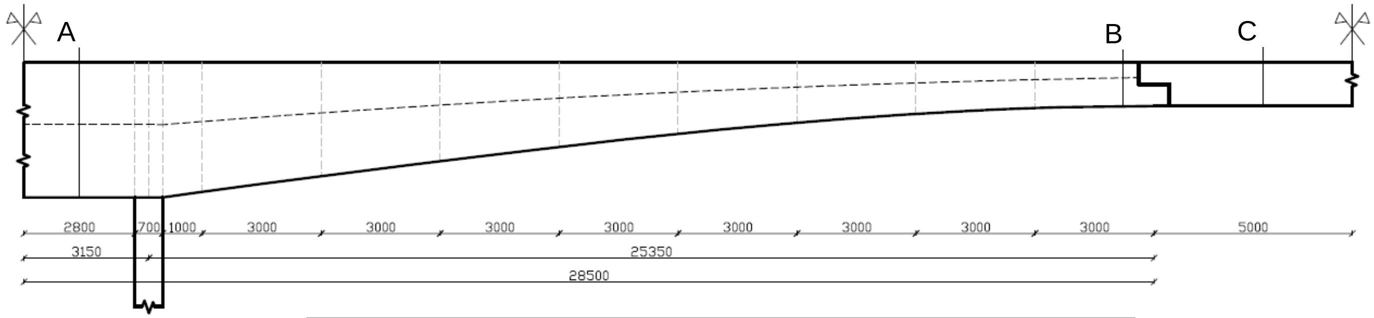

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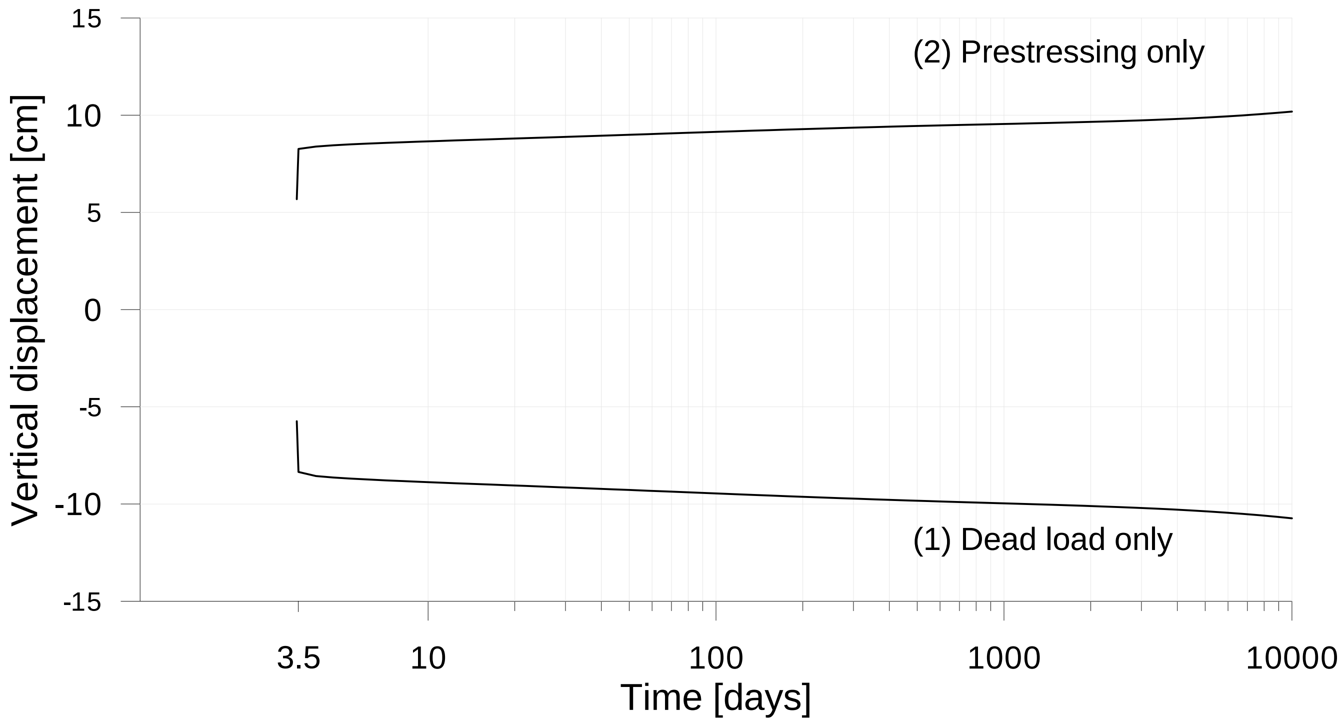

