

**Accordo tra il CSLP ed il Consorzio ReLUIS
attuativo dei DM 578/2020 e DM 204/2022**

Attività di formazione per i tecnici degli Enti Locali

Modulo II - Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

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**Napoli
25 settembre 2024**

Lezione 4 Analisi di casi di crolli di ponti

G. Michele Calvi

Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti

Experiences and trends in seismic assessment and strengthening of bridges

G. Michele Calvi

Seismic design and assessment of bridges

- Performances
- Associated cost / loss
- Accepted probability of occurrence
- Average annual loss

Design:

- Define probability of occurrence of ground shakings
- Define performances and related design parameters
- Design to attain design parameters at corresponding ground shaking

Analysis / Verification:

- Define performances
- Calculate probability of occurrence of ground shakings corresponding to performances
- Evaluate corresponding costs/losses and annual expected average

Typical Performance Requirements for Bridges

Serviceability limit state (Level 1) no elements suffer notable damage and the bridge continues to function normally following the earthquake without any need for repairs.

Repairable damage (Level 2) inelastic response may develop in pre-selected and adequately detailed plastic hinge zones. The plastic mechanism should be carefully selected such that eventual repairs do not require closure of the bridge. Cover concrete may spall and longitudinal reinforcement may yield but strains are limited to moderate values (without risk of re-bar buckling). Joints in the deck may be damaged but must remain traversable by emergency services and should be easily repaired.

Collapse prevention (Level 3) significant damage may develop in plastic hinge zones, and minor localised damage may occur in other parts of the bridge. The damage may require closure of the bridge to repair, but collapse of the bridge is avoided. Significant concrete spalling may develop and longitudinal re-bar strains may be close to the onset of re-bar buckling. Joints in the deck may be damaged but should remain traversable by emergency services.

Serviceability

... no elements suffer notable damage and the bridge continues to function normally following the earthquake ...

No need for repairs?

Continued service?

Repairable damage

... inelastic response may develop in pre-selected and adequately detailed plastic hinge zones ...

Cost of repairs?

Limitation of use?

Temporary closure?

Collapse prevention

... significant damage may develop in plastic hinge zones, and minor localised damage may occur in other parts ...

Cost of repairs?

Cost of closure?

Definition of the ground motion intensity for a site is an important part of the PBD process.

Structures of Low Importance →

Very Important Structures →

Importance Class*	Earthquake Design Intensity		
	Level 1	Level 2	Level 3
I	Not Required	50% in 50 years	10% in 50 years
II	50% in 50 years	10% in 50 years	2% in 50 years
III	20% in 50 years	4% in 50 years	1% in 50 years
IV	10% in 50 years	2% in 50 years	1% in 50 years

Relevant Bridges

**refer EC8.1 T4.3 for importance class definitions*

Typical Performance Indicators

Material	Level 1	Level 2	Level 3
Concrete comp. strain	0.004	Eq.	1.5xEq.
Rebar tension strain	0.015	0.6 ϵ_{su}	0.9 ϵ_{su}
Structural Steel Strain	0.010	0.025	0.040
Masonry comp. strain	0.003	Eq.	1.5xEq.
Timber tension strain	0.75 ϵ_y	0.75 ϵ_y	0.75 ϵ_y

Strain limits for different intensity levels

A glimpse into building design, assessment and strengthening:

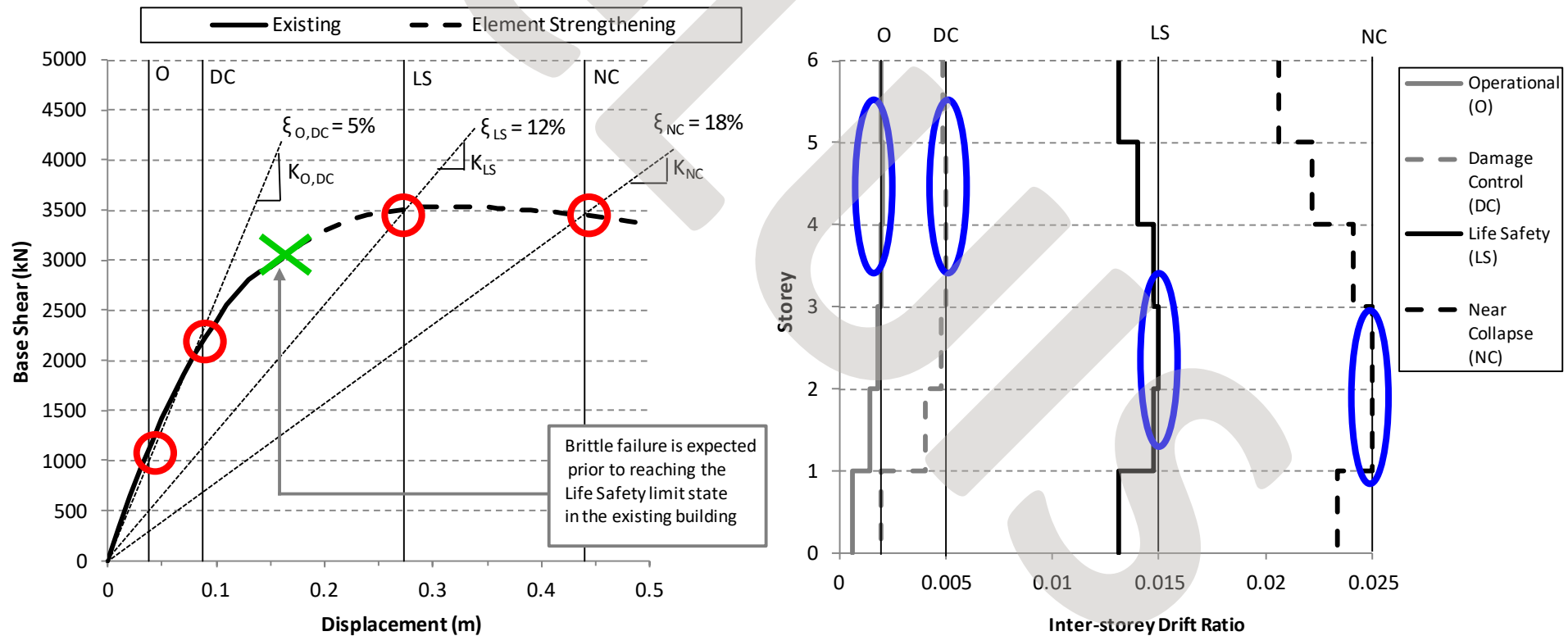
How can different solutions be compared and choices made?

Keynote lecture at the 15 WCEE, Lisbon, 2012
Revised version published by the Journal of Structural Engineering

EVALUATION OF EXPECTED ANNUAL LOSS

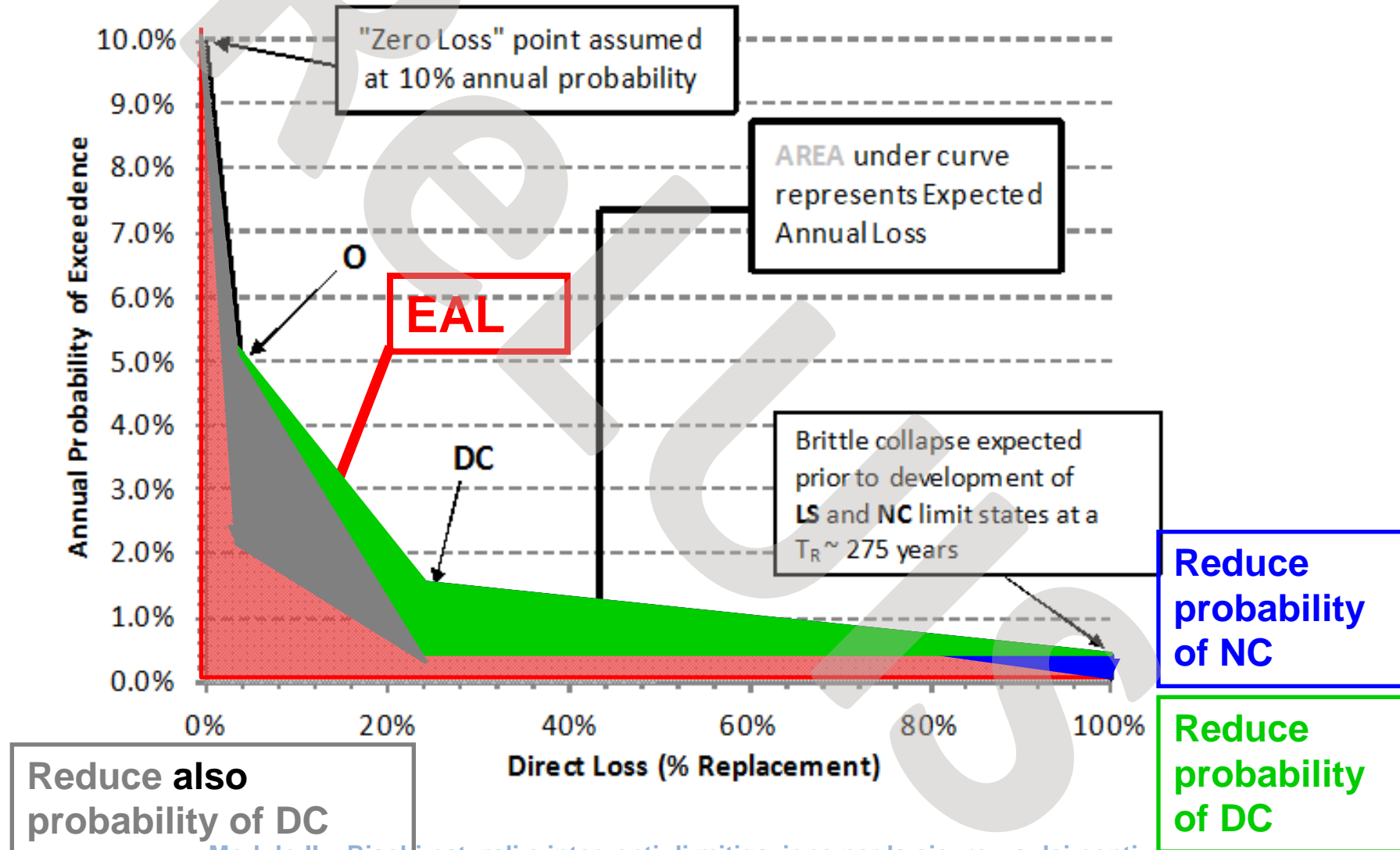
constant reference to a six storey reinforced concrete residential building damaged by the L'Aquila earthquake and later repaired and strengthened

Response assessment



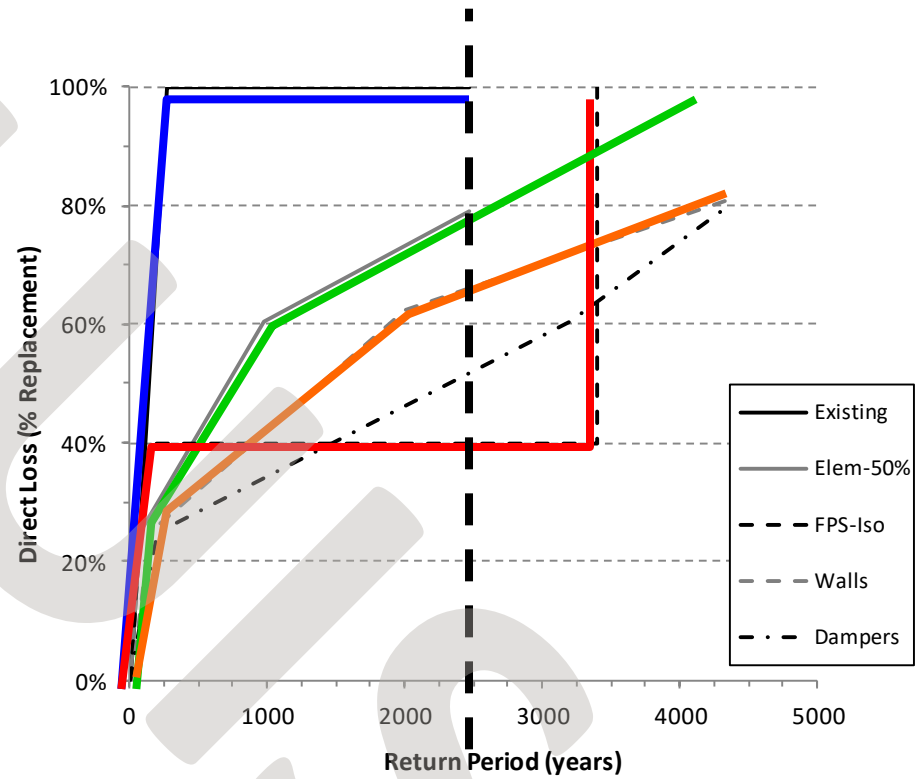
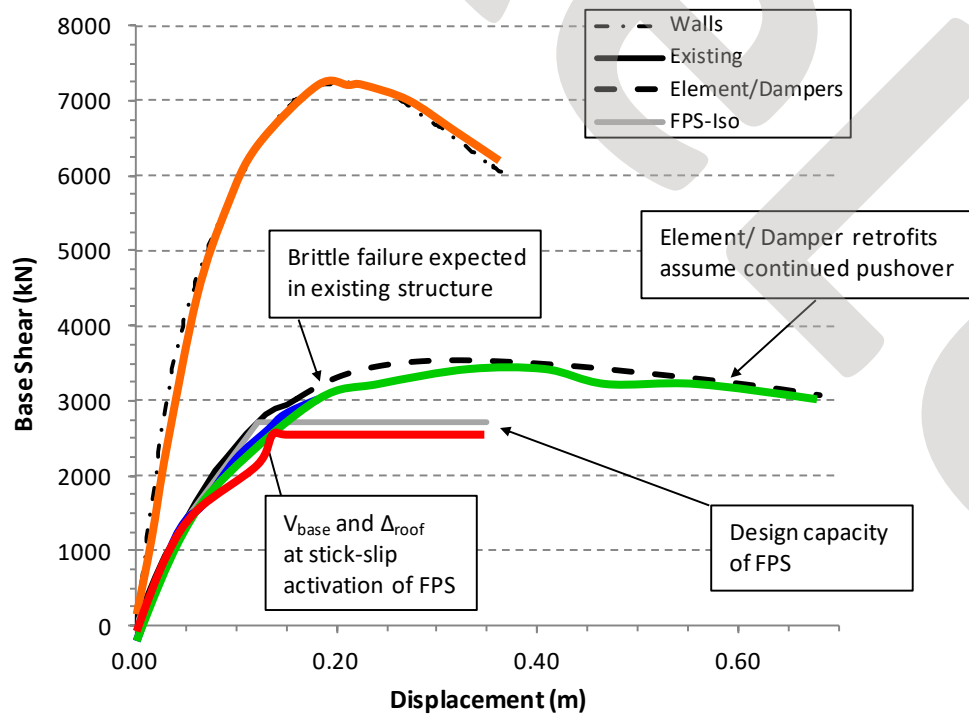
EAL – Expected Annual Loss

Loss assessment



Cost – benefit analysis

Measures for strengthening
Isolation system



Cost – benefit analysis

Indirect losses – residential case

Assume that each person occupies a building area of 25 m²

Assume that cost (to society or to an insurance company) to relocate is 50 €/day

Value of the area occupied by a person: 30,000 € (1,200 [€/m²] × 25 [m²])

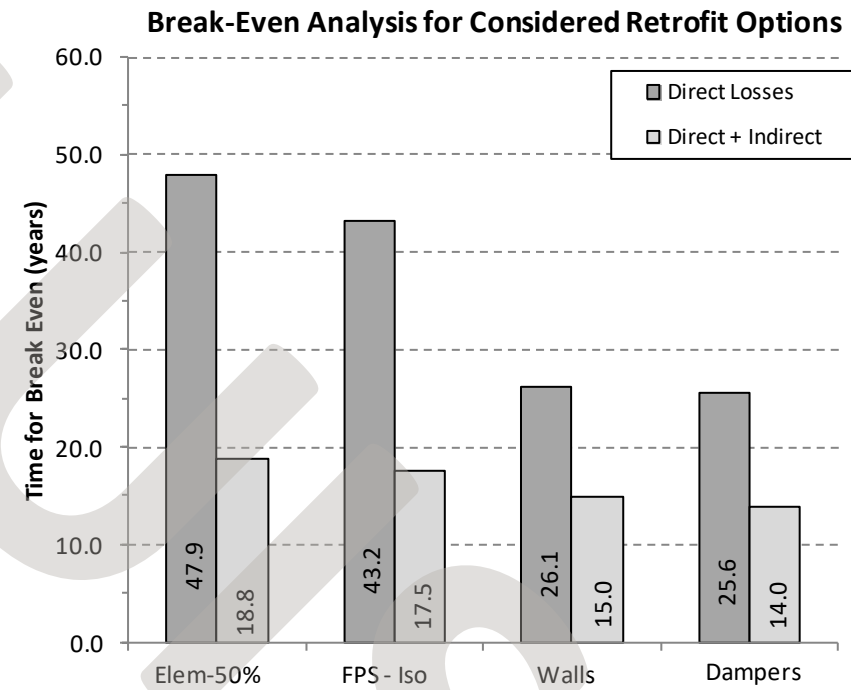
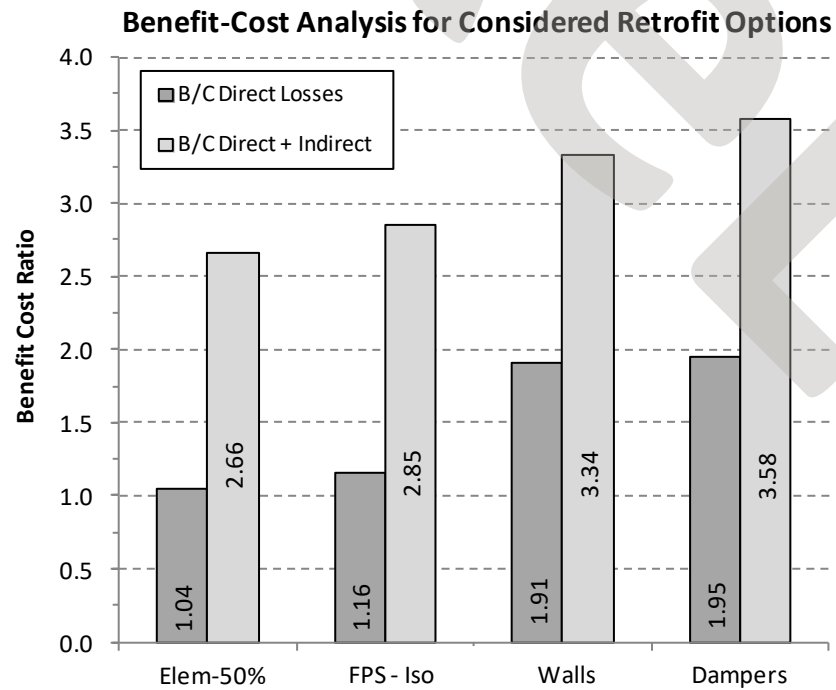
Indirect loss due to downtime 50 [€/day] / 30,000 [€]

➔ 0.17 % of RC per day

Performance	Interruption	Time (days)	Loss _{indirect} (% of RC)
Fully Operational	1 week	7	1.2 %
Damage Control	4 weeks	30	5.0%
Life Safety	6 months	180	30.0%
Near Collapse	1.5 years	540	90.0%

Cost – benefit analysis

Breakeven point



How to define direct and indirect losses for bridges?

Cost of enhanced design.

Cost of repair.

Cost of reduced or interrupted use.

Correlation with maintenance.

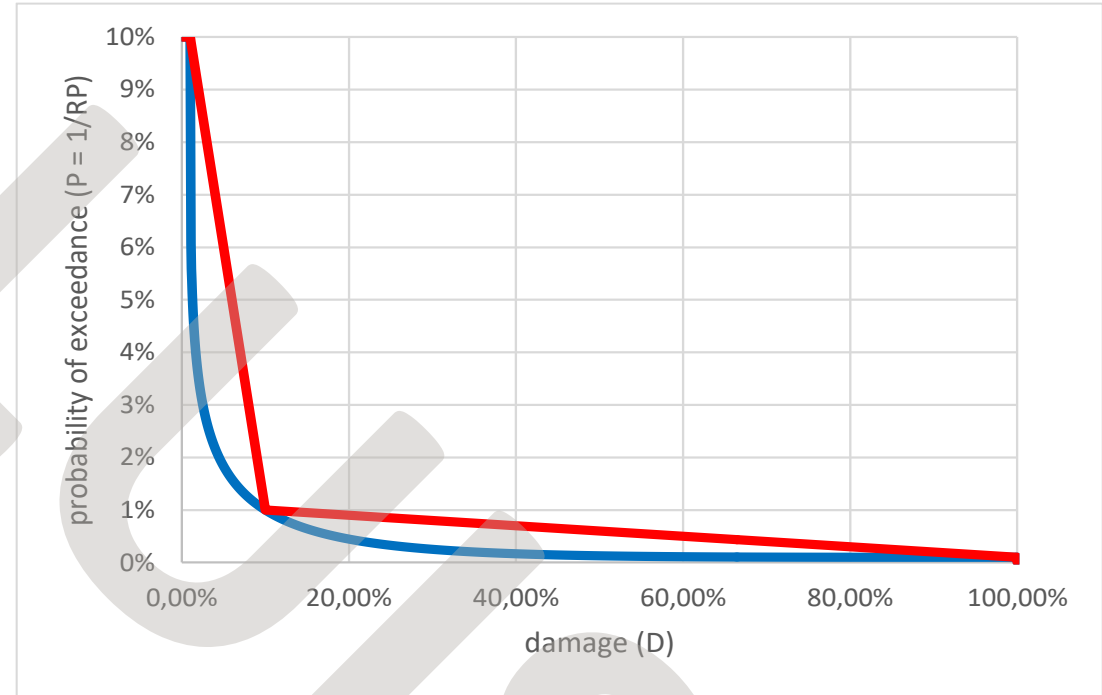
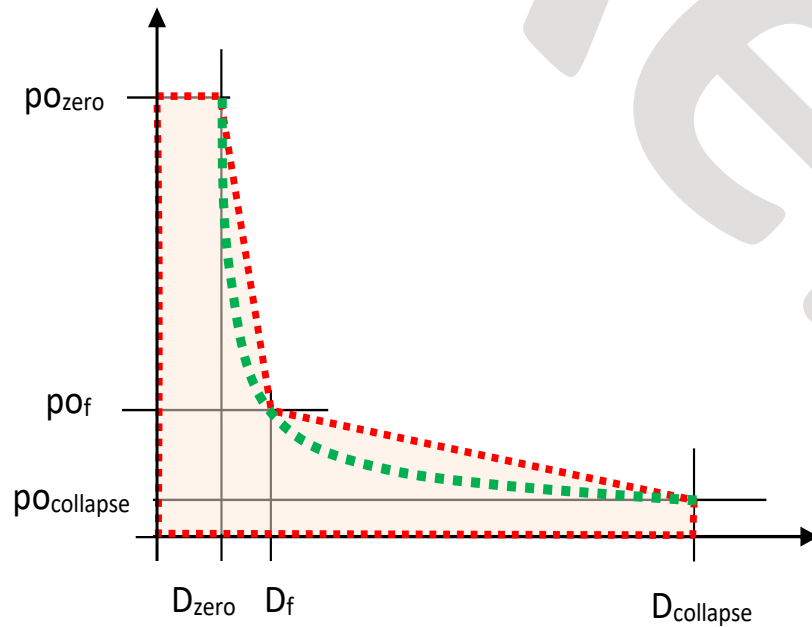
Definition of breakeven time.

Main objective of seismic design:

**Minimize total investment,
including construction and total losses
during the structure life**

EAL (expected annual loss) = $\int (po \times D)dD$

as a tool to design

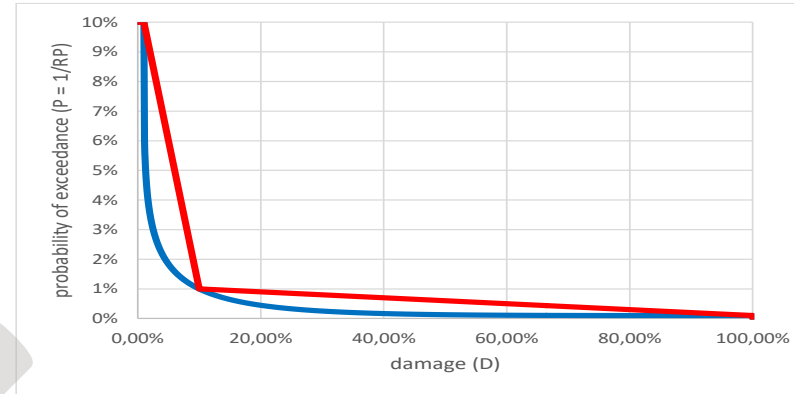


po = probability of occurrence

D = damage level

EAL as a tool to design

Derive an equation for the blue curve



$$P = P_{collapse} + (P_{zerodamage} - P_{collapse}) \cdot \sin^{\alpha} \left(\cos^{-1} \left(\frac{D - D_{zero}}{D_{collapse} - D_{zero}} \right)^{\frac{1}{\alpha}} \right)$$

forced to pass through the two extreme points and governed by the single parameter α to pass through the **f** point.

E.G.:

$$D_{collapse} = 100\%$$

$$D_{zero} = 1\%$$

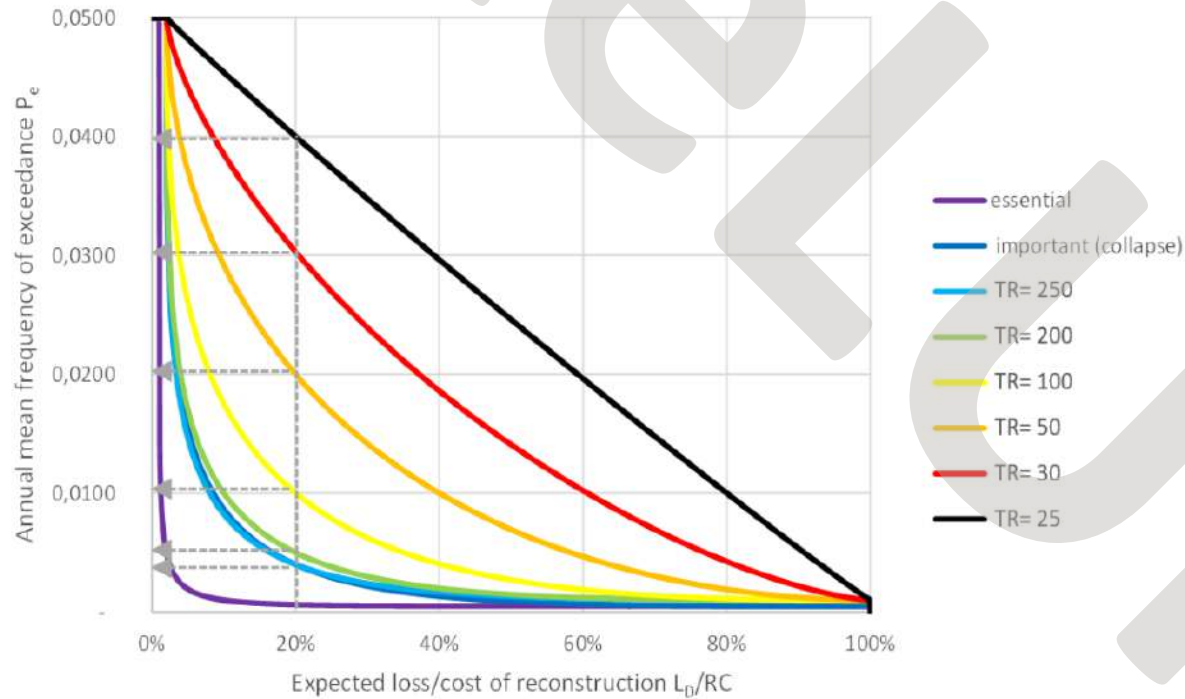
$$D_f = 10\%$$

$$P_{collapse} = 1/1000$$

$$P_{zerodamage} = 1/10$$

$$P_{zerodamage} = 1/100$$

$$P_e = P_{ec} + (P_{e0} - P_{ec}) \cdot \sin^\alpha \left(\cos^{-1} \left(\frac{L_D - L_{D0}}{L_{DC} - L_{D0}} \right)^{\frac{1}{\alpha}} \right)$$



$L_{D0} = 2\%$

$L_{DC} = 100\%$

$P_{e0} = 5\%$ $T_{RC} = 20$ years

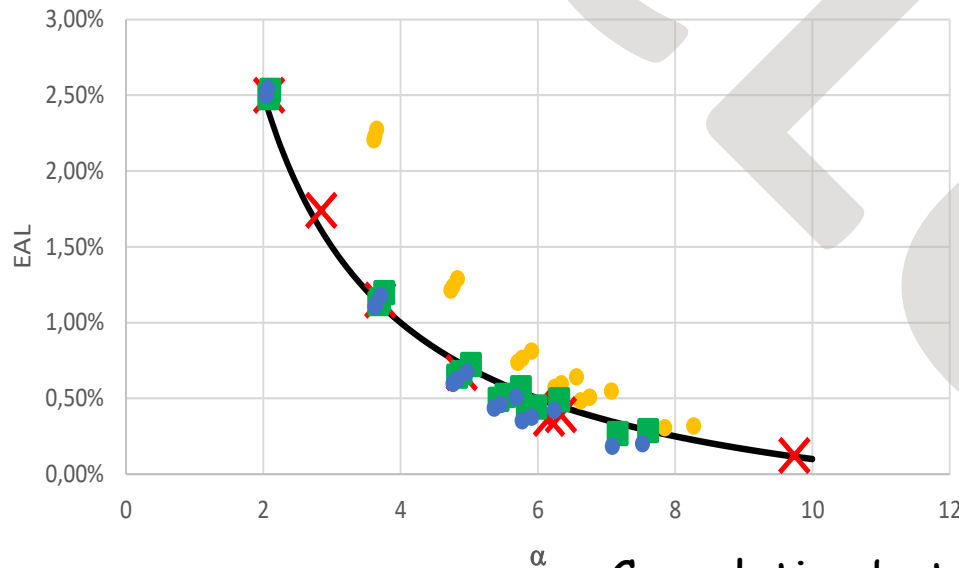
$P_{ec} = 0.05\%$ $T_{RC} = 2000$ years

$L_{Df} = 20\%$

	$\alpha=9.7$	$\alpha=6.2$	$\alpha=6.3$	$\alpha=5.9$	$\alpha=4.9$	$\alpha=3.7$	$\alpha=2.8$	$\alpha=2.1$
EAL	0.12%	0.37%	0.39%	0.44%	0.67%	1.15%	1.74%	2.50%

$$\alpha = \frac{6\%}{EAL + 0.5\%}$$

$$EAL = \frac{6\%}{\alpha} - 0.5\%$$

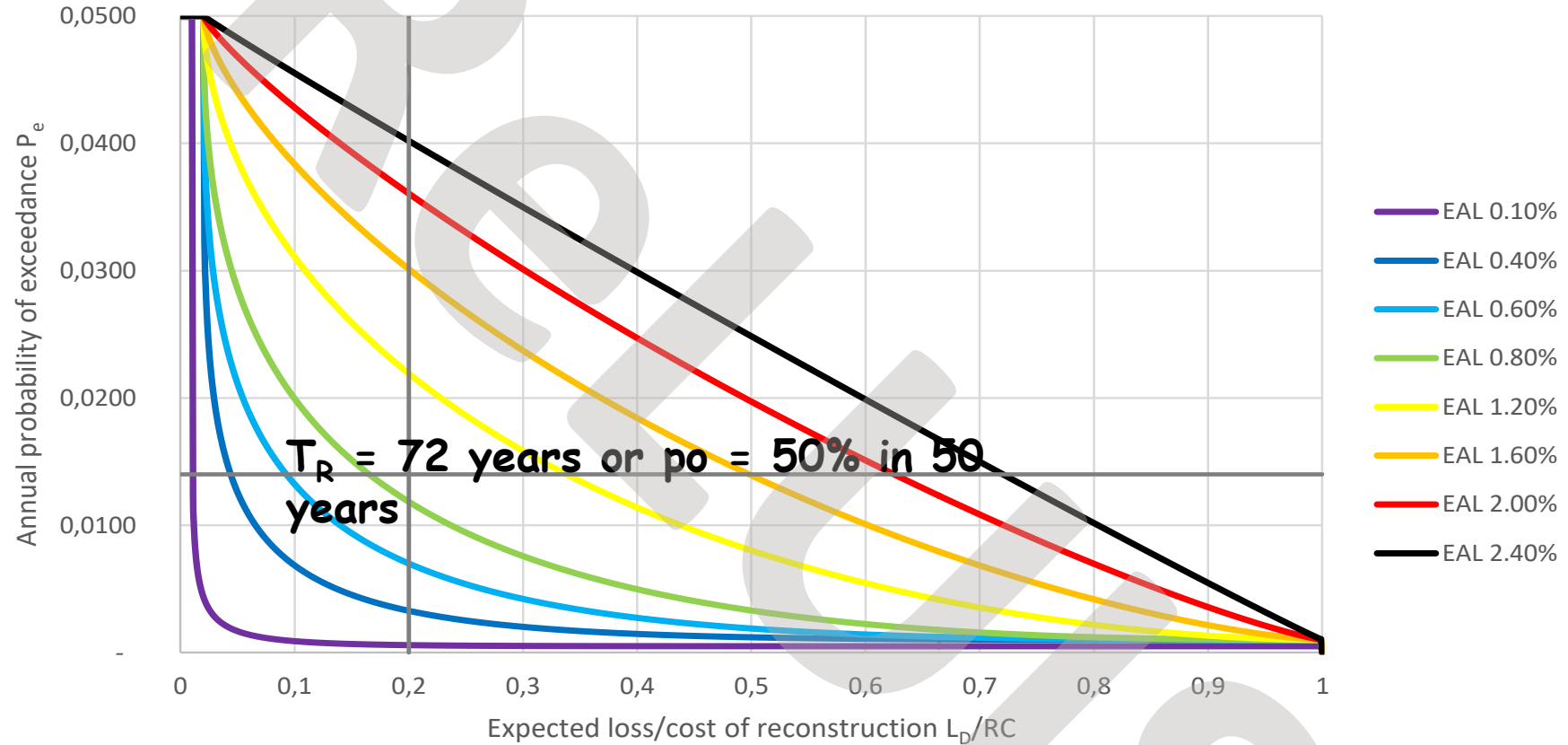


yellow dots: 1% loss
at $T_{RO} = 10$ years

green squares: 2% loss
at $T_{RO} = 20$ years

blue dots: 1% loss
at $T_{RO} = 20$ years

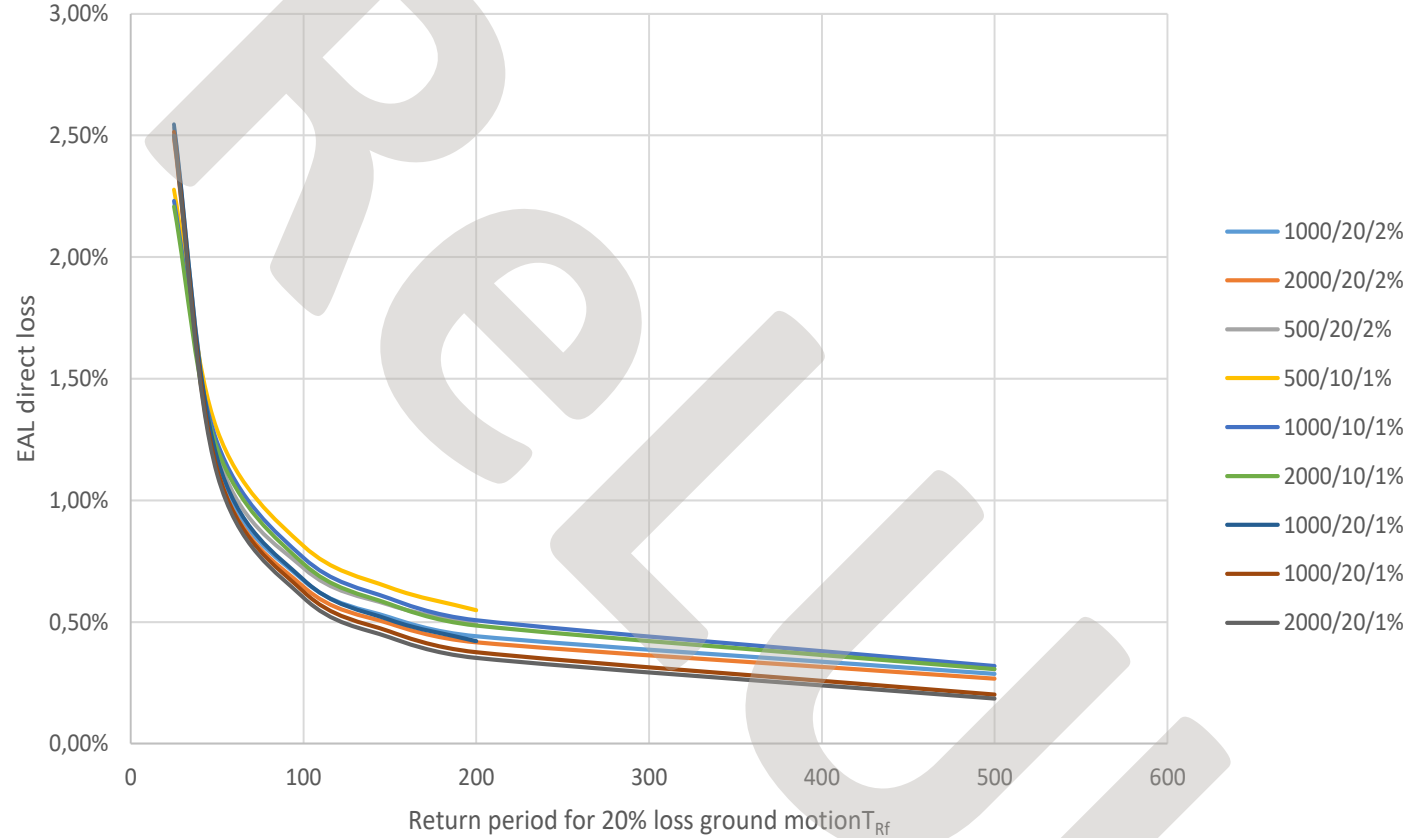
Correlation between P_e and L_D according to Eq. (2) for different EAL values



a code-conforming building may be characterized by an EAL in the range of 1.0 %

	EAL=0.1 %	EAL=0.4 %	EAL=0.6 %	EAL=0.8 %	EAL=1.2 %	EAL=1.6 %	EAL=2.0 %	EAL=2.4 %
$P_{ef}(\delta_i = 0.5\%)$	0.06%	0.33%	0.70%	1.18%	2.19%	3.01%	3.60%	4.02%
$T_{Rf}(\delta_i = 0.5\%)$ [years]	1700	305	143	84	46	33	28	25

Relationship between the annual probability of exceedance of shaking and EAL in order to respect **a drift threshold of 0.5%**



Correlation between EAL and damage control point f , as a function of the return periods of the ground motion inducing collapse and inducing the onset of damage

Loss estimation

**Indirect (business interruption) losses
may dominate**

Residential:

- Cost of homeless relocation $C_{rl}=35 \text{ €/d}$,
- Cost of reconstruction $C_R=1000 \text{ €/sqm}$ (100 % direct loss)
- Average area per person $S_p=25 \text{ sqm}$
- Total time of reconstruction $T_{rc}=2 \text{ y}=730 \text{ d}$

$$\frac{L_{IM(res)}}{C_R} = \frac{C_{rl} \cdot T_{rc}}{S_p \cdot C_R} = \frac{35 \cdot 730}{25 \cdot 1000} = 1.02$$

Societal cost is about $0.14 \%R_c/d$
 i.e ratio between indirect and direct cost ≈ 1

Bridges:

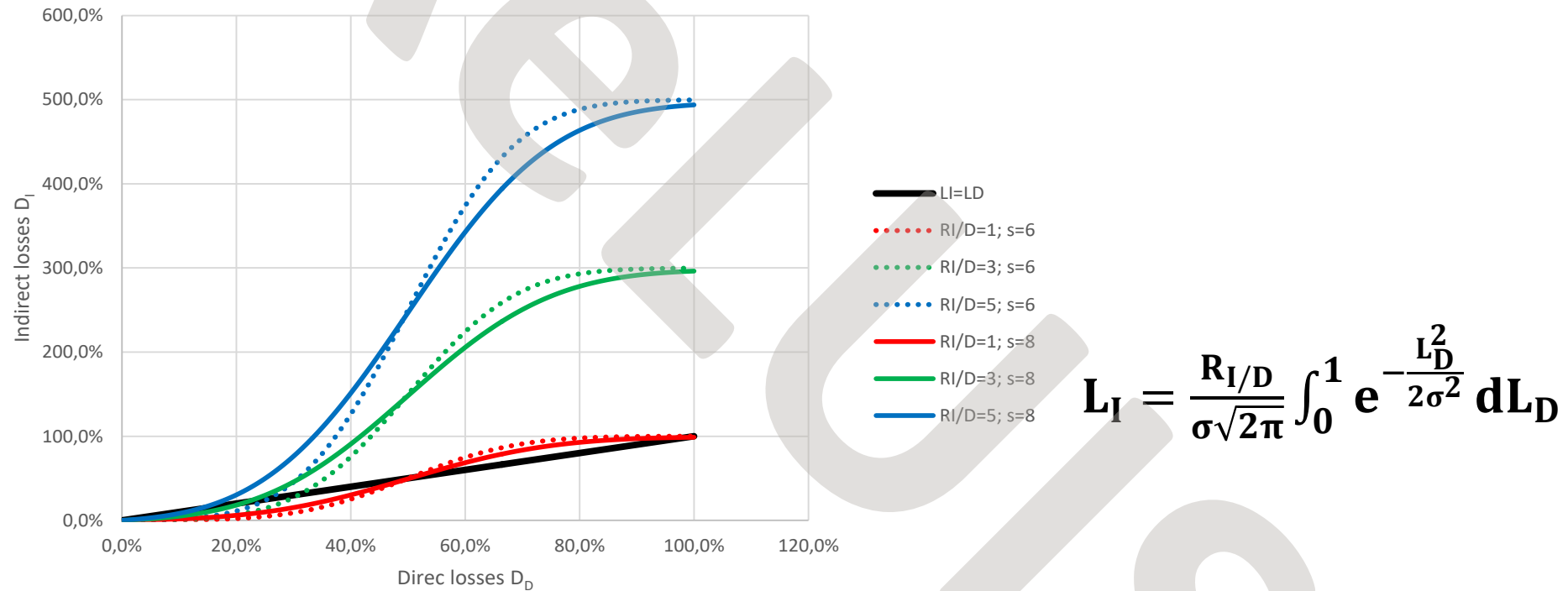
- number of vehicles (N_v) crossing the bridge,
- required detour length (D_d [km])
- unitary cost per added travelled km (C_{km})
- time required to reconstruct or repair the bridge (T_{rc})

$$\frac{L_{IM(bridge)}}{C_R} = \frac{N_v \cdot D_d \cdot C_{km} \cdot T_{rc}}{C_R}$$

Societal cost is about 0.3-0.8 % R_c/d

i.e ratio between indirect and direct cost $\approx 2 - 5$

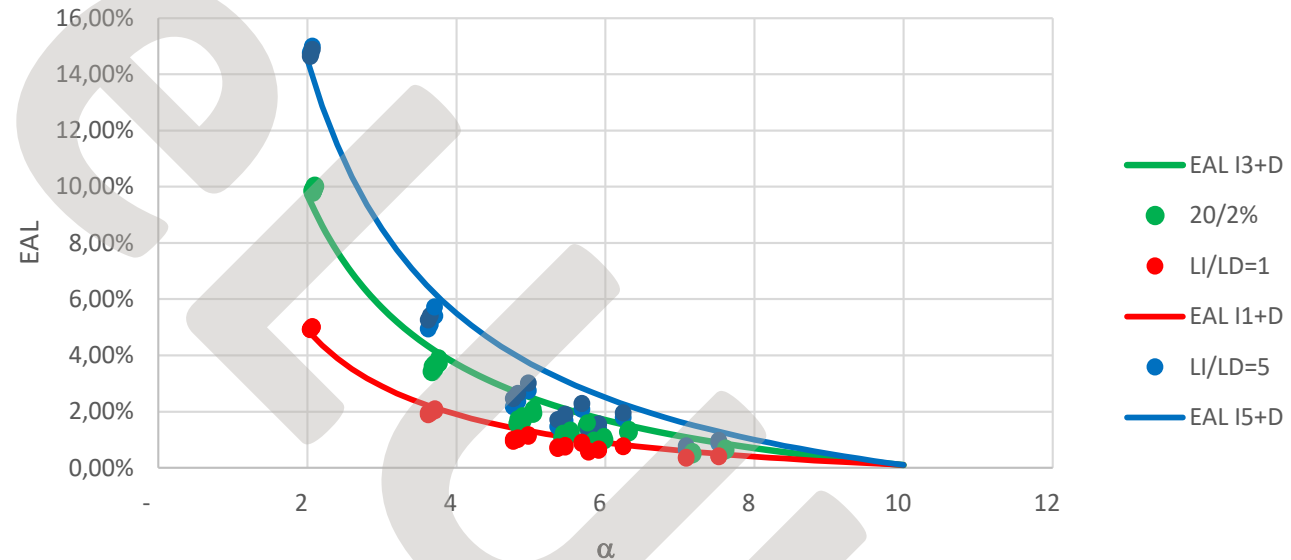
Correlation between indirect and direct cost



EAL including indirect loss

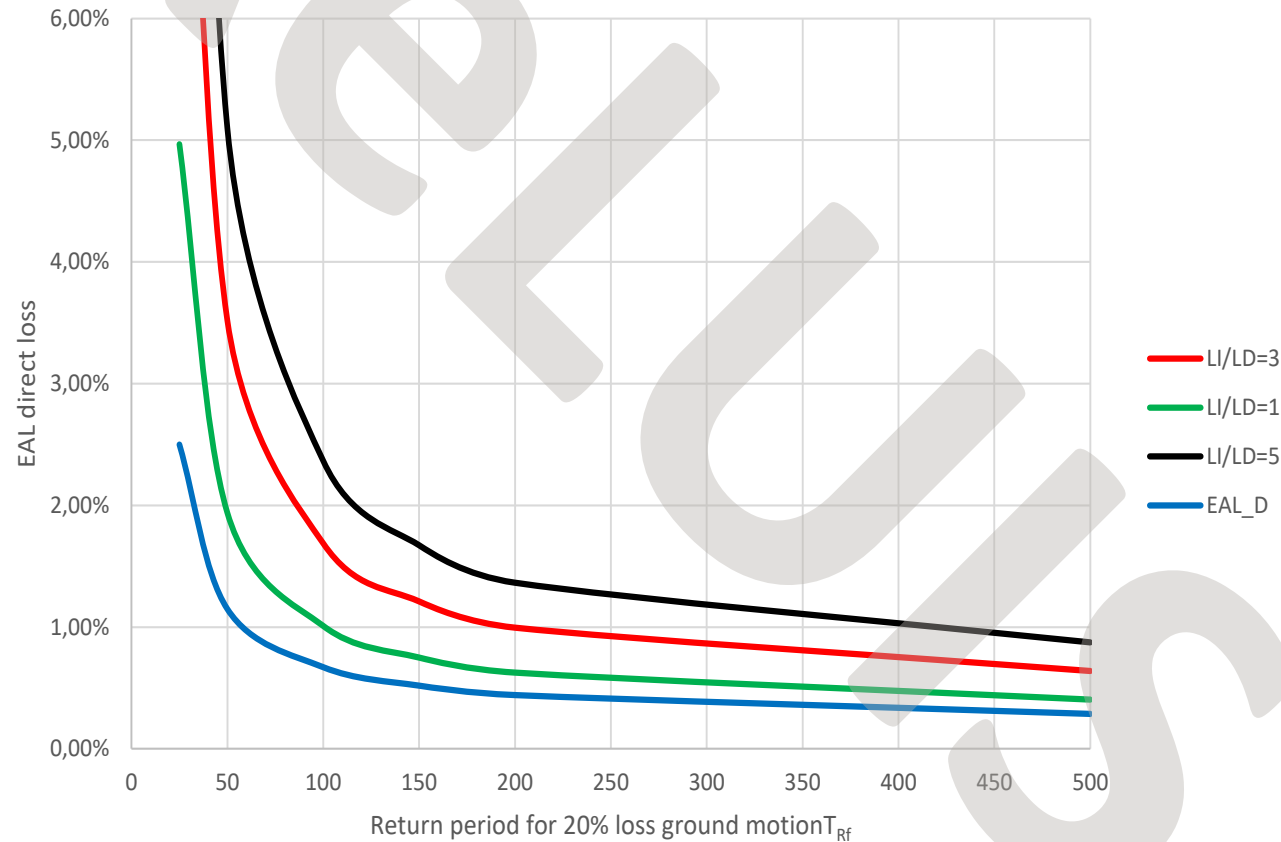
$$EAL = \frac{6\%}{\alpha} - 0.5\%$$

$$\alpha = \frac{6\%}{EAL + 0.5\%}$$



$$EAL_{I+D} = 0.06 (1 + R_{I/D}) \left(\frac{10 - \alpha}{10\alpha} \right) + 0.001 \quad \alpha = \frac{0.06(1 + R_{I/D})}{EAL_{I+D} + 0.006(1 + R_{I/D}) - 0.001}$$

Correlation between EAL and return period of a ground motion inducing 20% of R_c direct loss

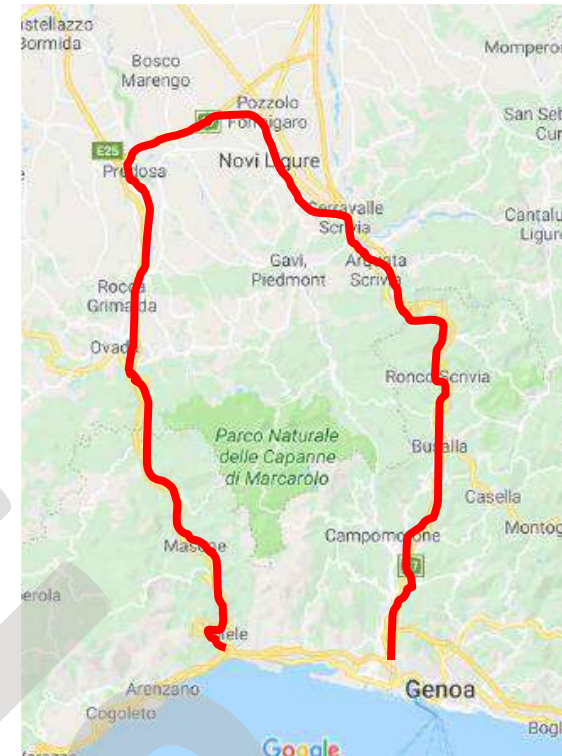


**Collapse of the Morandi
bridge:**

**Detour 20 km?
(70,000 vehicles)**

**Or 120 km?
(12,000 trucks)**

**Indirect cost,
per day \approx 2,000,000 €
i.e.: 3 months \approx reconstruction**



Bridge over the Po River on the A7

Detour 22 km

~ 100,000 vehicles/day

~ 1,000,000 €/day

Cost of reconstruction (CR) ≈ 50 M€

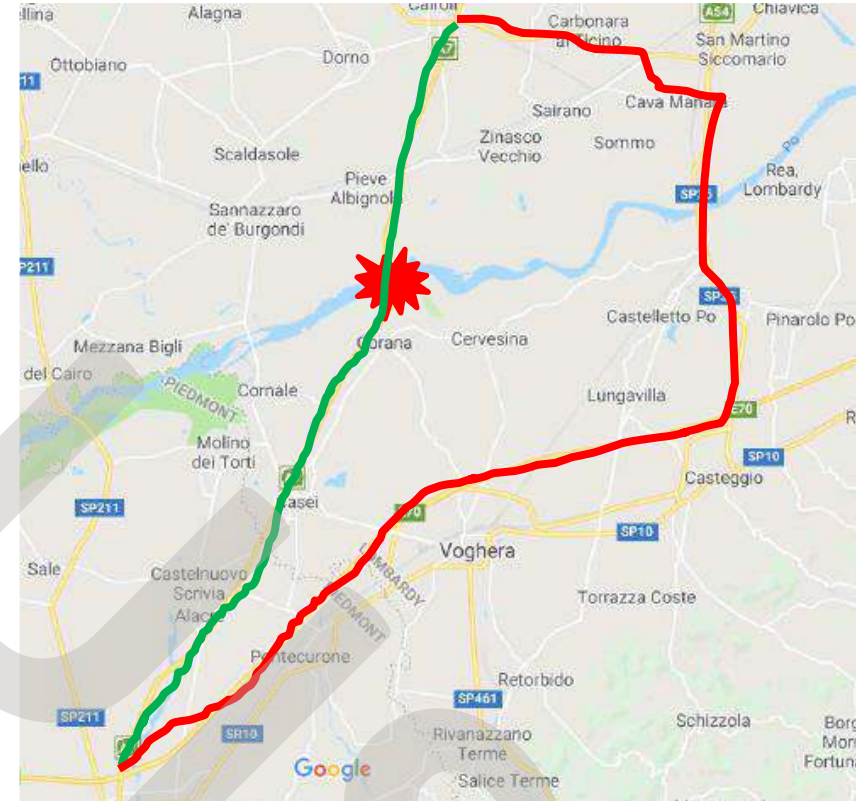
EAL/day ≈ 2%

Cost of intervention ≈ 3 M€ = 6% CR

10 % damage: from 50 to 200 y

100 % damage: from 500 to 2,000 y

EAL from 6.16 % (direct 0,83)
to 2,71 % (direct 0,48)
(1 year repay time)



Prestressed concrete

Il viadotto del Polcevera

Construction of road infrastructure in the 1960s

US

1956	President Eisenhower signs the Federal-Aid Highway Act
	the "Greatest Decade": 60,000 km, 5,000 km/y until 1960
1961	President Kennedy maintains the project and \$ 0.01 gas tax per
gallon	
1966	29,000 km completed; \$ 25 billion spent

Complete project:

- 12,957 interchanges requiring 22,252 individual structures
- 20,748 other highway grade-separation structures
- 4,361 railroad grade separations
- 14,806 other bridges and tunnels



Construction of road infrastructure in the 1960s

Europe

1920s
1930s

wisdom of
1960s
of the

1956-1964 "Autostrada del Sole": 853 bridges, 572 overpasses

First freeways in Italy

4,000 km in Germany

Eisenhower: "Germany had made me see the broader ribbons across the land"

Most European countries construct the backbone modern freeway system

2018 *The New York Times:*

- In France, the highway system comprising 12,000 bridges is in a state of chronic underinvestment, with 7% having damage that could eventually result in collapse if not addressed
- In Germany, of the 39,621 bridges monitored by the Federal Government, 10.6% are in a condition that is not satisfactory and 1.8 % are in "inadequate" condition
- Similar examples reported for other European countries



Design of bridges and pre-stressed structures in Italy in the 1960s

Arch bridges with spans in the range of 60-80 m

Pre-stressed concrete:

- Manuals and patents on elastic coaction, cable anchoring
- Gifted designers, e.g. Levi, Cestelli-Guidi, Pizzetti, Oberti, Zorzi

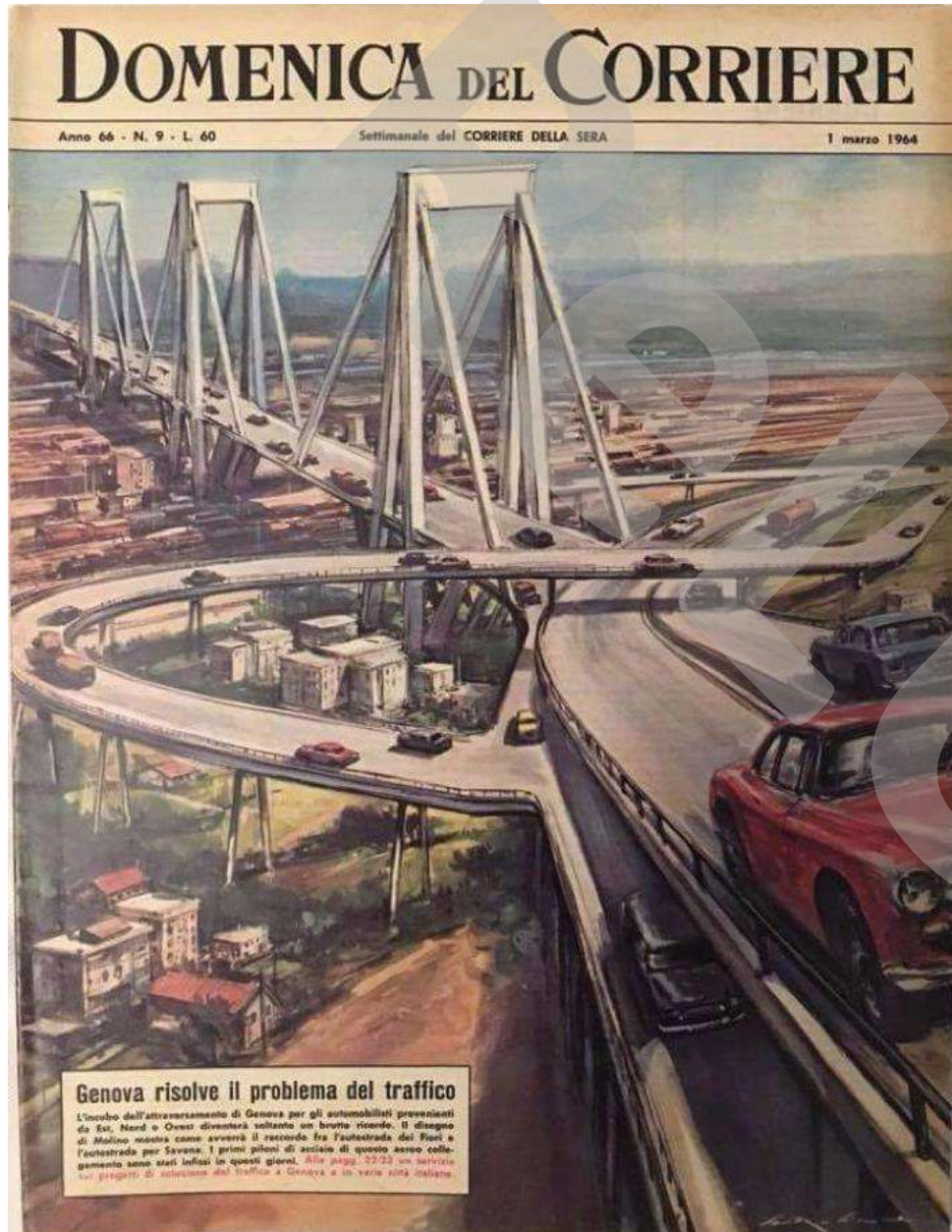
Riccardo Morandi and the bridge over the Polcevera river stood out

- Cable-stayed with single post-tensioned concrete stays
- Post-tensioned spans exceeding 200 m

But only intuitive consideration of

- Fatigue, corrosion and carbonation potential
- Time dependent effects (creep, fluage, strand relaxation)
- Temperature variations
- Redistribution effects in statically indeterminate structures
- Non-linear and ultimate response





itigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti

Earthquakes and disasters in the 1960s

- May 22 1960 the Great Chilean earthquake, $M_w = 9.5$
tsunami waves up to 25 meters devastate Hawaii, Japan, Philippines
- July 26 1963 earthquake in Skopje, Macedonia, ($M_w = 6.1$)
1,000 victims 200,000 homeless
8.5% buildings destroyed, 34% to be demolished, 36% to be repaired
no damage to the 15th century stone bridge over the Vardar
first isolated building: rubber bearings
- March 27 1964, earthquake in Alaska, $MW=9.2$
permanent ground displacements 9 m
severe damage to bridges
the Million Dollar Bridge slips off its pier due to soil liquefaction
- October 9 1963 Vajont dam disaster
260 million cubic meters slid into the reservoir
flood wave kills 2,000
the 262 m tall concrete dam undamaged
"engineering masterpiece built in the wrong place"