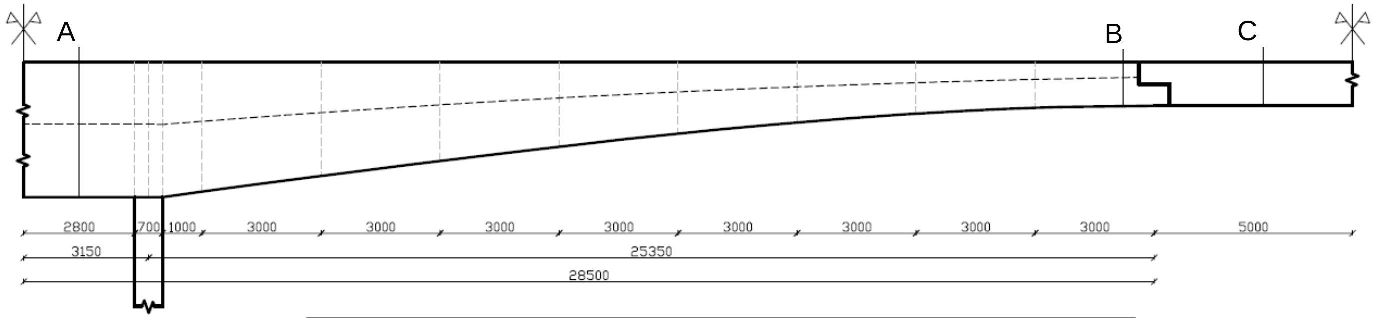

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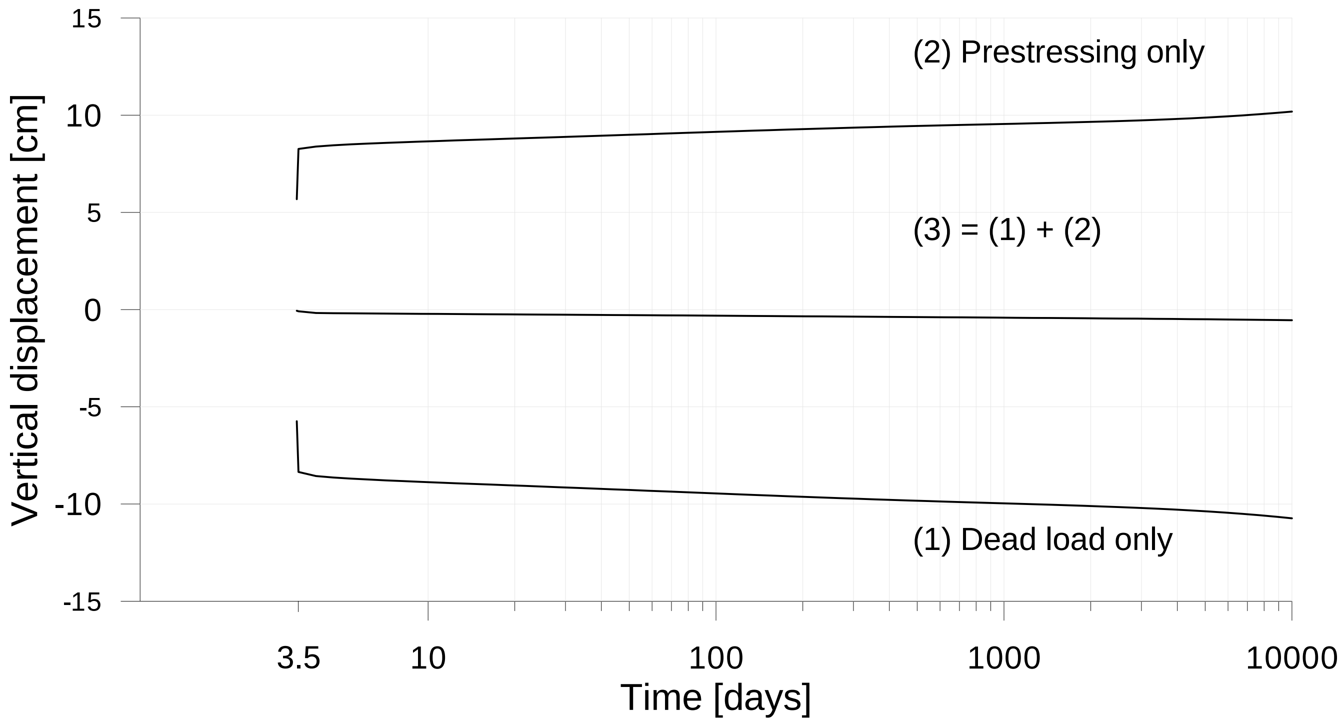