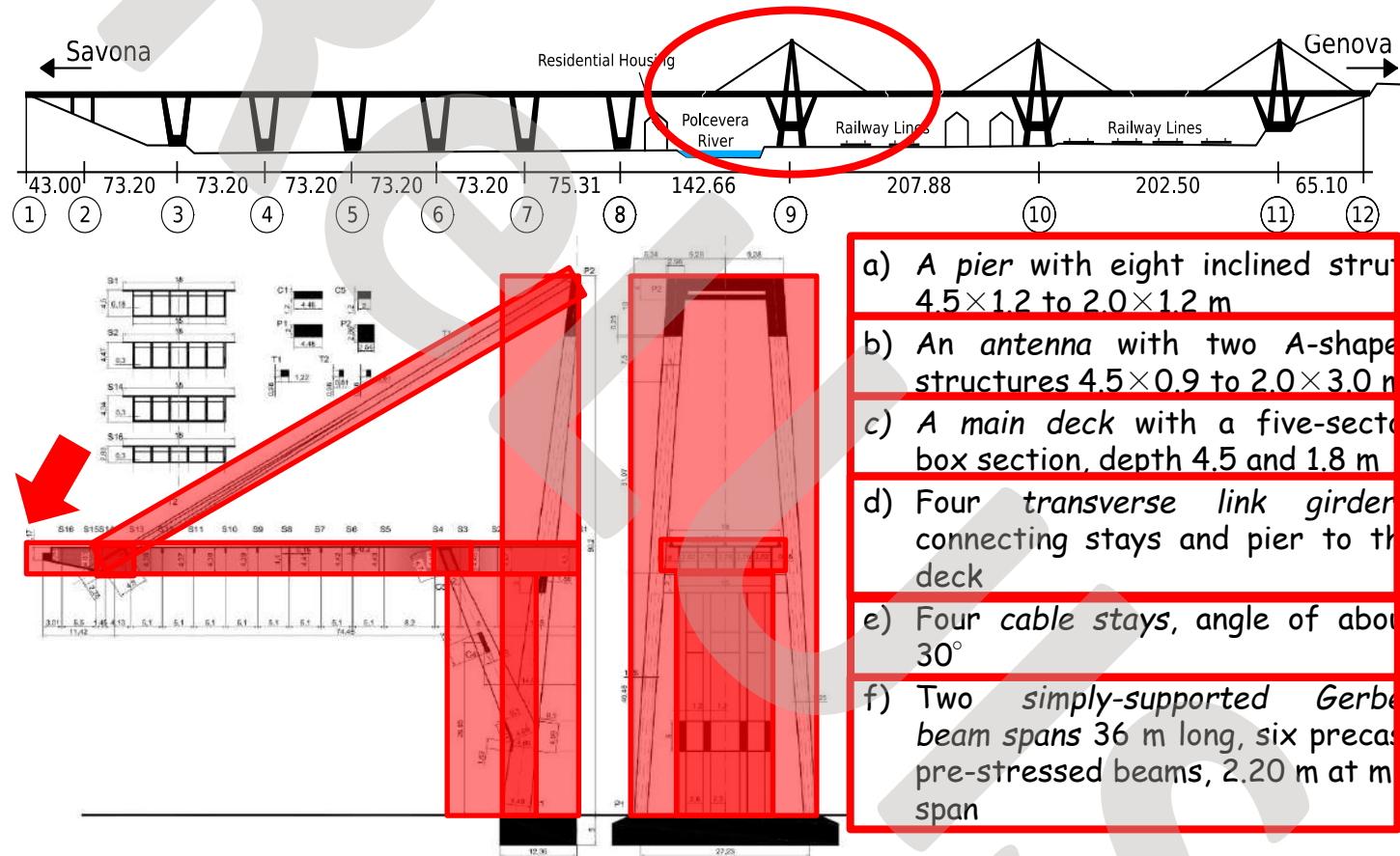
Italian OPCM 3274 (2003): compulsory verification of seismic safety of infrastructures *which functionality was fundamental for the purpose of civil protection or for which collapse would imply relevant consequences.*
Time constraints not provided in 15 years

Bridge collapses

- The Laval overpass Quebec, Canada, failed in shear killing five in 2006
- A highway overpass failed in 2016 in Lecco, Italy, killing one and injuring five, brittle failure near the drop-in span support ledge
- A post-tensioned viaduct near Fossano, Italy, in 2017, possibly loss of post-tensioning
- The bridge "Santo Stefano", near Messina, collapsed in 1999, designed by Morandi, box deck post-tensioned with the same system employed in the Genoa bridge



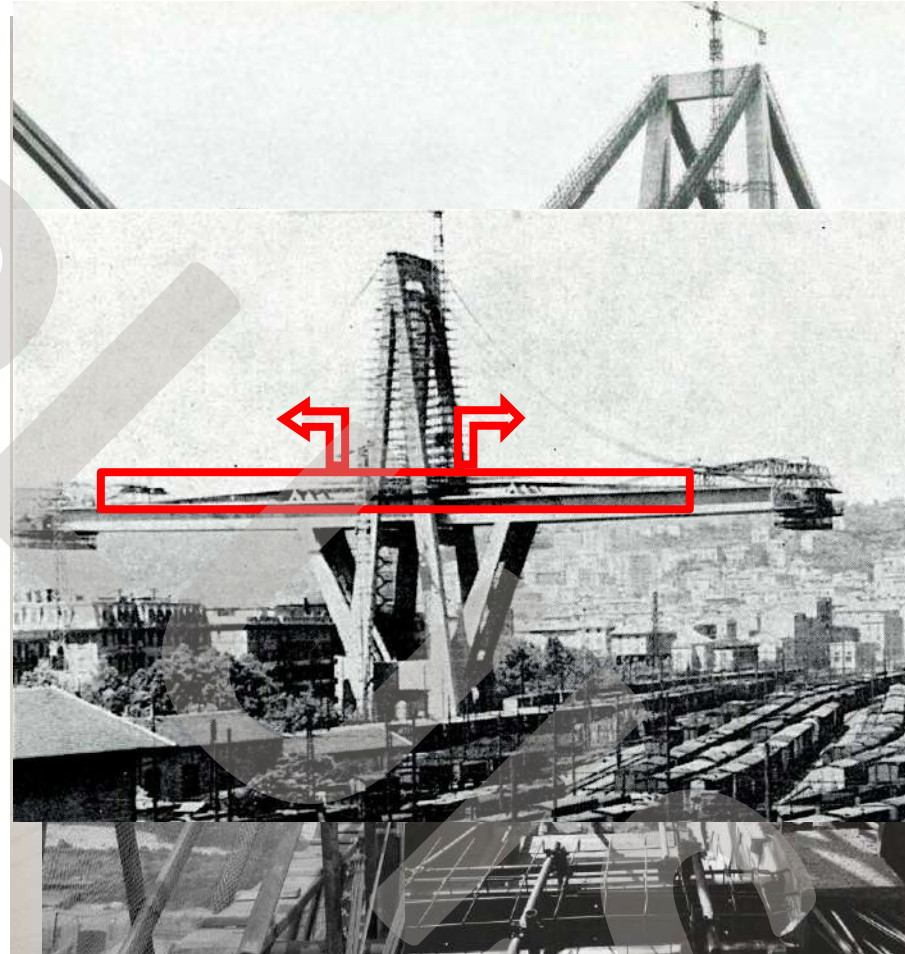
Description of the bridge

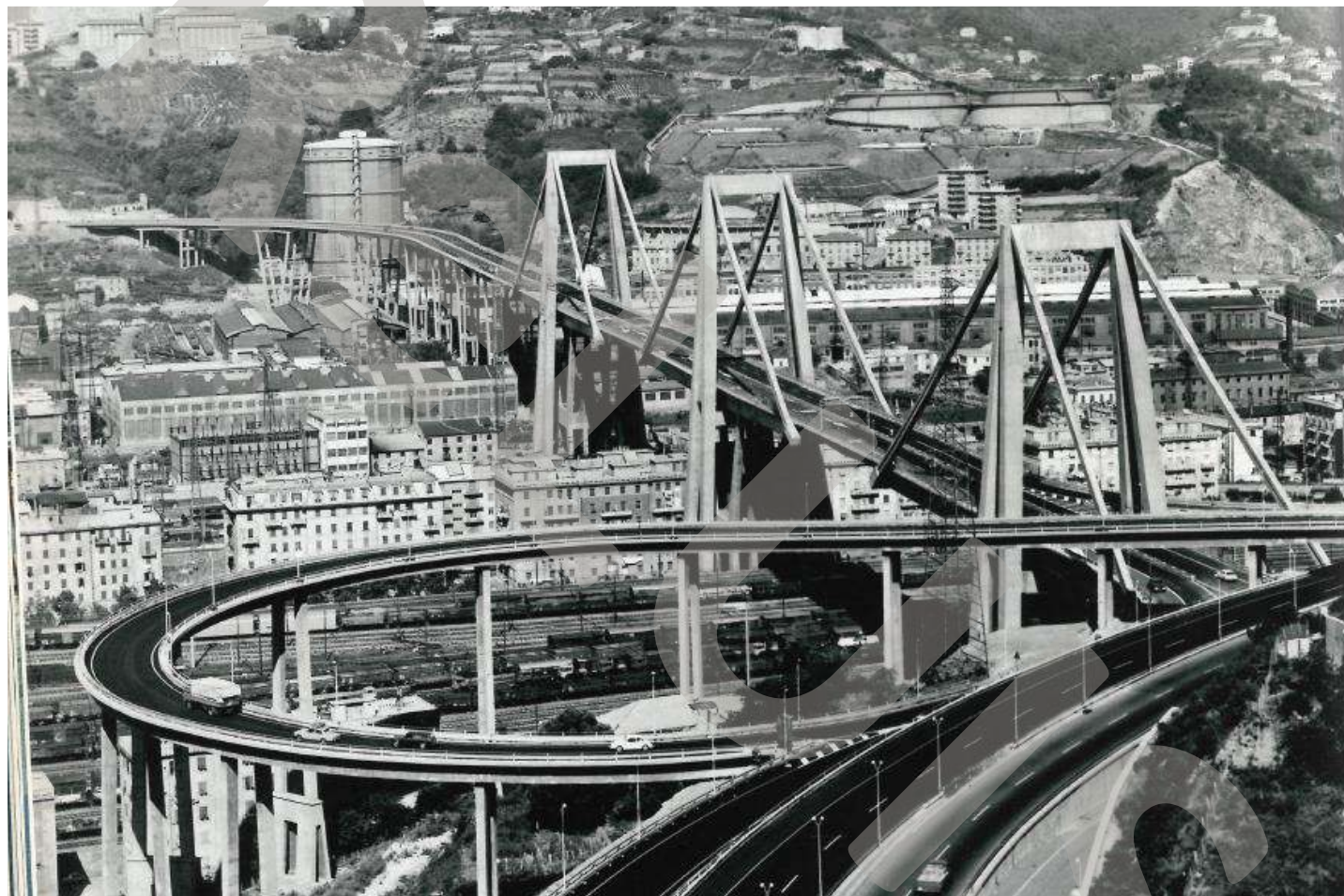


- a) A pier with eight inclined struts 4.5×1.2 to 2.0×1.2 m
- b) An antenna with two A-shaped structures 4.5×0.9 to 2.0×3.0 m
- c) A main deck with a five-sector box section, depth 4.5 and 1.8 m
- d) Four transverse link girders, connecting stays and pier to the deck
- e) Four cable stays, angle of about 30°
- f) Two simply-supported Gerber beam spans 36 m long, six precast pre-stressed beams, 2.20 m at mid span

Construction process

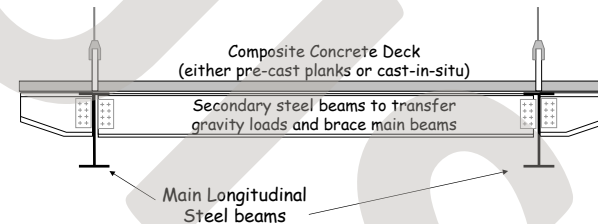
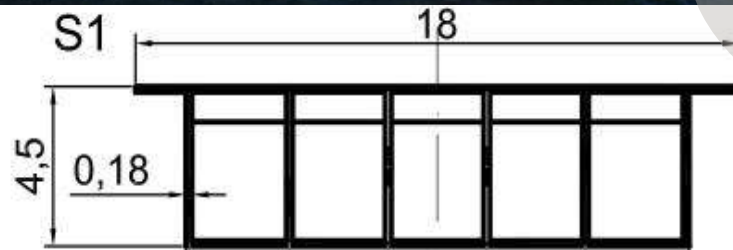
- Pier and antenna: traditional methods
- Main deck in a sequence of 5.5 m segments
- Temporarily connection by cables laying on the deck
- Post-tensioning of the stays and upward bending
- Concrete casting around the steel cable stays
- Post-compression, while still not bonded
- "Usual injection" of all ducts
- Mounting of supported spans





Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

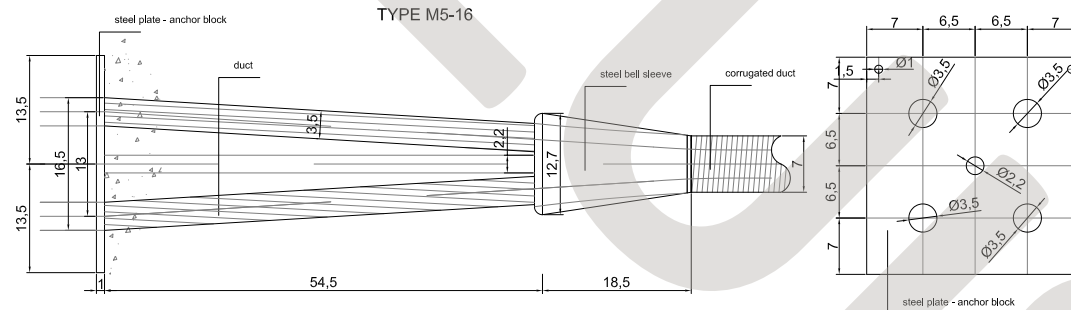
Le linee guida per i ponti esistenti



The Morandi pre-compression system

Seven-wire strands

Nominal diameter	12.7 mm ($\frac{1}{2}$ inch)
Nominal section	92.90 mm ²
Minimum strength	163 kN (i.e. 1,758 MPa)
Minimum elongation capacity	3.5 % (measured 610 mm)
Working stress	900 - 1,000 Mpa
Initial stress	1,200 - 1,300 MPa



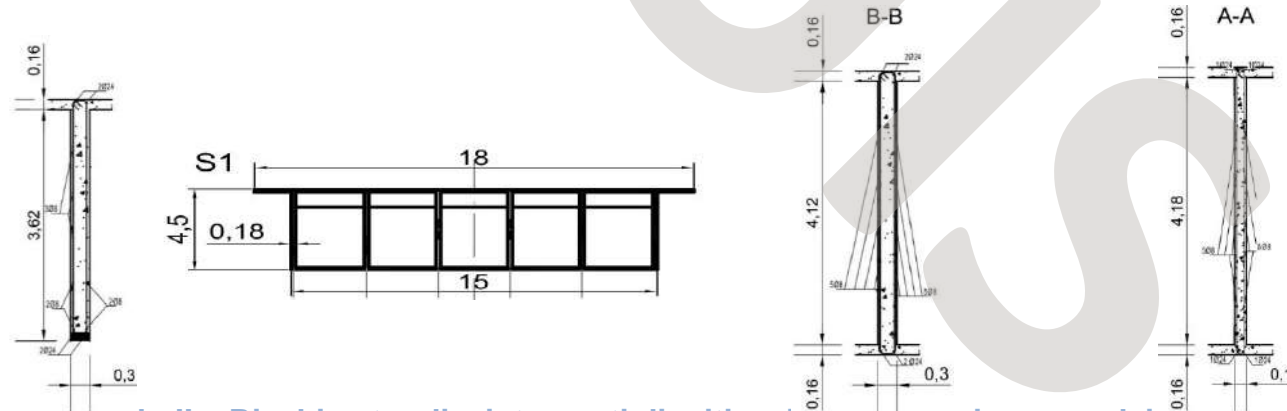
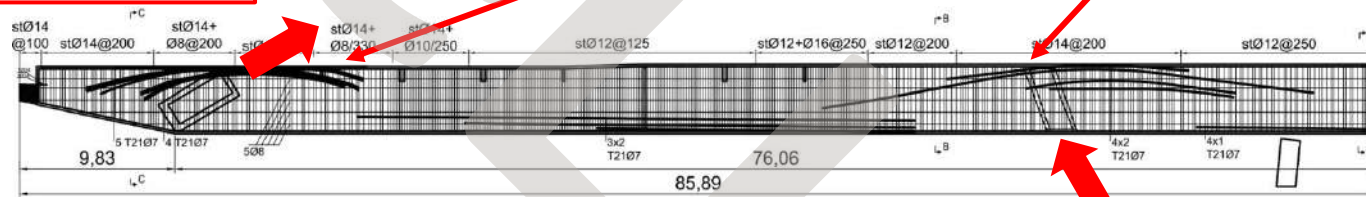
Density of steel 50% higher than today's practice

Pier and Antenna

High vertical force
 High bending moment capacity
 $LL < 20 \% (W + DL)$
 $\rho_s < 0.3 \%$

2.5 Main deck

21 Ø7 mm wires (total $A_{sp}=6,465 \text{ mm}^2$)



Transverse link girders

4.5 × 2.0 m

t = 0.5 - 1.0 m

No detail available about reinforcement and pre-stressing, if any

Cable stays

**464 total strands, diameter $\frac{1}{2}$ inch
 352 located first and connected to the deck
 112 strands to post-compress the concrete section**

**Force induced by W: 12,000 kN
 Force induced by DL: 10,500 kN
 Force induced by LL: 4,000 kN.**

<p>Phase 1 - Deck connection: 12,000 kN on 352 tendons</p> <p>Phase 2 - Post-tensioning at 900 MPa on 112 tendons</p>	<p>$\sigma_{s352} = 367 \text{ MPa}$ $\sigma_{s112} = 900 \text{ MPa}$</p>	<p>$\sigma_c = -8.7 \text{ MPa}$</p>
<p>Phase 3 - Addition of supported span and DL, 10,500 kN (assuming a ratio between elastic moduli $E_s/E_c = 10$)</p>	<p>$\sigma_{s352} = 367+75 = 442 \text{ MPa}$ $\sigma_{s112} = 900 \text{ MPa}$</p>	<p>$\sigma_c = -8.7+7.5 = -1.2 \text{ MPa}$</p>
<p>Phase 4 - At the extreme condition of maximum LL, 4,000 kN</p>	<p>$\sigma_{s352} = 436+28 = 464 \text{ MPa}$ $\sigma_{s112} = 900 \text{ MPa}$</p>	<p>$\sigma_c = -1.2+2.8 = 1.6 \text{ MPa}$</p>

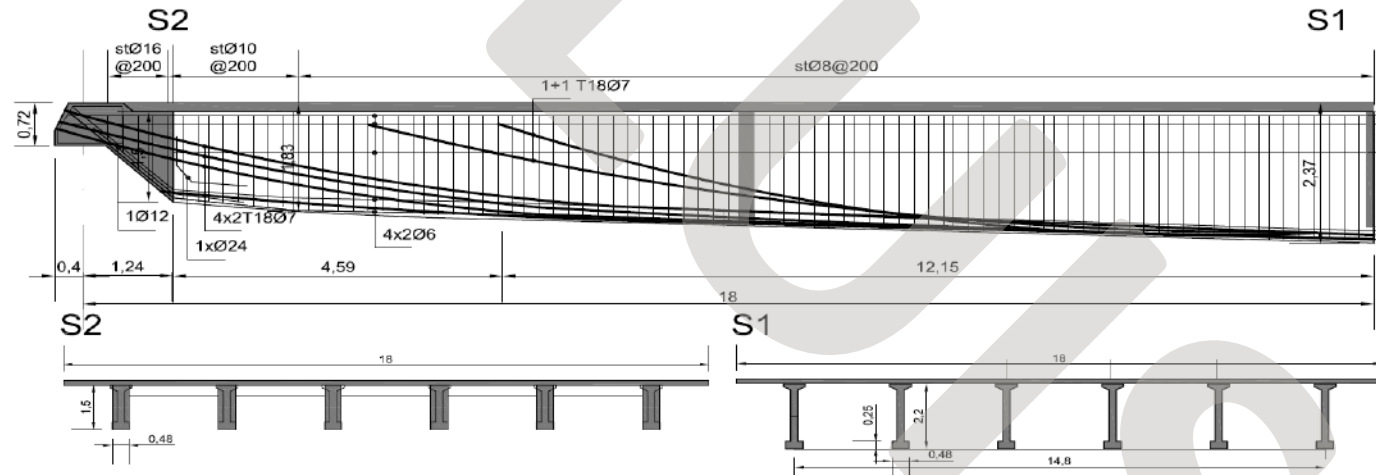
Cable stays

Effect of absence of injection:

1. **Bonded tendons:** tension stiffening reduce the cable elongation.
Unbonded: no tensile force in concrete, stiffness of tendons alone.
2. **Bonded tendons:** wire fracture induce opening of a crack.
Unbonded: global elongation and loss of compression in the entire stay.
3. **Strands at top of pier, curved on a saddle:** flexural fatigue component.
4. **Sensitivity to aggressive environment and potential corrosion much higher**

Supported spans

- Beam depth 2.20 m, cast in place slab 0.16 m
- 10 cables, 18 $\varnothing 7$ mm wires each
- Total pre-stressing force, after losses, 6,235 kN (900 MPa steel stress)
- Pre-stressing bending moment at mid span 6,235 kNm



Structural assessment, strengthening and monitoring from

early 1990s: "during maintenance and repair activities, it was discovered that the stays of the three balanced systems were suffering from widespread general deterioration, as well as several instances of concentrated degradation"

Pier 11: all tendons fully replaced by external cables

Pier 10: "the criticalness was mainly concentrated in the sections stretching to the cross-beam at the top of the tower, and hence the interventions were limited to these areas"

Pier 9 (now collapsed): "the stays are in better condition due to the more limited corrosion present in both primary and secondary cables. Hence no intervention of any type is scheduled"

As a safety measure: "control over time of the state of conservation of the pre-stressing cable is assured through the installation of a system of continuous reflectometer control"

Conclusion: "from an estimate of the intervention time limit, and considering the empirical laws which govern the speed of degradation, **the limit condition is estimated to be around 2030**"

From monitoring:

- a) Absence of injection repeatedly evident
- b) Presence of corrosion reported, extension between 10 and 30 % (year 2015)
- c) Loss of post-tensioning reported, some strands free to move (2011-2015)
- d) Dynamic identification tests: inconsistent responses of different stays, but correlation with numerical models described (2017).