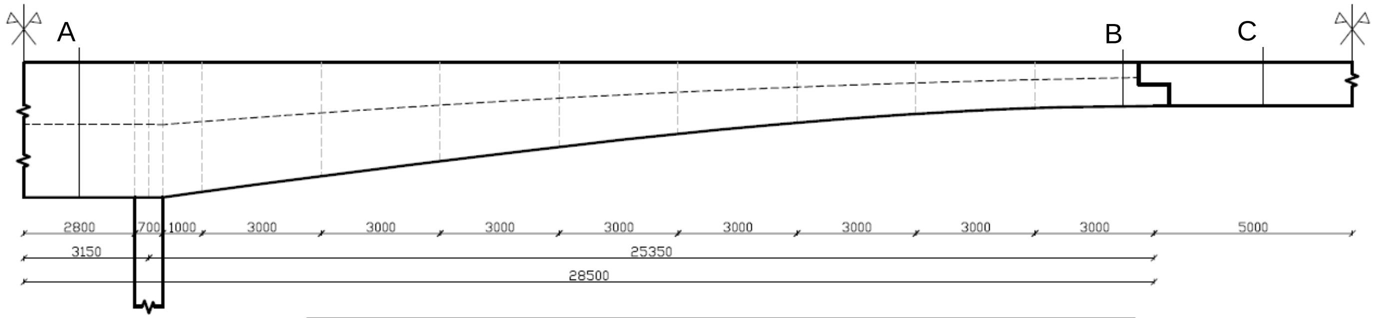

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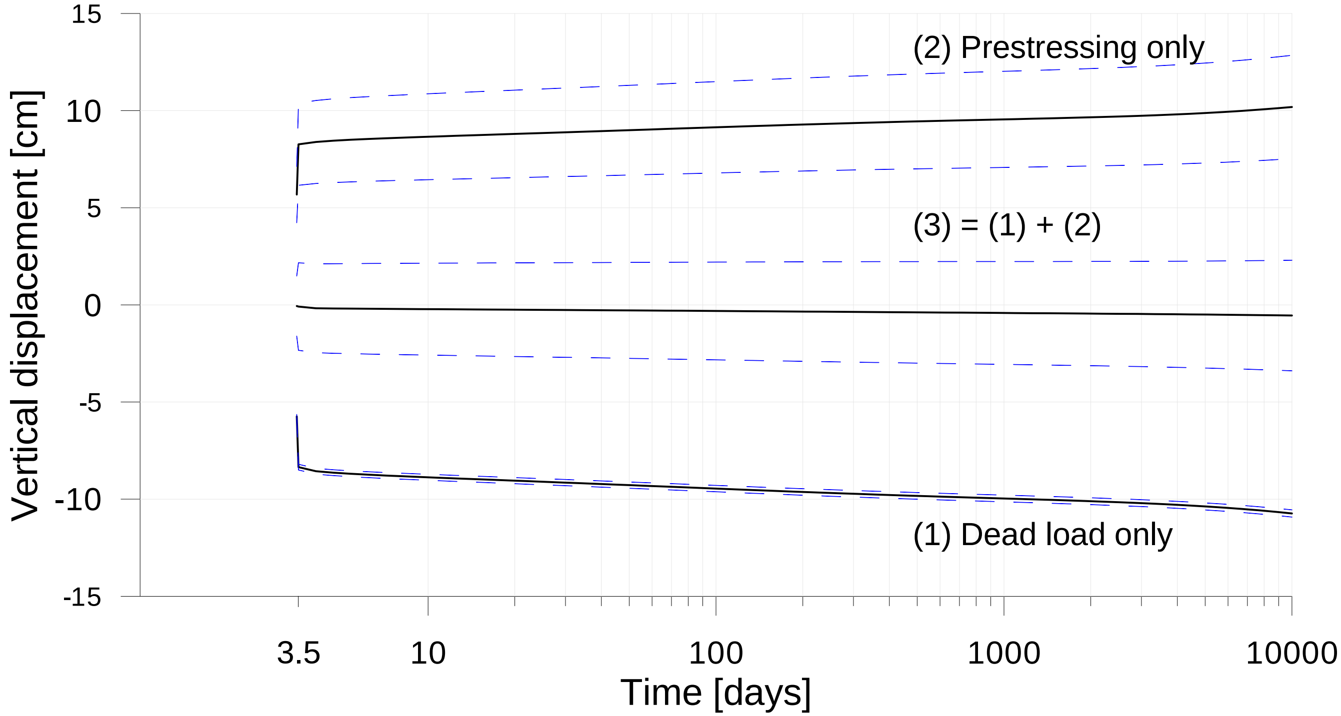
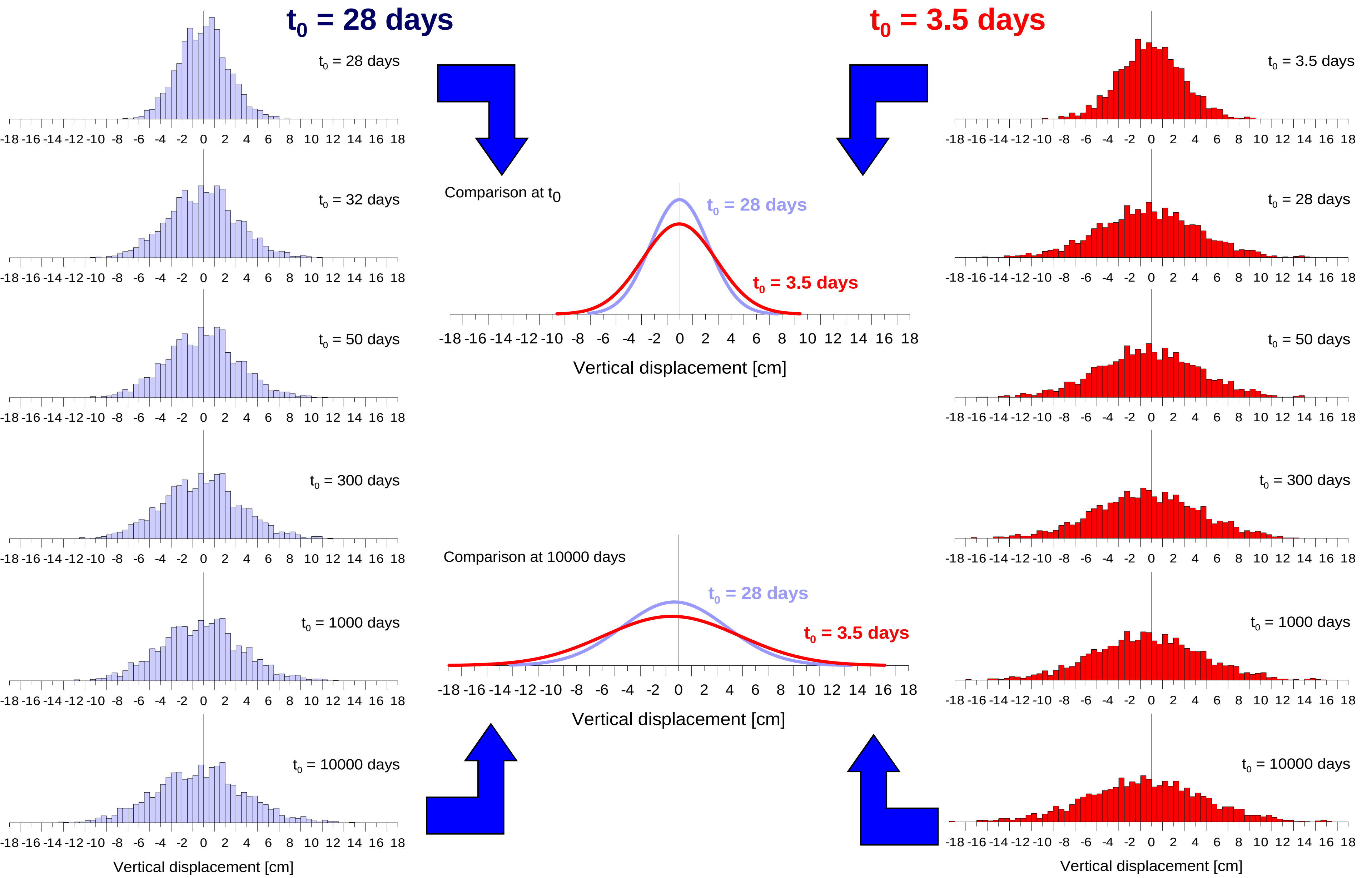