Estimation of member demands and assessment of capacity

- a) **Deterioration of the stay strands (fast elongation or fatigue collapse)**
 - Migration of the tensile force in the opposite stay
 - In-plane rotation of deck and torsional effect
 - No solid estimate of the exact time of the event
- b) **Elongation of stay cables and shear/torsion collapse in the deck**
 - Shear strength of deck counting on compression induced by stays
 - Local point load increment triggering event (tumble of the steel coil?)
 - Unbonding of cables favours this failure sequence
- c) **Local failure in some part of the deck**
 - Loss of post-tensioning in the terminal cantilever?
 - Shear collapse and loss of support for the simply supported span?
- d) **Shear failure of the simply supported span**
 - Region next to the Gerber saddle
 - Triggered by a local load increment
 - Sudden release of applied force on the main deck
 - Migration of compression force to adjacent beams
 - Torsional effects and failure of main deck and stays

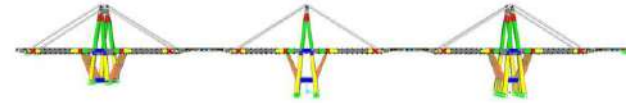
Estimation of member demands: elastic static analysis

1) pier, antenna and central span, self-weight only

2) deck and stays (352 $\frac{1}{2}$ inch tendons)

- 140 mm total shortening
- Stay force 12,300 kN
- Vertical reaction at pier base 170 MN
- Deck displacements +96 mm and - 120 mm

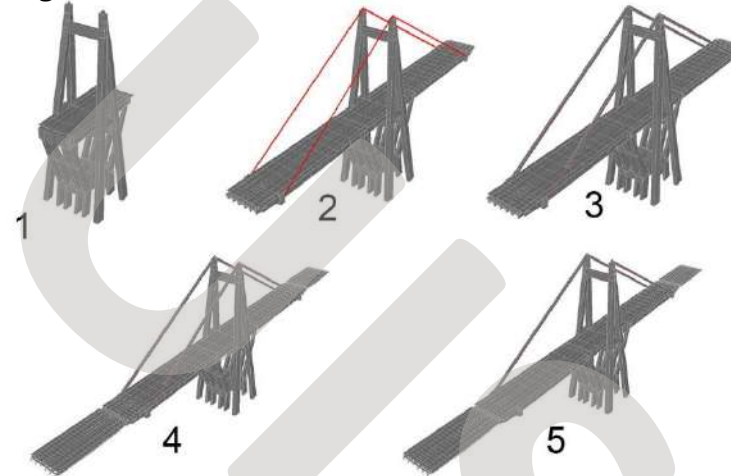
SAP2000



3) Cable post-tensioning and concrete casing

4) Supported spans and DL

- Stay force 22,600 kN
- Vertical reaction at base 212 MN
- Deck compression 21-28 MN
- LL: variations 5 - 20 %
- LL: deck displacements 5 mm
- Tendons stress 650-750 Mpa



5) S-W stay removed

- Stay force 39,000 kN
- Deck displacement 400 mm
- Base overturning moment 918 MNm

Estimation of member demands: dynamic analysis

OPENSEES: essentially same results for gravity loads

Seismic analyses and verifications

(ten spectrum-compatible accelerograms at each intensity)

Limit states: operational, damage limitation, life safety and collapse prevention

Return periods: 120, 201, 1808 and 2475 years

Nominal life: 200 years

Soil type: C, shear wave velocity 180 - 360 m/s

PGA (NC) 0.184 g

Essentially no problems in deck

At base:

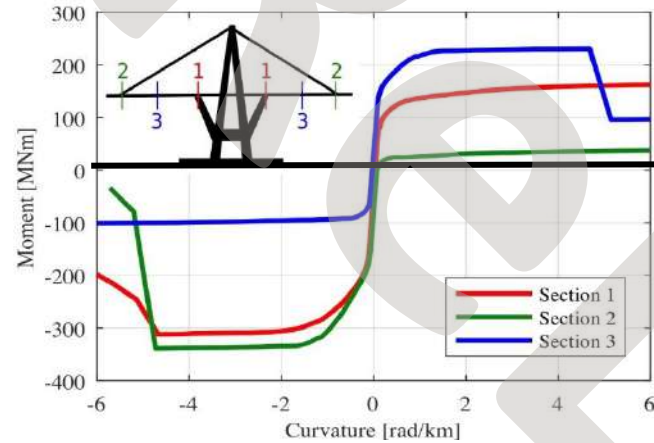
In antenna legs axial load ratio from $n=0.18$ to $n=0.21$

In pier legs from $n=0.05$ to $n=0.11$

Only minor tensile stresses

Essentially no problems in pier and antenna

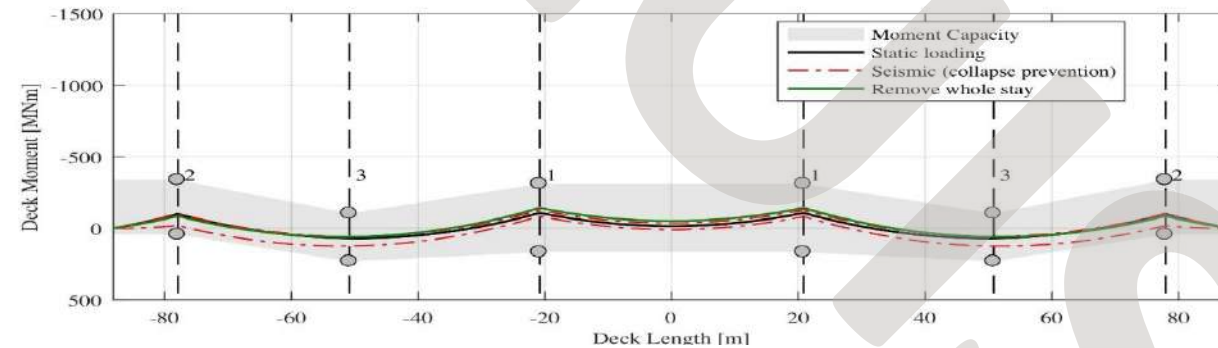
Flexure capacity of main deck and supported spans



Response-2000

Main deck

Collapse for stay removal



Shear capacity of main deck and supported spans

Minimum safety factor for impulse load
(m=30 t, dynamic amplification)

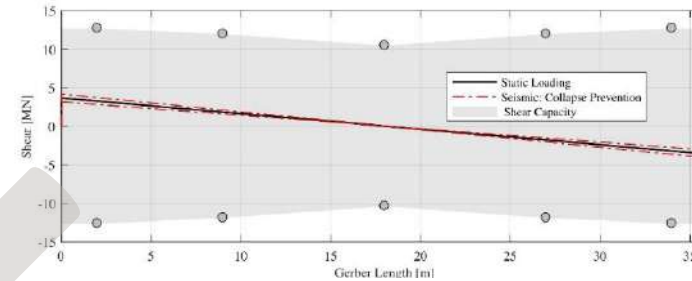
1.53

Failure at impulse
for **60 % loss** of tendons steel

Ledge safety factor for impulse

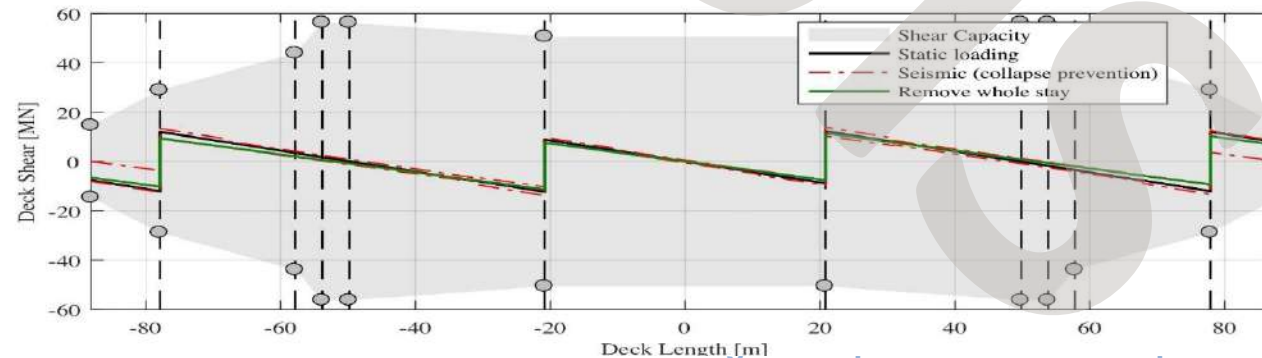
2.52

Modified
Compression Field
Theory (MCFT)



Supported span

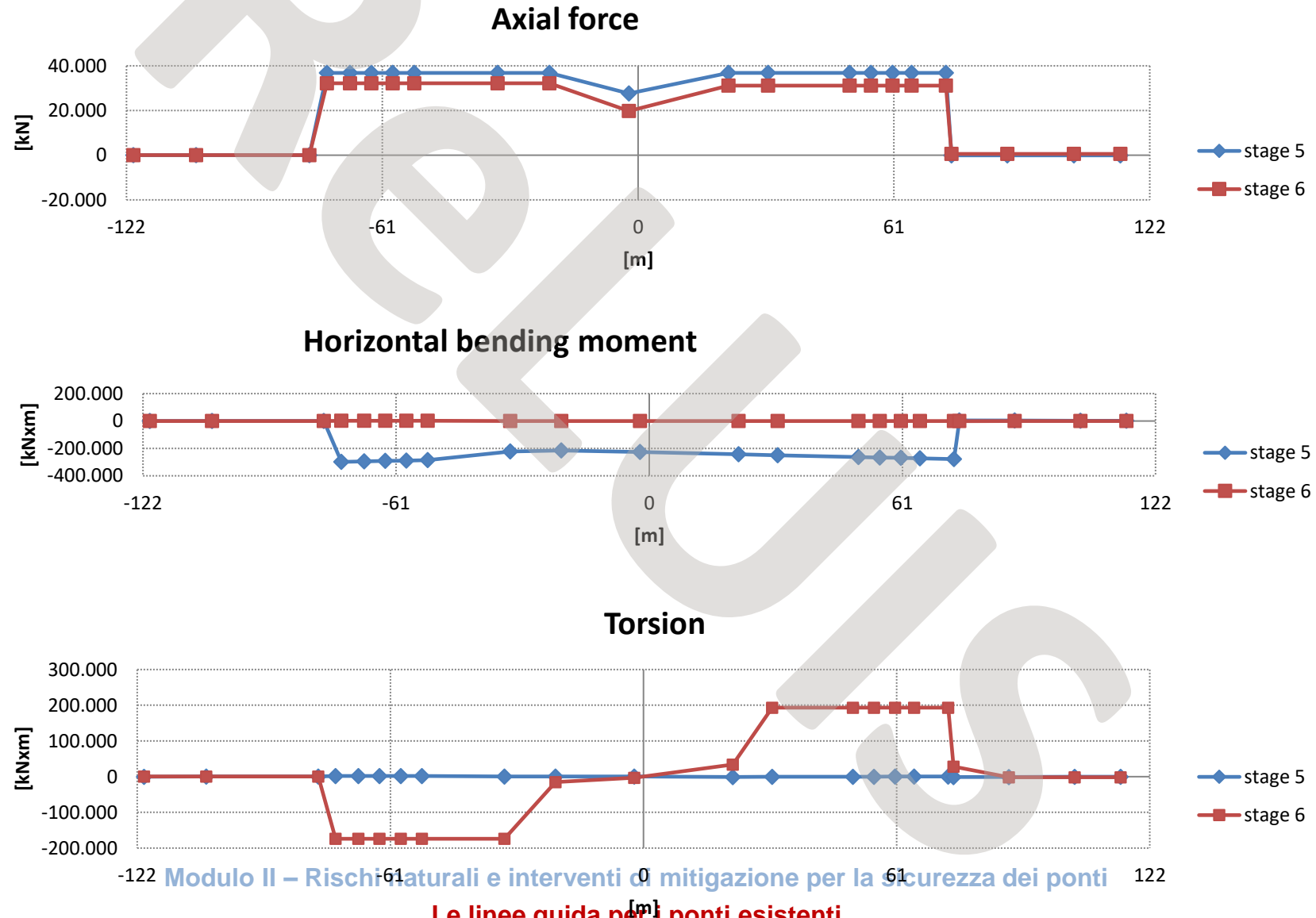
Main deck



Le linee guida per i ponti esistenti

Torsion capacity of main deck

Variable-Angle Truss Model



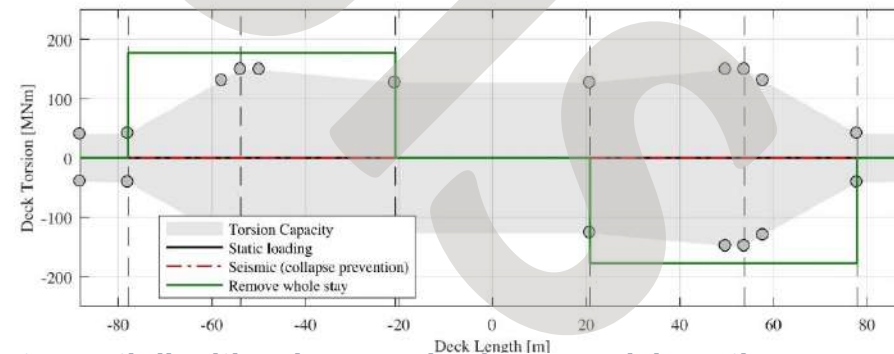
Torsion capacity of main deck

Variable-Angle Truss Model

Simultaneous presence of torsional, shear and flexural actions

- Moments and axial forces resisted by axial stresses in the chords
 - Shears and torsions (non-uniform) resisted by shear flows in the walls
 - Torsion (uniform) distributed along the perimeter of the cross section
-
- Large safety factors in standard conditions
 - Failure for stay removal ($M_+ \approx 175 \text{ MNm}$)
 - Likely at section 1 (peak of moment and shear)

Moments, axial force and shear from gravity:
remaining torsion capacity



Summary and temporary conclusions

- The flexural shear and torsional capacities of the deck are in the range of two or more times the demand under normal loading conditions
- The shear and flexural capacities of the deck are so large that may even sustain the impact of a stay removal
- The live loads are only a small fraction of the permanent loads and cannot change significantly the stress and strain demand

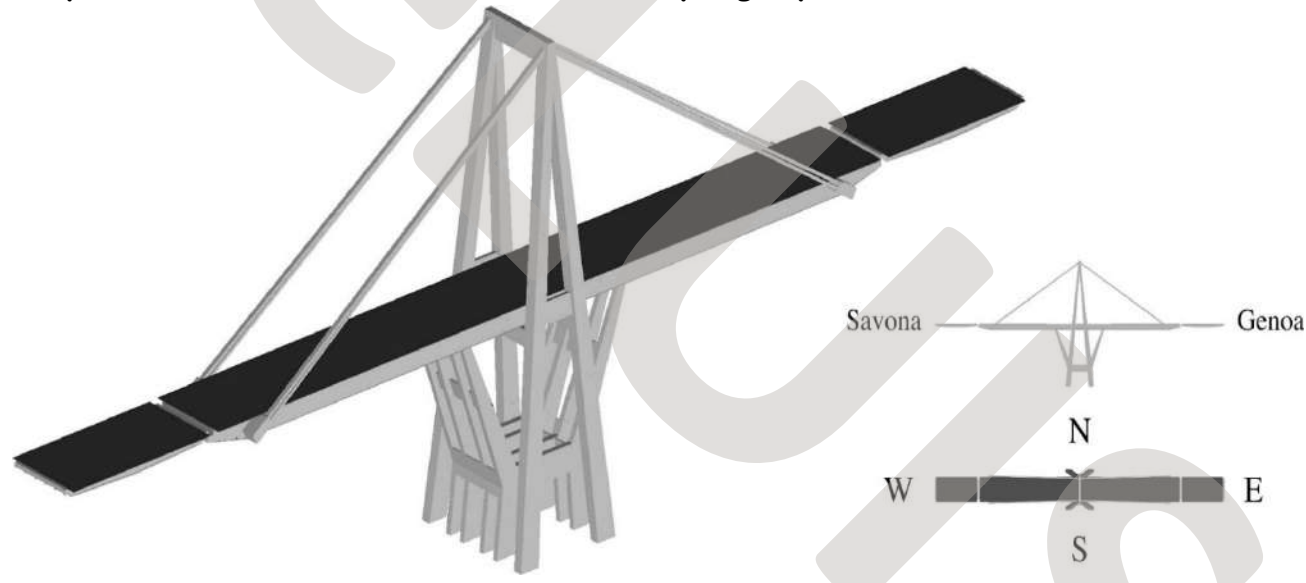
A shear-torsional collapse in the deck (in a section close to the pier) can be induced by a stay removal

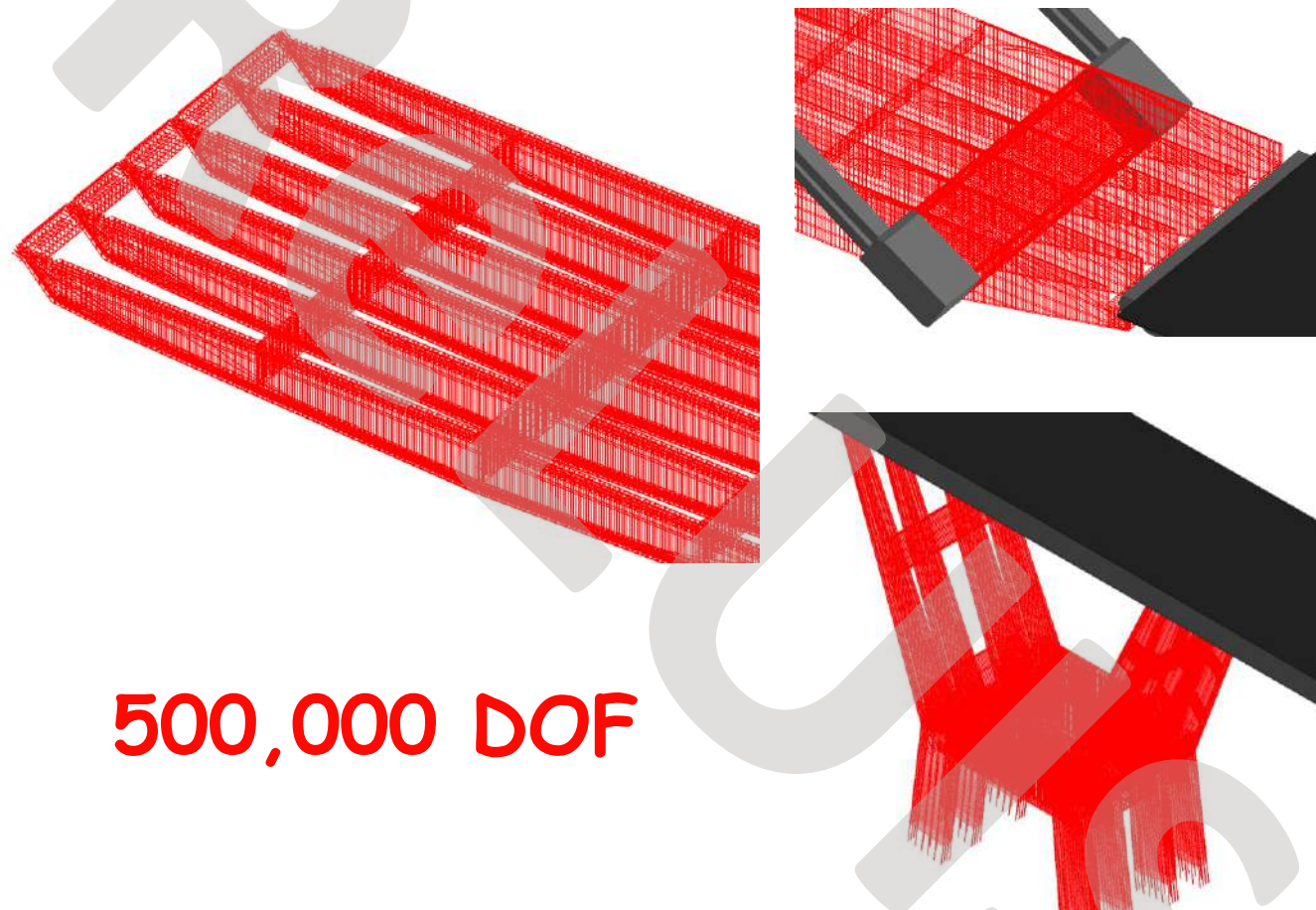
Exceptional point loads, possibly in combination with some pre-stressing tendon deterioration, may induce local failures, that not necessarily imply a global collapse of the bridge

Assessment and explicit modelling of possible collapse mechanisms

Extreme Loading for Structures

Applied Element Method (AEM)
 mechanical interaction between rigid bodies connected to each other
 by zero-thickness interface non-linear spring layers





500,000 DOF

5 Assessment and explicit modelling of possible collapse mechanisms

Verification of ELS against SAP2000 and OpenSees:

Initial cable shortening

148 mm

(SAP2000: 140,

OpenSees: 145)

Vertical reaction at base

165 MN

(SAP2000: 170,

Open

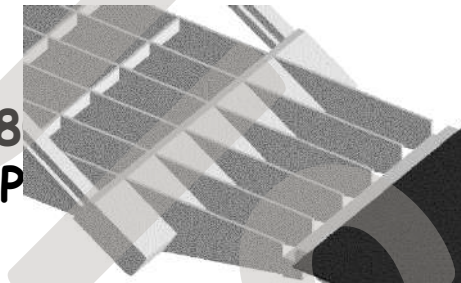
Axial

Open

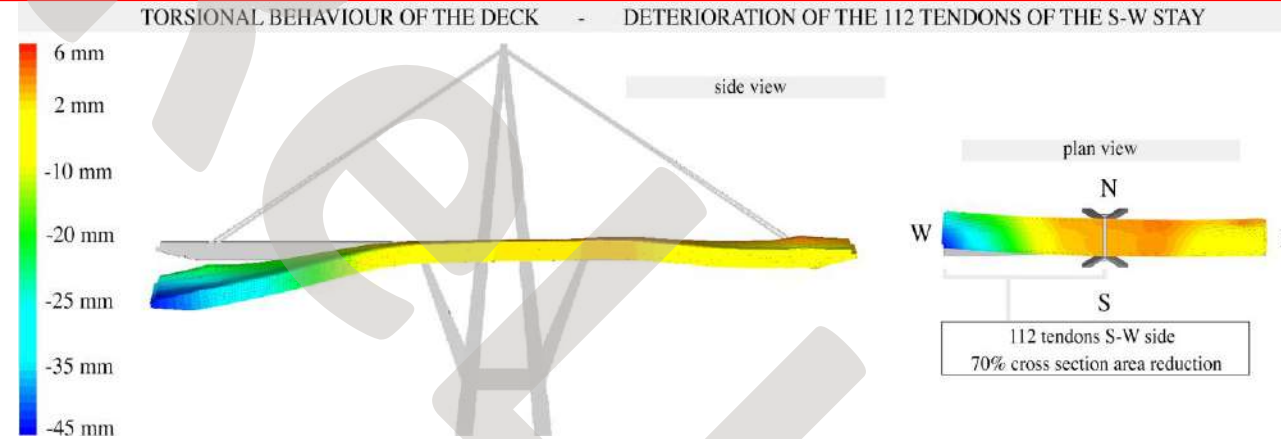


20,8

(SAP



Scenario 1 – Progressive deterioration of the reinforcement in the stays



70% reduction of area of the 112 post-compression tendons

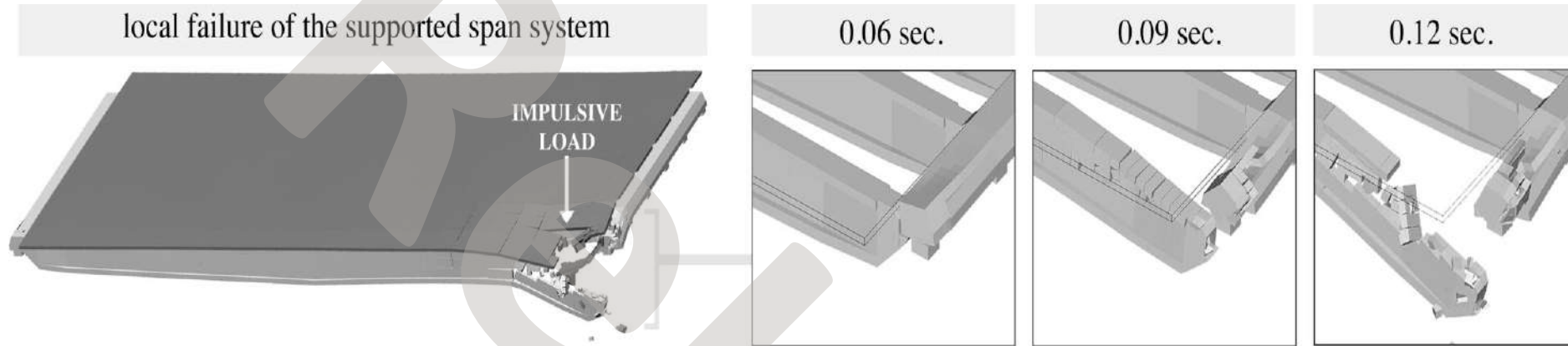
45 mm displacement

Progressive area reduction also for the 352 pre-tensioned cables

- **At 50% reduction: maximum displacement 480 mm**
- **No global collapse**

- **At 70 % reduction of all tendons: collapse**

Scenario 2 - Collapse induced by an impulsive load on critical sections



Isolated pre-stressed beam with 60 % pre-stress reduction

Ultimate capacity 650 kN (according to MCFT 687 kN)

Entire span without reinforced slab

Ultimate capacity 800 kN

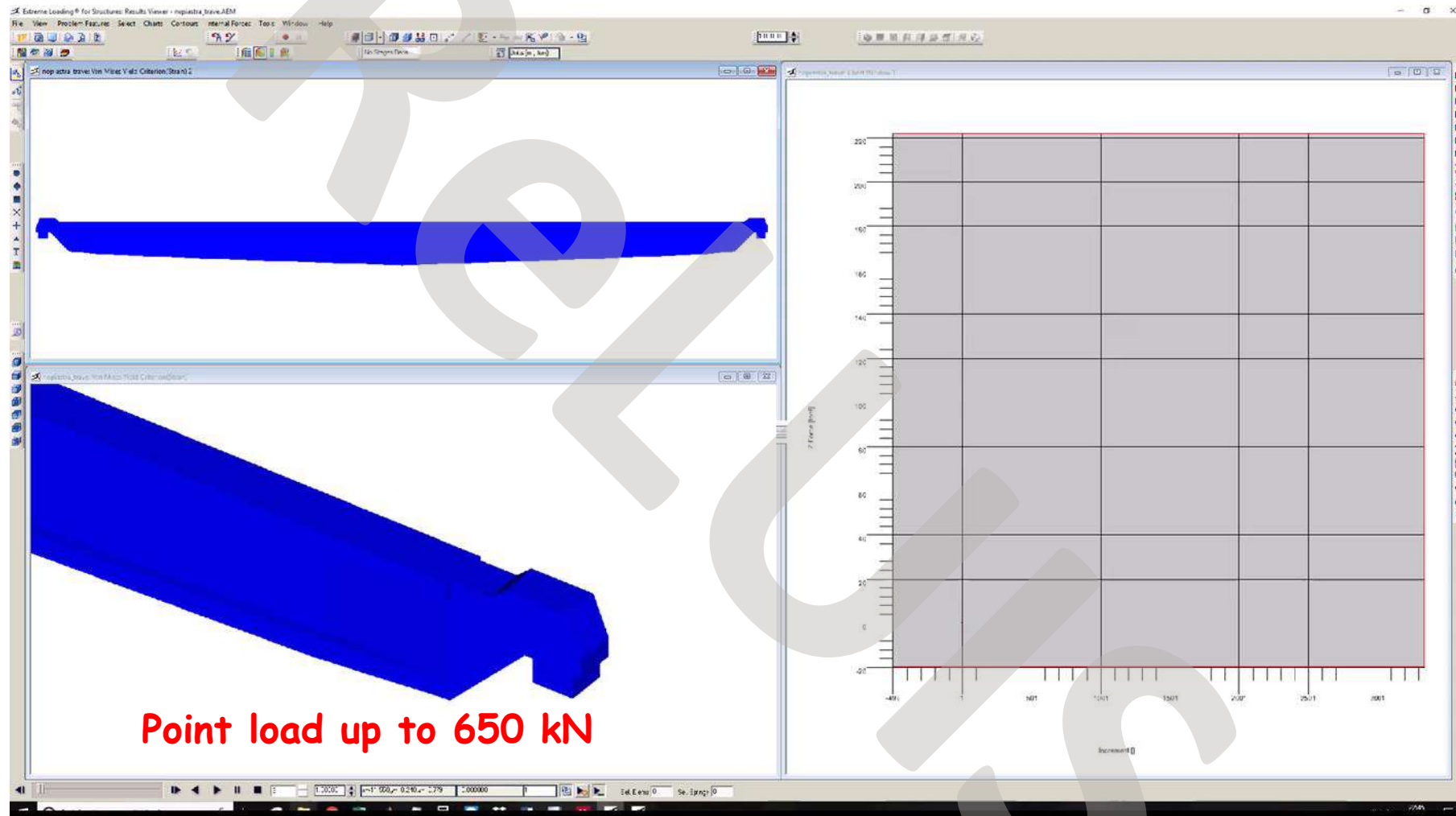
Entire span including 160 mm reinforced slab

Ultimate capacity 2250 kN

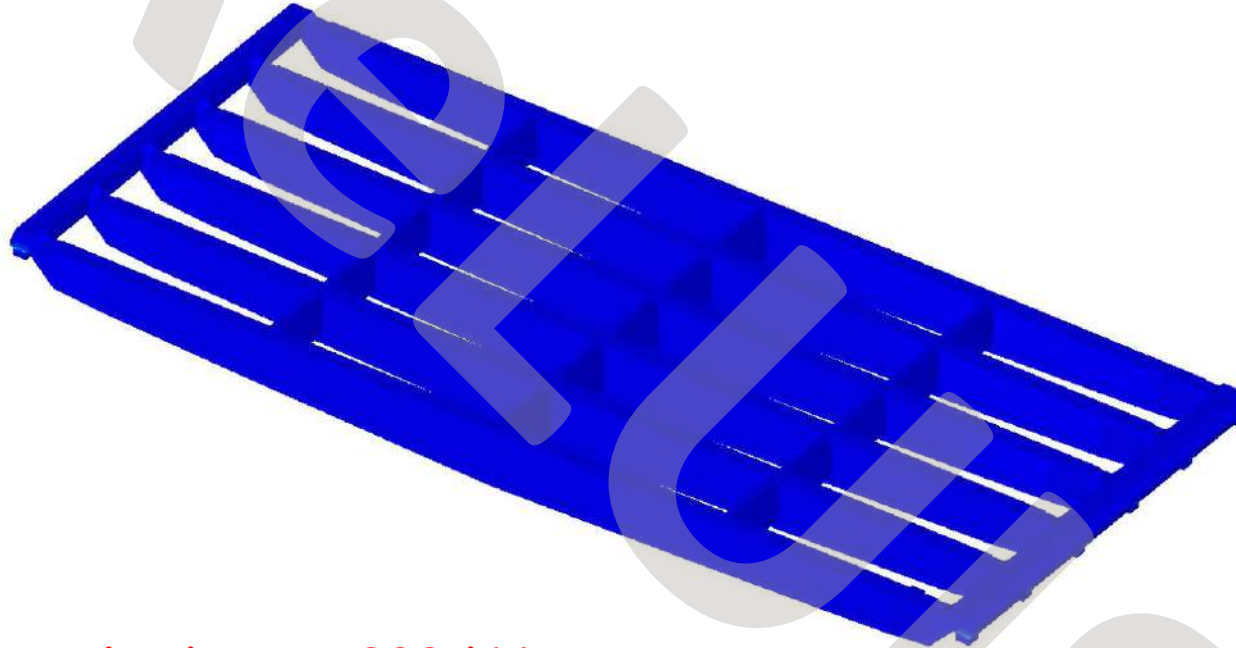
Even in case of beam failure

No global collapse

Scenario 2 - Collapse induced by an impulsive load on critical sections

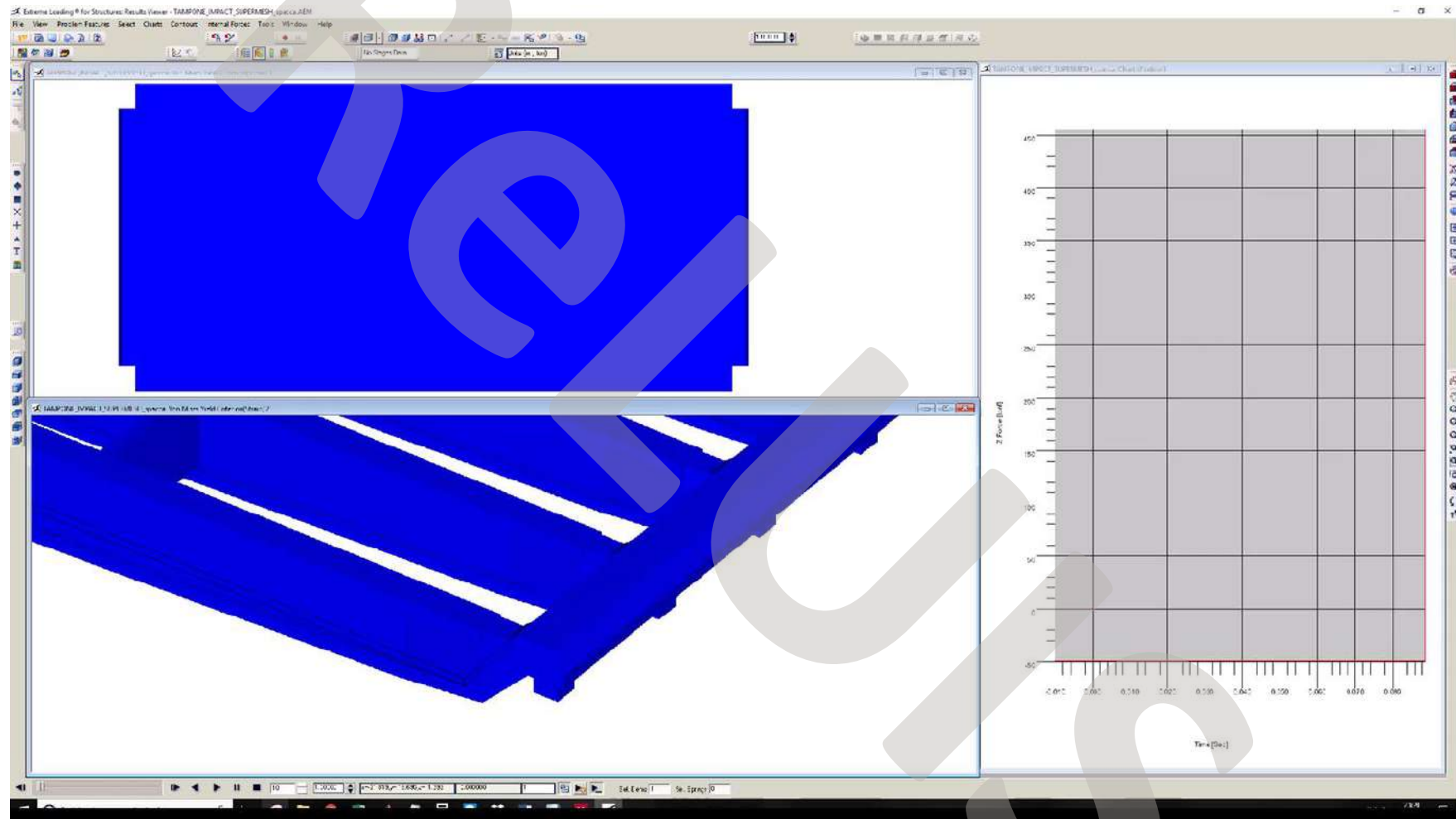


5.2 Scenario 2 - Collapse induced by an impulsive load on critical sections



Point load up to 800 kN

5.2 Scenario 2 - Collapse induced by an impulsive load on critical sections



Point load up to 3,500 kN

Le linee guida per i ponti esistenti

Scenario 2 - Collapse induced by an impulsive load on critical sections

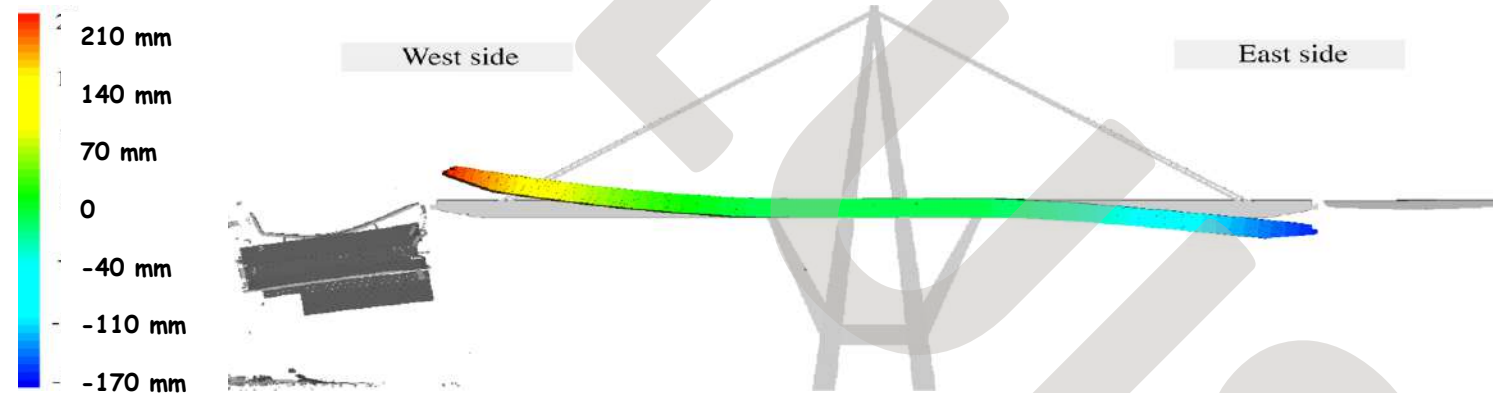
Effect of failure of supported span

Vertical displacements:

+160 and +170 mm towards N-W and S-W

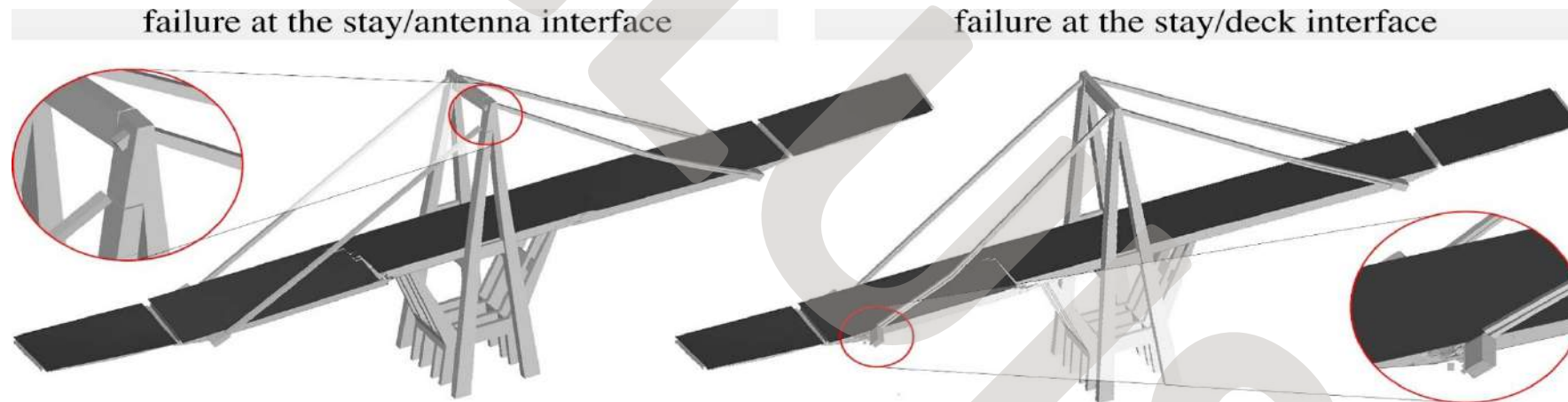
-135 and -145 mm on the N-E and S-E sides

no collapse of the bridge

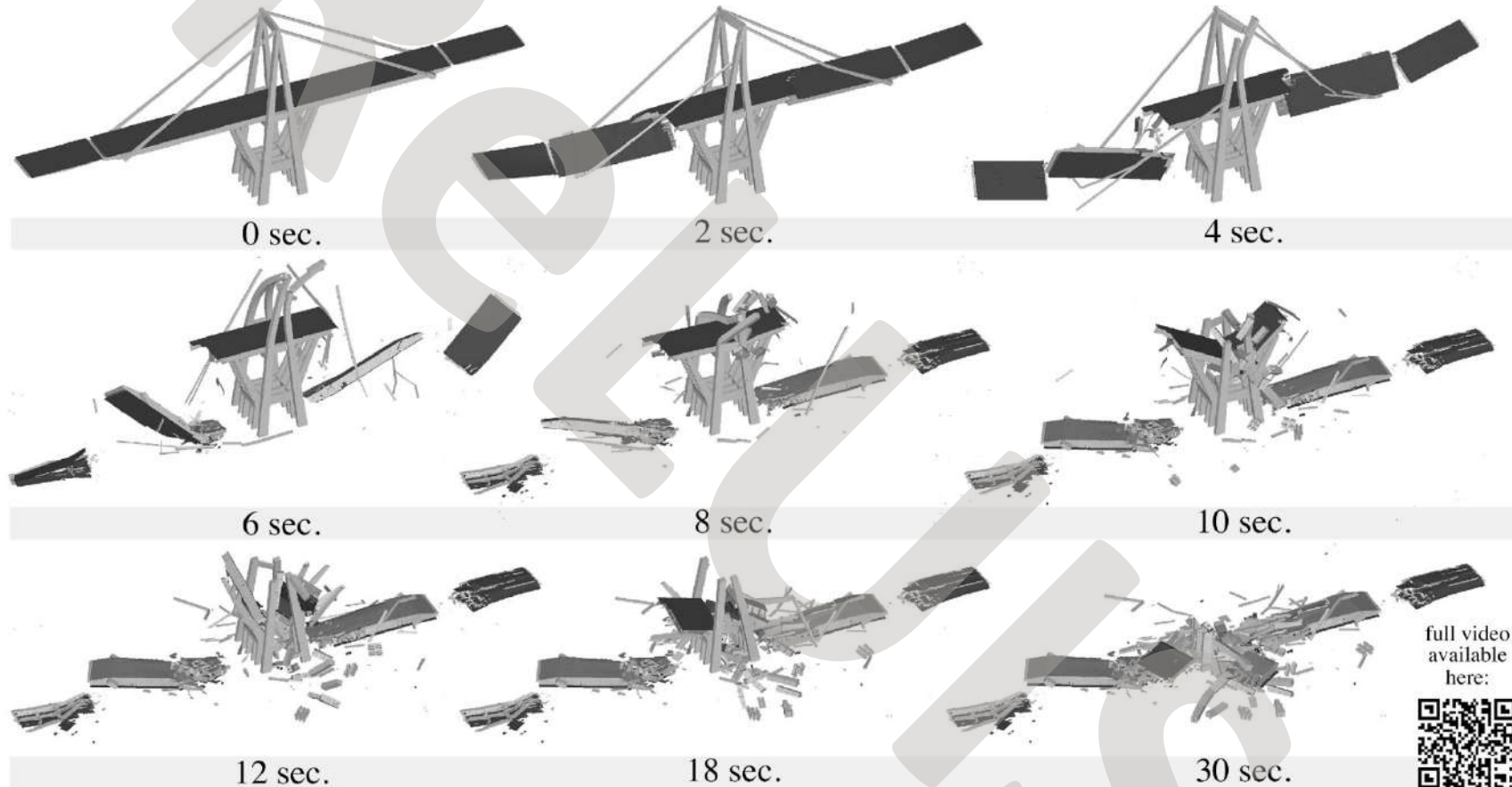


Scenario 3 - Failure of the deck-stay or antenna-stay connections

- a) failure at interface between S-W stay and antenna (possibly related to fatigue in the tendons)
- a) sudden loss of connection between and the main deck (transverse link details unknown)

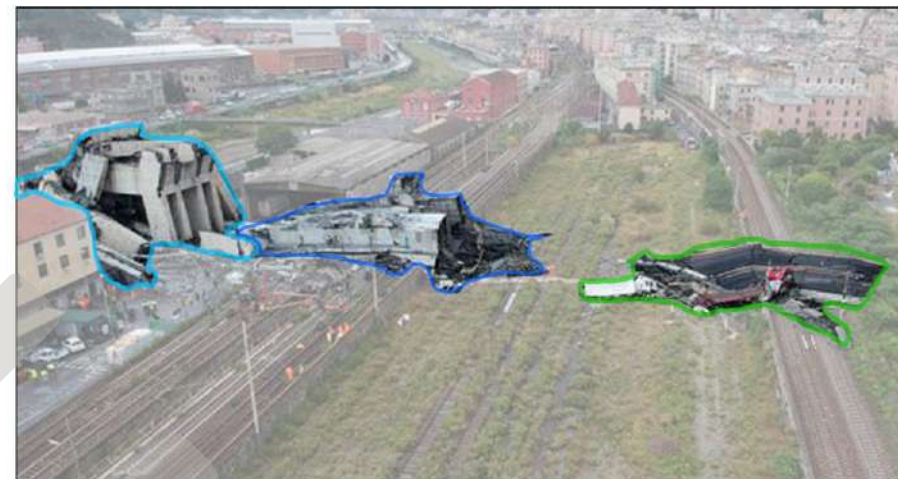
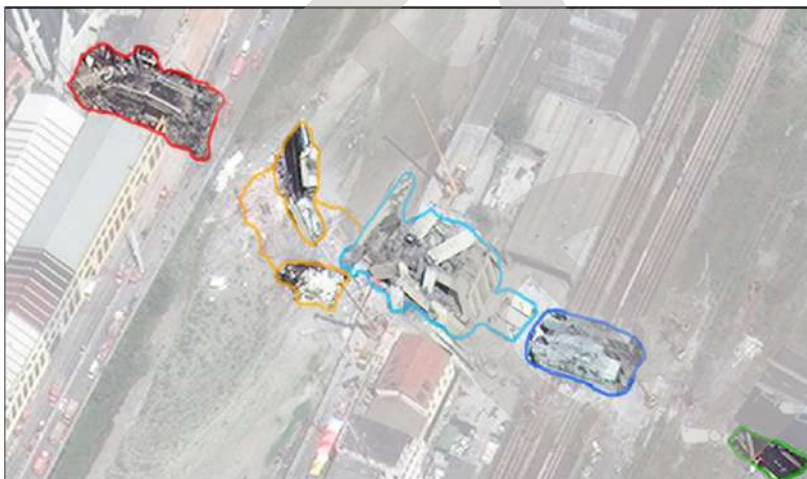


Scenario 3 - Failure of the deck-stay or antenna-stay connections

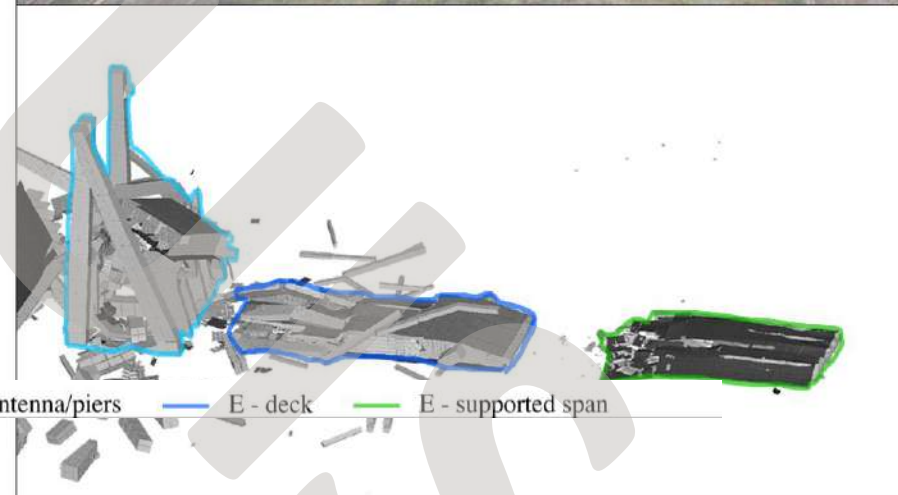
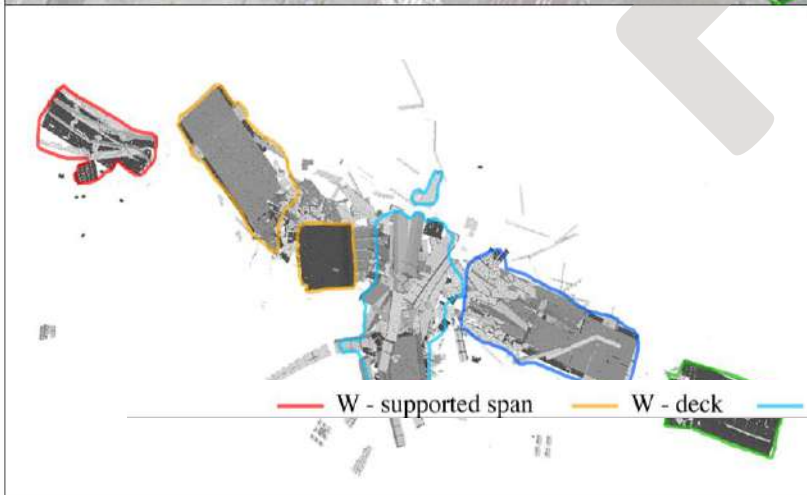




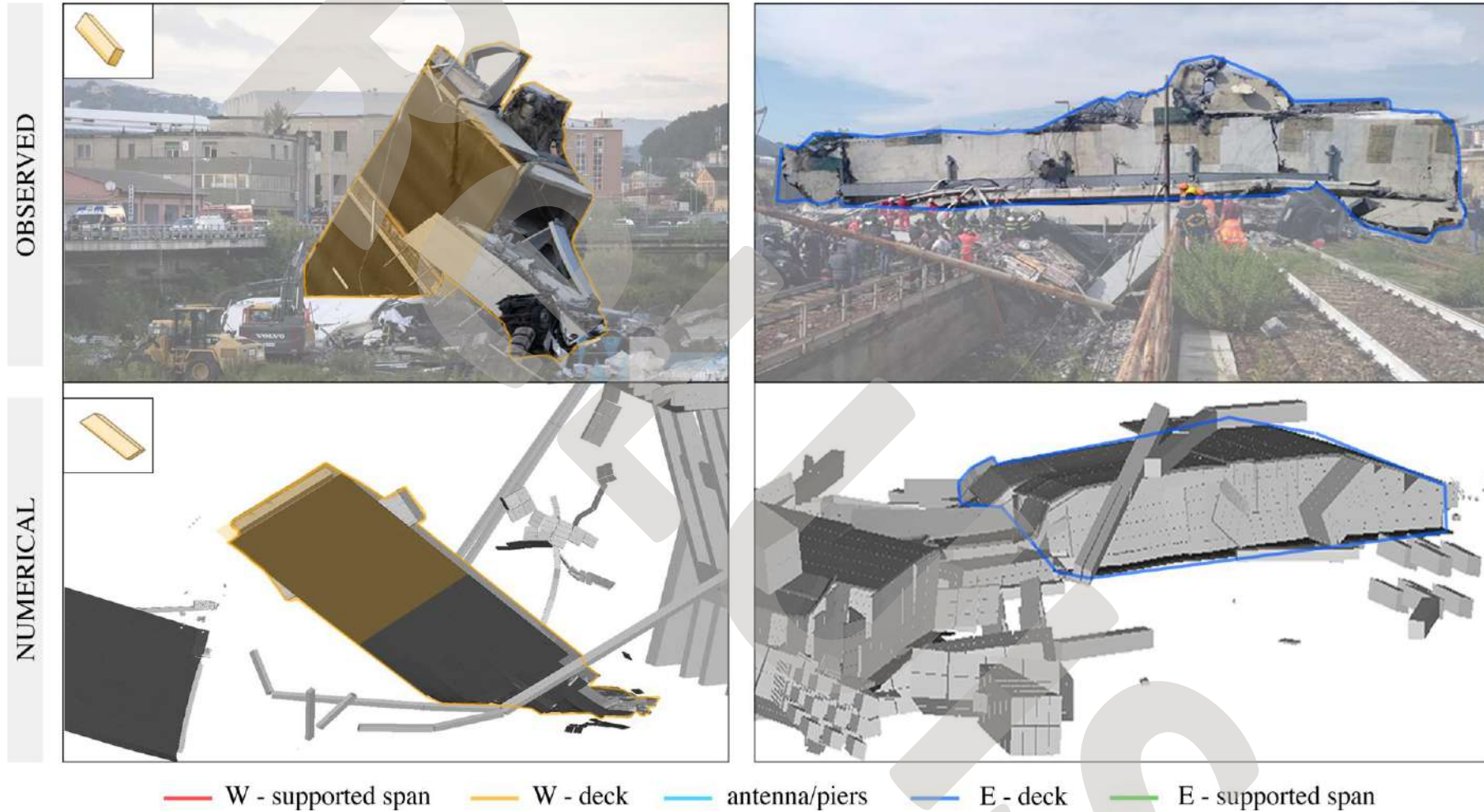
OBSERVED



NUMERICAL



— W - supported span — W - deck — antenna/piers — E - deck — E - supported span



La zona dello strallo in cui ha avuto origine il collasso è nota.

Consideriamo il confronto tra domanda e capacità in una sezione approssimativamente corrispondente.

Domanda

Peso proprio

Miglior predizione 25,4 MN

$\sigma \approx 0,1$ MN

Probabilità 99,7 % tra 25,1 e 25,7 MN

Traffico

Miglior predizione 1,5 MN

$\sigma \approx 0,5$ MN

Probabilità 99,7 % tra 0 e 3,0 MN

Combinati

Miglior predizione 25,4 MN

$\sigma \approx 0,51$ MN

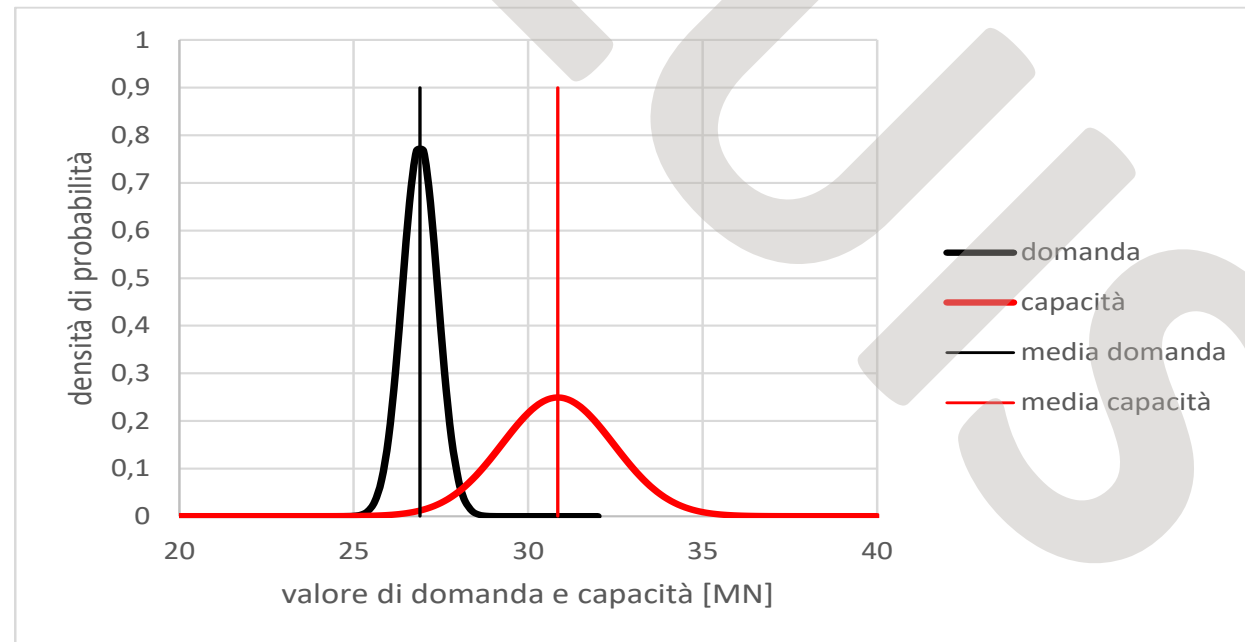
Probabilità 99,7 % tra 25,4 e 28,4 MN

Capacità

Miglior predizione 30,85 MN

$\sigma \approx 1,7$ MN

Probabilità 99,7 % tra 25,7 e 35,9 MN



Situazione originale

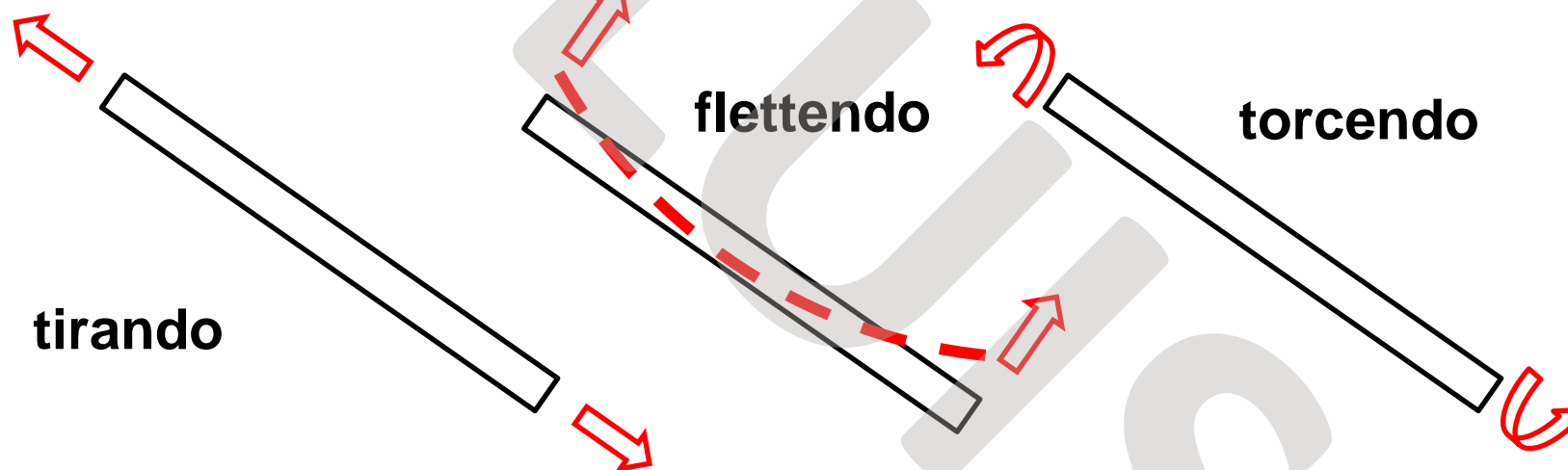
**Situazione indotta dalla corrosione
prodottasi a causa del difetto
nascosto**