t0 = 28 days
t0 = 3.5 days
t0 = 28 days
t0 = 32 days
t0 = 50 days
t0 = 300 days
t0 = 1000 days
t0 = 10000 days
t0 = 3.5 days
t0 = 28 days
t0 = 50 days
t0 = 300 days
t0 = 1000 days
t0 = 10000 days
Comparison at t0
t0 = 28 days
t0 = 3.5 days
Comparison at 10000 days
t0 = 28 days
t0 = 3.5 days
-18 -16 -14 -12 -10 -8 -6 -4 -2 0 2 4 6 8 10 12 14 16 18
Vertical displacement [cm]

The expected attitude of long span bridges results from the **antagonist roles** of the **self-weight** and of the **prestressing.**

- Final value of the deflections = difference of two large and opposite numbers (downward and upward deflections), strongly depending on the several uncertainties involved.
- avoid **articulated** Gerber type decks. Privilege continuous structural schemes;
- search for a **tendon layout** optimizing curvature and deflections;
- prestress the structure after a **suitable curing time** and adequate strength characteristics;
- prearrange **empty ducts for additional cables leave**, to be used in contrasting unfavourable time depending effects;
- although auxiliary cables could be **activated after decades** after the building stage, it is important to design their layout according to a set of possible diverging schemes.

CONCLUSIONS

- The role of uncertainties on the time dependent behaviour has been investigated and two topical cases have been studied.
- In the case of a cable stayed bridge the uncertainties in the pretensioning forces, in the R.H. % and in the concrete strength have little influence. **The system seems to be self stabilizing over time**.
- For the case of a prestressed cantilever bridges, the uncertainties in the pretensioning forces and in the concrete strength may cause **relevant variance of the tip deflections**.
- This outline the **limits of the traditional deterministic analyses** and suggest further studies, suitable to deal with all the complexities of the actual characteristics of the real structures.

A COLLAPSE INDUCED BY SHORTENING IN A MULTISPAN VIADUCT

A COLLAPSE INDUCED BY SHORTENING IN A MULTISPAN VIADUCT

- The deck of the viaduct had a so-called “**tied-deck girder”** or “**kinematic chain”** scheme.

- A kinematic chain viaduct is **composed of a train of simply supported decks,** longitudinally connected through short and flexible continuity slabs.

- By avoiding joint devices between the spans, such scheme makes the **carriageway continuous** and **improves riding comfort.**

A COLLAPSE INDUCED BY SHORTENING IN A MULTISPAN VIADUCT

- Dealing with long span continuous or tied-deck-girder bridges, **the time dependent effects** (shortening) and temperature excursions, **cause longitudinal displacements which increase from the fixed point towards the free ends**.

- The movements are allowed by the relative displacement between the upper and lower plates of the bearing devices. The upper plate follows the superstructure in its own movements. The lower one is built in the head of the piers or the abutments.

A COLLAPSE INDUCED BY SHORTENING IN A MULTISPAN VIADUCT

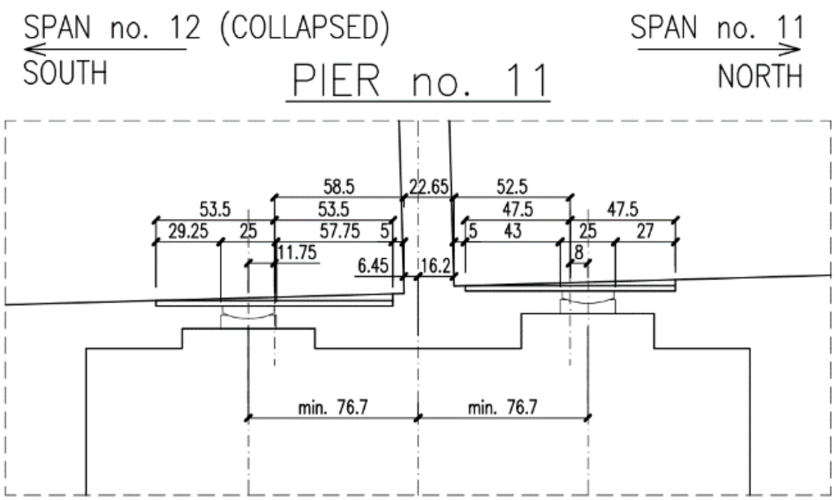
- According to the design analyses, after about 20 years, the **relative maximum displacement between the plates** at the top of pier no. 11, increased of a safety margin of 50mm, **equals 467.8 mm** (0.4678/467.5 ≅ 1/1000).

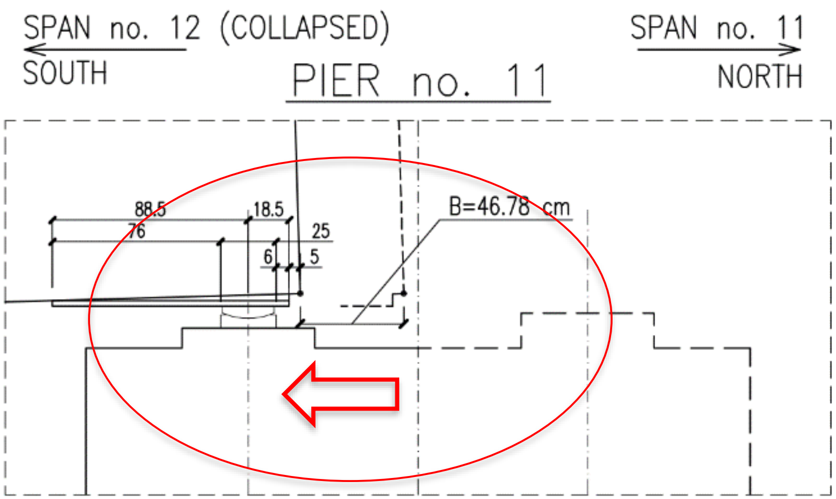
- In this configuration the upper plate still rests on the lower one.

- While the upper plate is resting on the bottom plate, **it seems impossible that the supported structure could fall from the support.**

- Hence the first evaluation seems reliable.

INITIAL AND FINAL POSITION OF THE EDGE OF THE BEAM



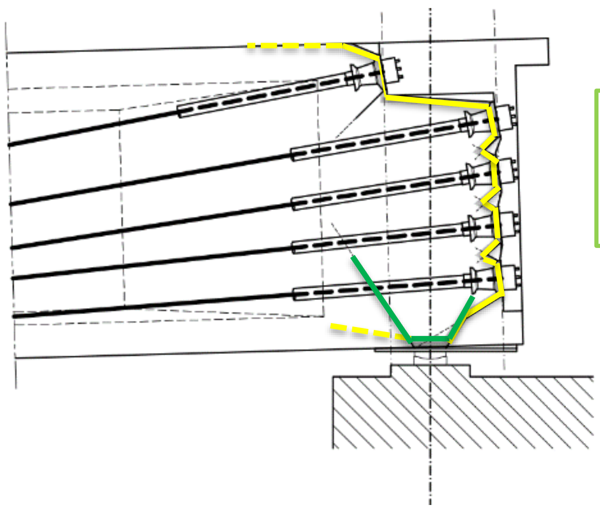


Shortening (Theoretical): 467.8 mm

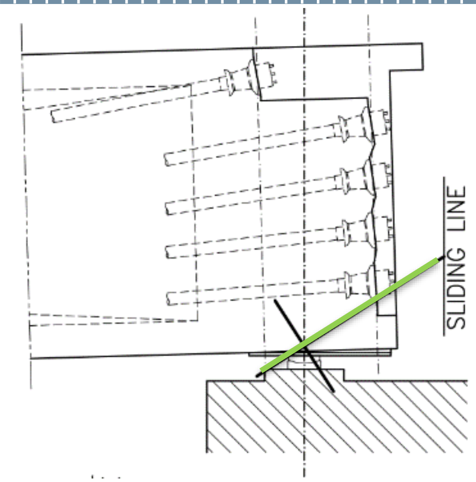
(a) **(b)**

Distance of the edge of the beams from the axis of the bearing supports:
(a) in the design configuration; (b) after 20 years.

COMPRESSION FIELD DUE TO PRESTR. FORCES AND VERTICAL REACTION

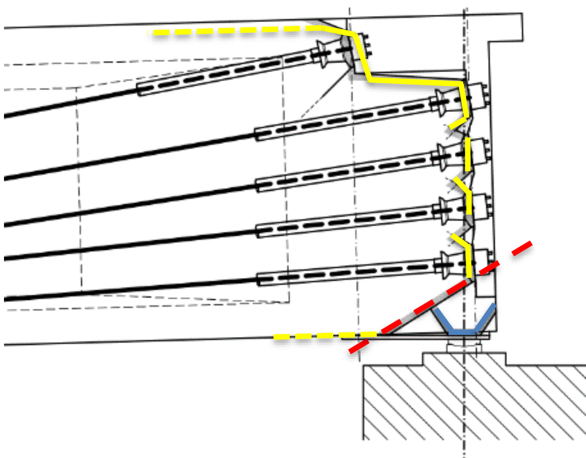


Design (initial) configuration.

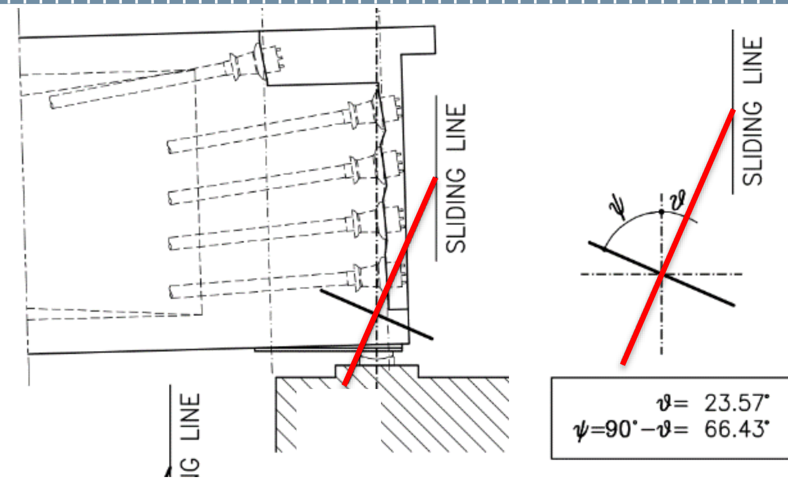


- Yellow contours outline the diffusion zones of prestressing forces and support reaction.
- The contour envelope fully compressed zones.
- **The diffusion angle of the vertical reaction falls fully inside the biaxially compressed head of the beam.** Dead zones are not interested by the set of forces which control the beam static (Guyon 1960).
- Local reinforcement behind the head of the anchorages and above the support plate, provide a suitable confinement action.

COMPRESSION FIELD DUE TO PRESTR. FORCES AND VERTICAL REACTION



After the shortening



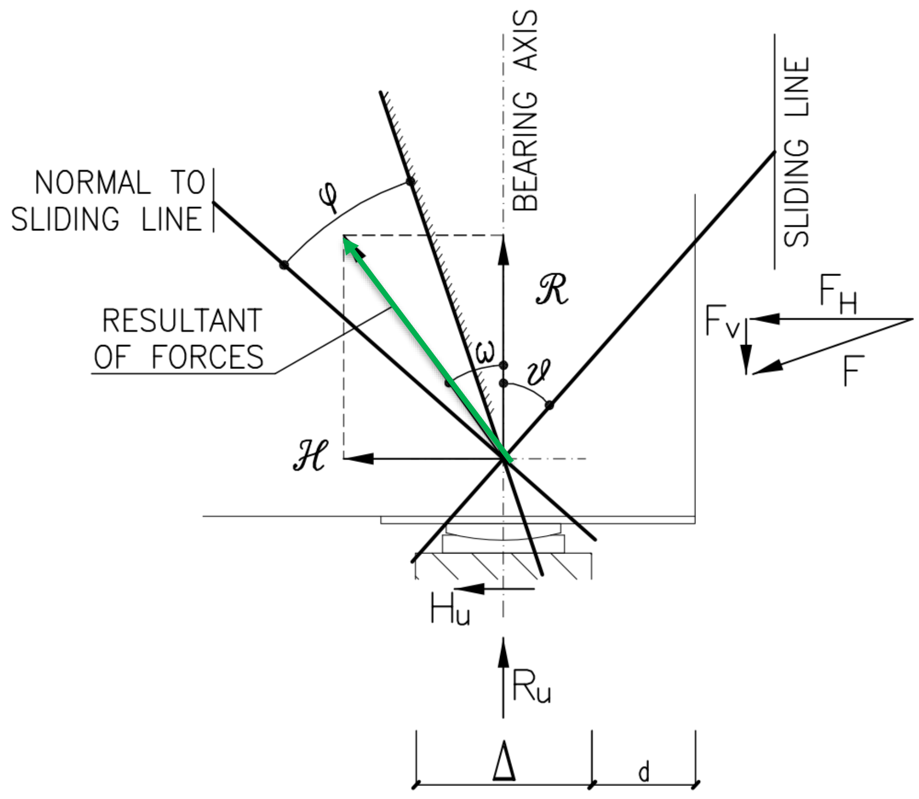
- Compression stress diffusion and corresponding sliding lines after the shortening of the tie-deck-slab viaduct.
- Yellow contours outline the diffusion zones at the maximum displacement.
- **If the support reaction is applied near the bottom corner of the head of the prestressed beam, a split failure may start.**
- The reinforcement crossing the potential failure line should sustain the forces acting in that zone.

FAILURE MECHANIMS Forces governing the equilibrium of the corner.

With reference to Figure the equilibrium equations at sliding line are are:

$$R = R_u + F_V$$

$$H = -H_u + F_H + A_e \cdot \sigma_s$$

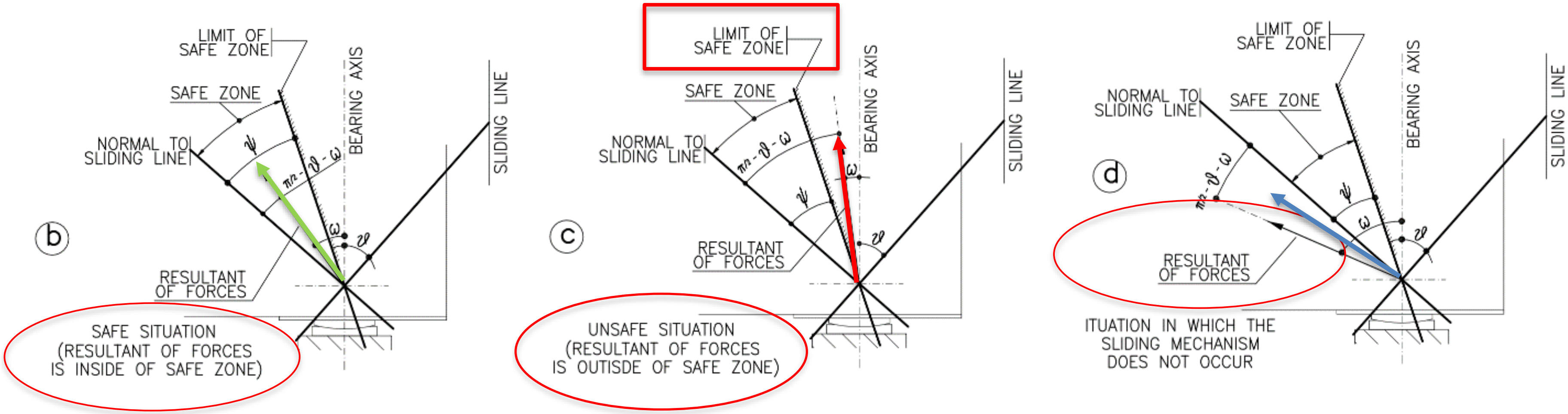


Let φ be the friction angle and ω the angle of the force resultant with respect to the vertical axis. The equilibrium is satisfied if the following inequality holds:

$$(\pi/2 - \vartheta) - \omega \leq \varphi$$

CHECK FOR EQUILIBRIUM

The check is satisfied if the resultant of the forces, below the sliding line, falls inside an angle having the opening equals to the internal friction angle of concrete (Thonier 1985, Chaussin et al. 1992, BPEL.)



Equilibrium

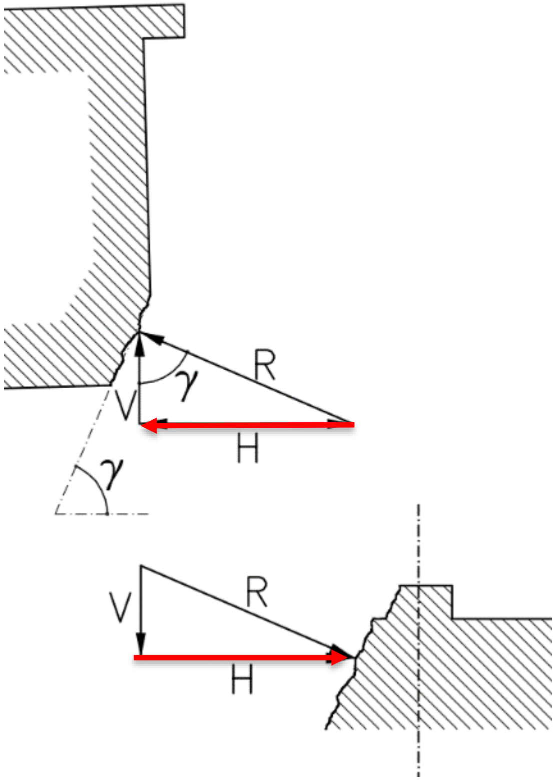
No Equilibrium

Sliding mechanism does not occur

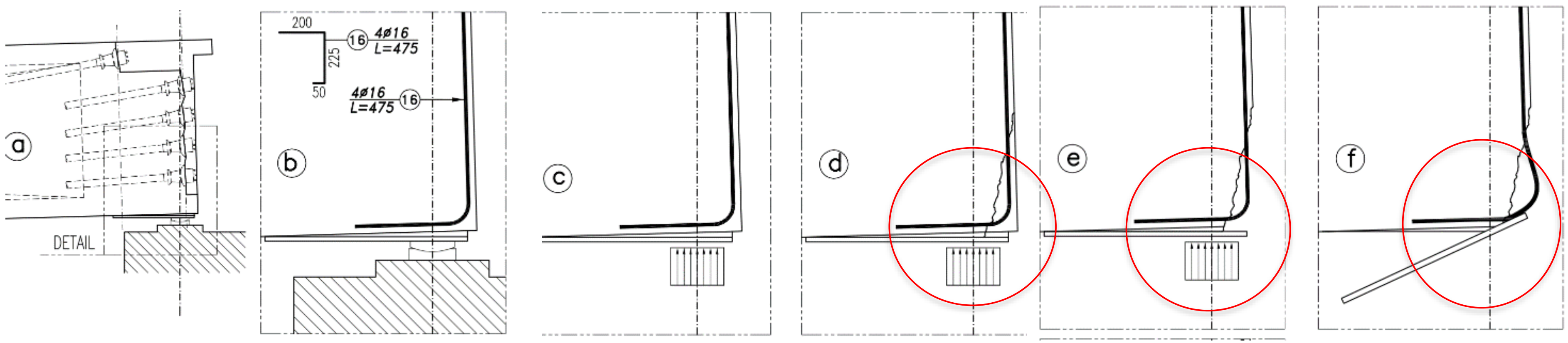
Same conclusion also by taking into account the reinforcement contribution - French Norms BPEL

ORIGIN OF THE HORIZ. FORCE ACTING AT THE TOP OF THE PIER

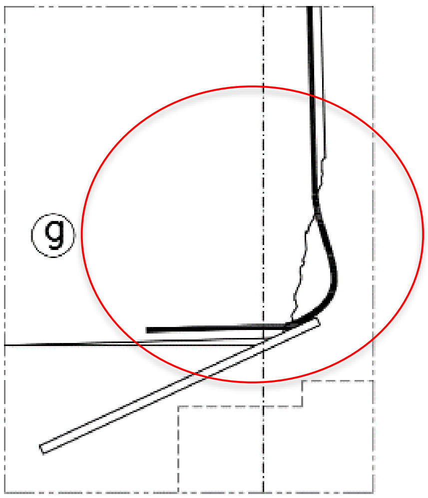
- The local failure, caused by the splitting of the wedge, causes **the beam** to be **supported by a sloping surface.**
- Moreover, the **horizontal component of the support reaction force, together with the increase of the eccentricity of the reaction support** of the beam still laying on the pile cause an abnormal deflection of the pile and, at the end, the collapse of the beam.
- **This explain also the huge curvature of the pier**.



ROLE OF THE UPPER PLATE OF THE BEARING DEVICE



a) situation at the maximum displacement;
b) local detail of the reinforcement;
c) local pressure distribution;
d) spalling of the concrete cover;
e) contact surface reduction due to spalling;
f) buckling of the bars;
g) rotation of the upper plate;
h) final situation.



THE LESSON

A – About the beam-end zones

- In tie-deck slabs as well as in continuous viaducts, the bearing support zones must be carefully designed.
- The magnitude of the relative displacements**, the changes in the position of the contact zone and the evolution of local diffusion mechanisms have to be examined for a broad set of possible situations.**
- During the service life of viaducts, time dependent phenomena must be monitored as they develop their effects during a long time span. This allows detecting the evolution of the shortening at the heads of all piles.
- Thorough **periodical inspections** allow to establish a sound correlation among time, displacements and support positions and the adoption of suitable maintenance measures.

B - About the whole viaduct design

- In order to standardize the construction process, the same scaffolding and formwork devices are usually used for all piers, independently of the displacement range associated to the bearing supports.
- This means that **the heads of the piers far from the null point have the same longitudinal depth of those close to it.**
- This causes a **progressive reduction of the safety margins** at the top of the pile heads bearing the more external supports, on which larger displacements occur.
- A good practice suggests that **the geometry of different pile heads must differ according to their positions, allowing for an enlargement of the longitudinal dimension as needed.**

CONCLUSIONS

DESIGN SUGGESTIONS - 2

B- About the whole viaduct design

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CONCLUSION . . . OF THE CONCLUSIONS

CONCEPT . . .

Se proviamo a pensare le equazioni della Scienza delle Costruzoni contenenti le tensioni come dati noti e come incognite le grandezze che descrivono la geometria della struttura, compiano un passo verso un nuovo modo di utilizzare il nostro patrimonio scientifico

(Sergio Musmeci, Ricercatore e Ingegnere, 1926-1981).

CONCEPT . . .

If, in the equations of mechanics we assume the stresses as given data and geometry of the structure as unknown, we are moving towards a new way to exploit our scientific heritage.

(Sergio Musmeci, Italian scholar and designer, 1926-1981).

CONCLUSION . . . OF THE CONCLUSIONS

DETAILS

. . . two points of view:

God is in the detail (attributed to Ludwig Mies van der Rohe, 1886–1969).

Whatever one does should be done thoroughly.

The devil is in the detail (attributed to Henry Petroski)

Catch the potentially hazardous elements hidden in the details.

ACKNOWLEDGEMENTS

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CONCEPTUAL DESIGN: FROM ABSTRACT REASONING TO CONSISTENT DETAILS

THANK YOU FOR YOUR KIND ATTENTION

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