**Enorme rilevanza del difetto nascosto
nell'avvicinare domanda e capacità**

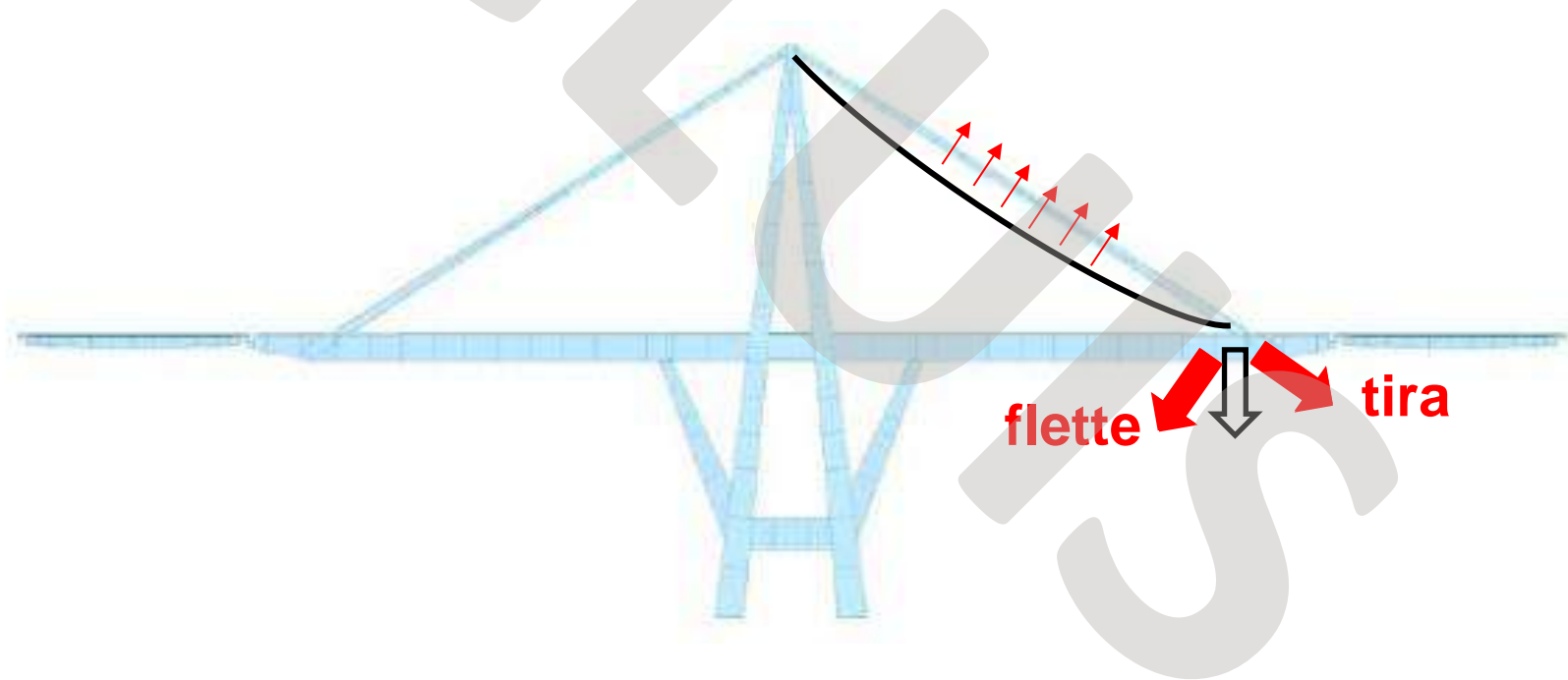


Confronto domanda capacità in termini di caratteristiche di sollecitazione pluriassiali

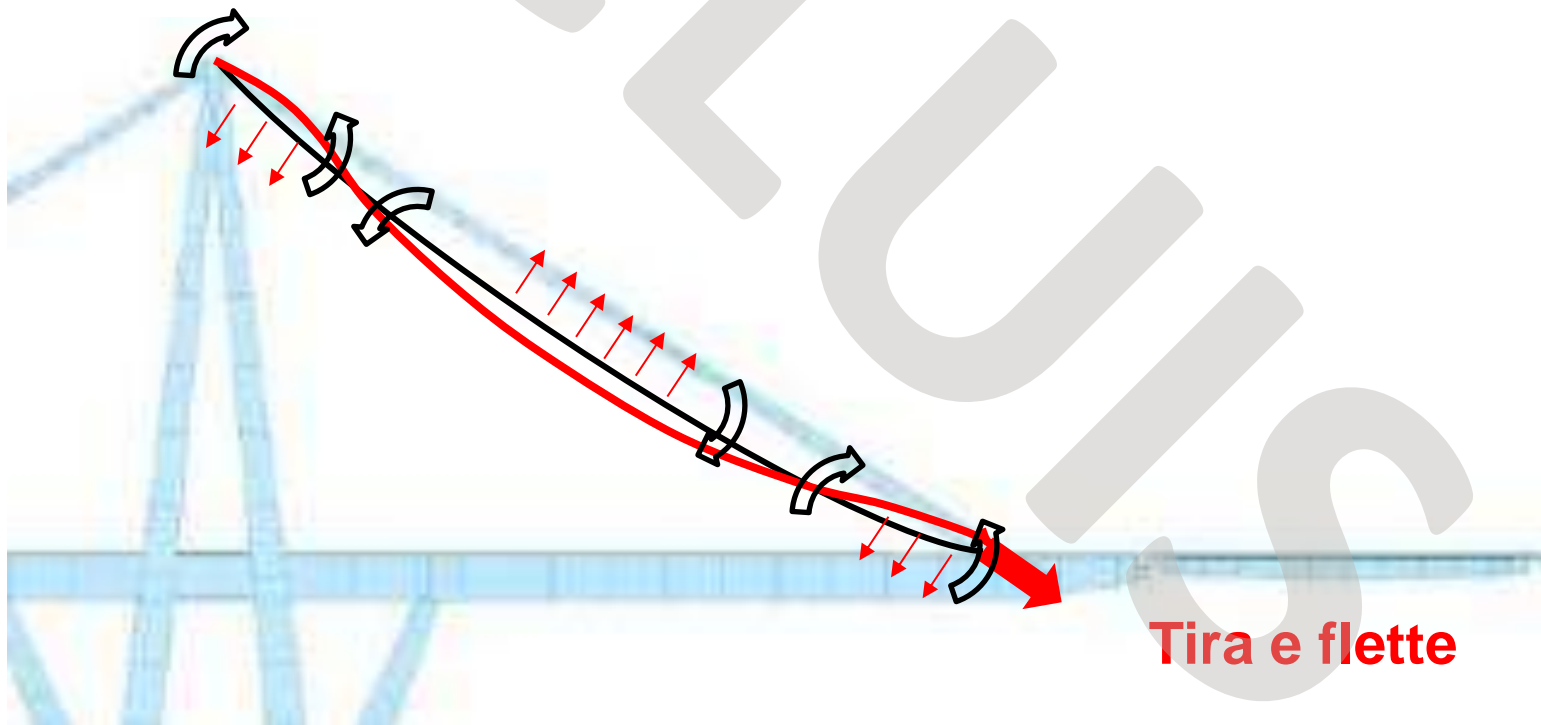
È esperienza comune: è possibile rompere un'asta in molti modi



Confronto domanda capacità in termini di caratteristiche di sollecitazioneE pluriassiali



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Tira e flette

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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Carichi verticali	Tirano e flettono (in misura dipendente dalla posizione)
Vento orizzontale	Flette attorno all'asse verticale Flette attorno all'asse orizzontale
Vento verticale	Può indurre trazione, flessione, torsione
Temperatura	Può indurre trazione, flessione, torsione
Carichi asimmetrici	Possono indurre trazione, flessione, torsione

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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

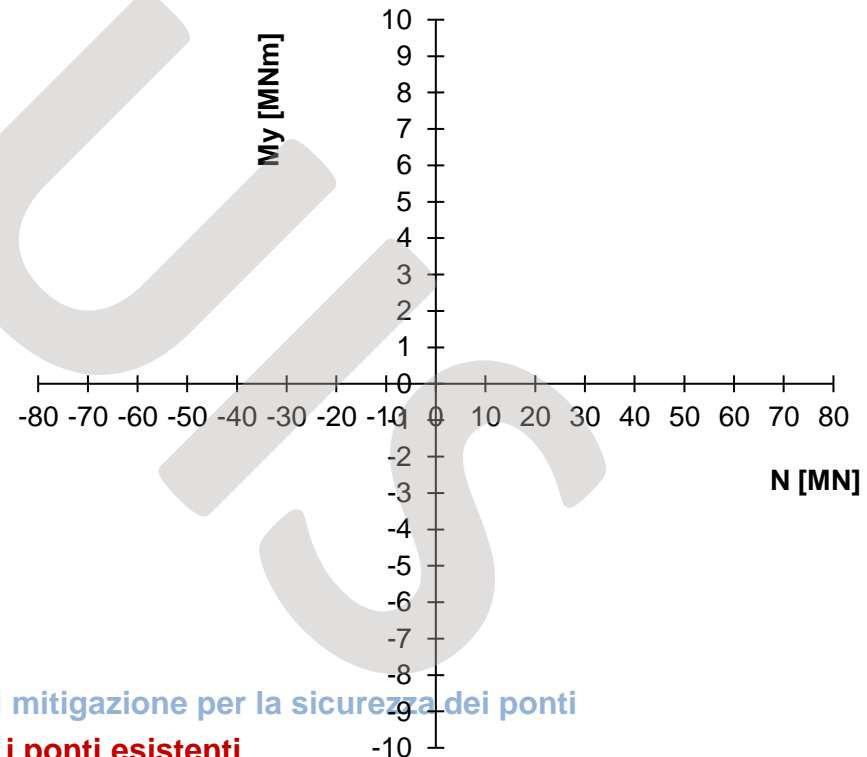
Ipotesi di carico	N [MN]		M _y [MNm]		M _z [MNm]	
	Modello semplificato	Modello completo	Modello semplificato	Modello completo	Modello semplificato	Modello completo
Soli permanenti	25,143	25,437	0	0,850	0	0
Accidentali e vento	26,188	26,796	0	1,482	0	0,895
Transito da 223 t	26,474	27,124	0	1,745	0	0

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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si combinano sollecitazioni diverse?

Per ora, consideriamo solo N e M_y



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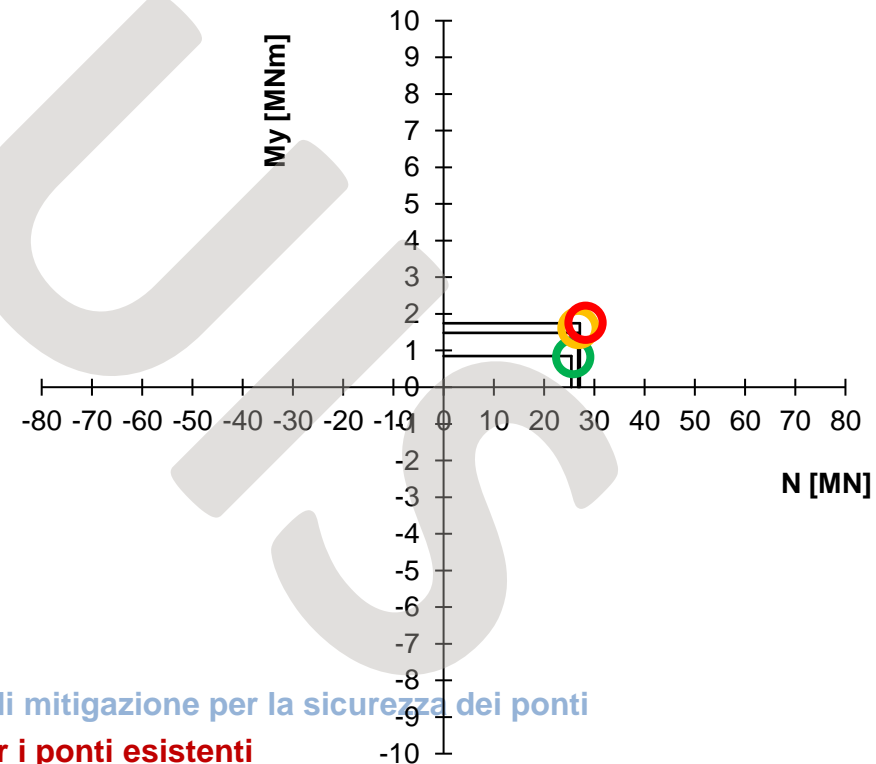
CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si combinano sollecitazioni diverse?

Per ora, consideriamo solo N e M_y

Il momento agente per i permanenti è basso

Al crescere dei carichi il momento cresce più rapidamente del tiro



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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si calcola la capacità in termini biassiali?

**Equilibrio tra
risultante delle
azioni esterne e
delle tensioni
interne**

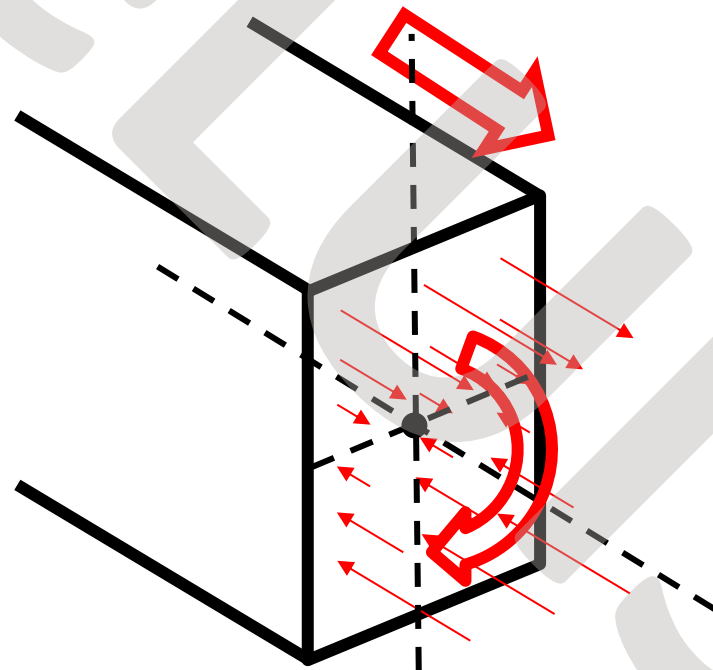


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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si calcola la capacità in termini biassiali?

Equilibrio tra risultante delle azioni esterne e delle tensioni interne



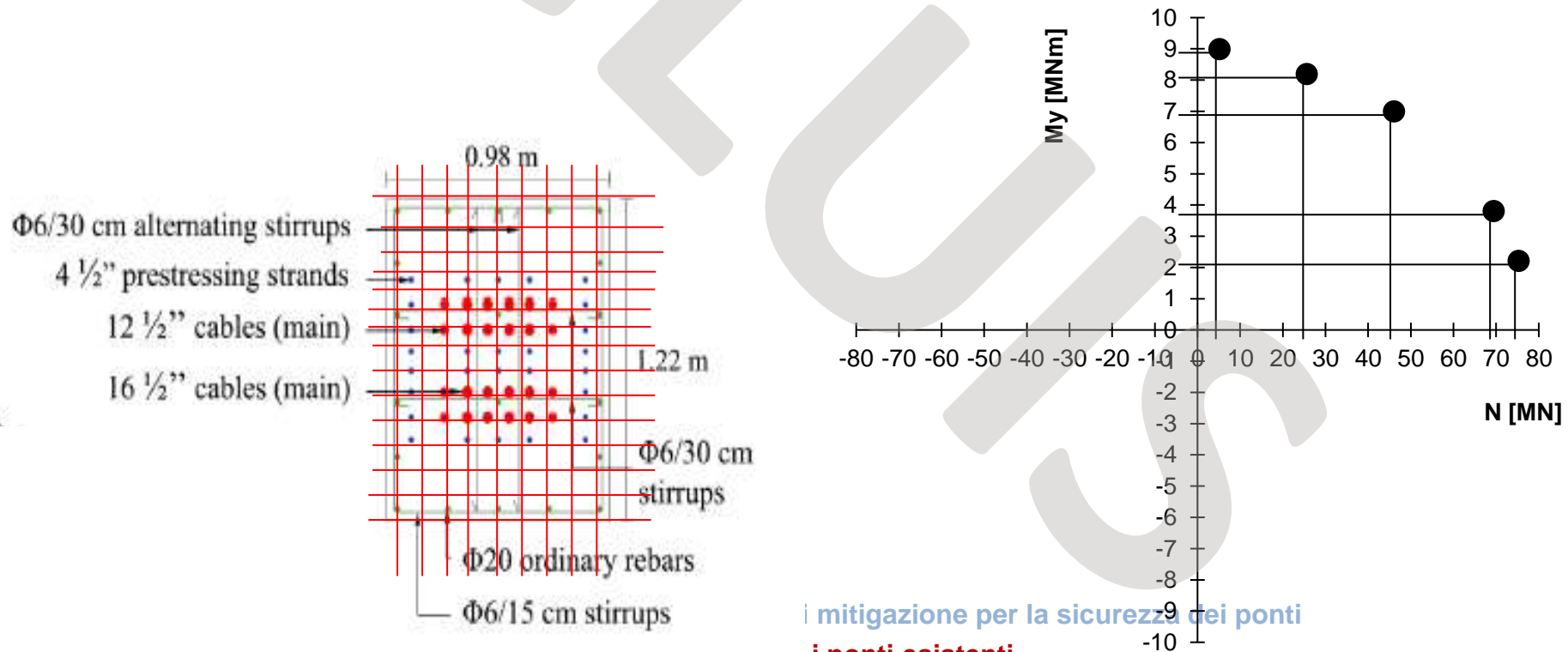
Momento flettente attorno all'asse orizzontale M_y

Le tensioni variano linearmente sull'altezza

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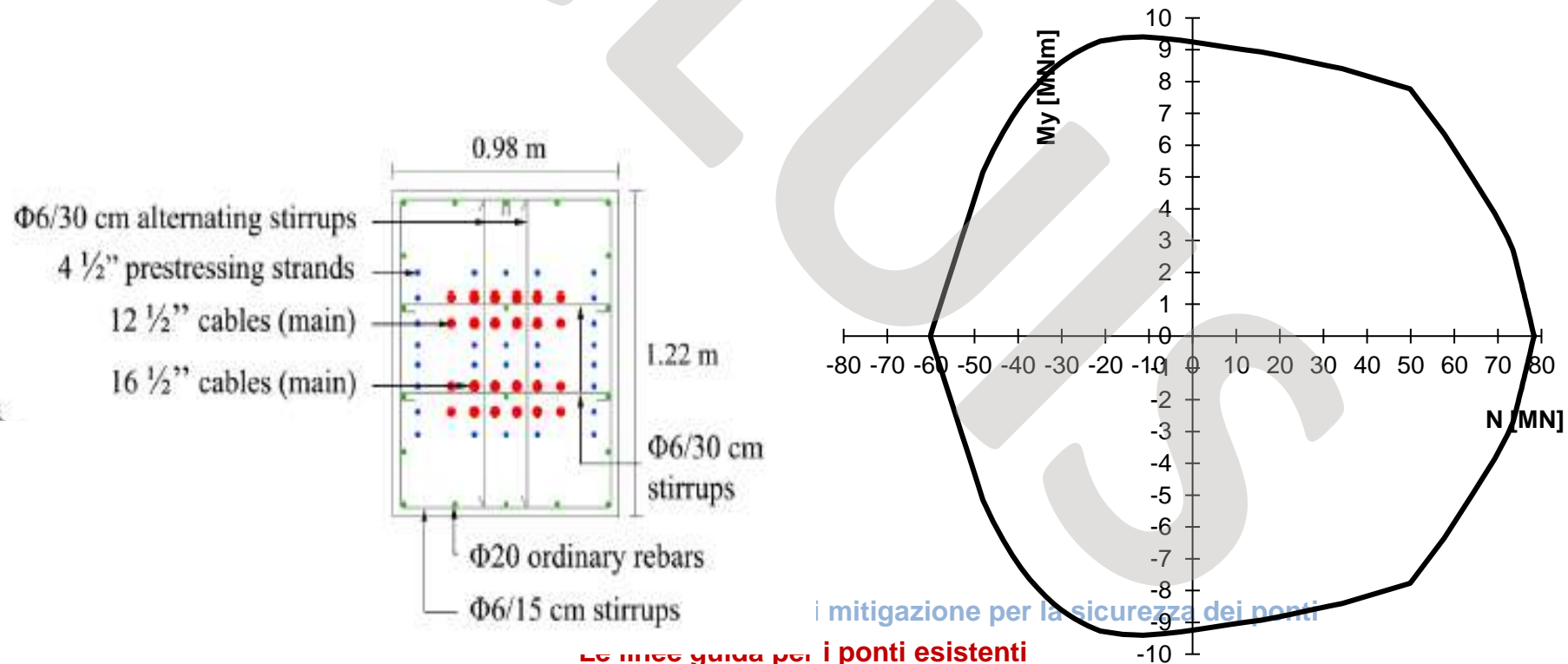
Come si calcola la capacità in termini biassiali?



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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si calcola la capacità in termini pluriassiali?



la mitigazione per la sicurezza dei ponti

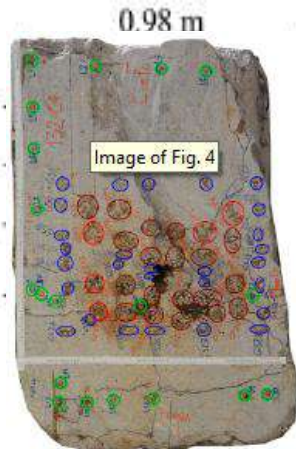
Le linee guida per i ponti esistenti

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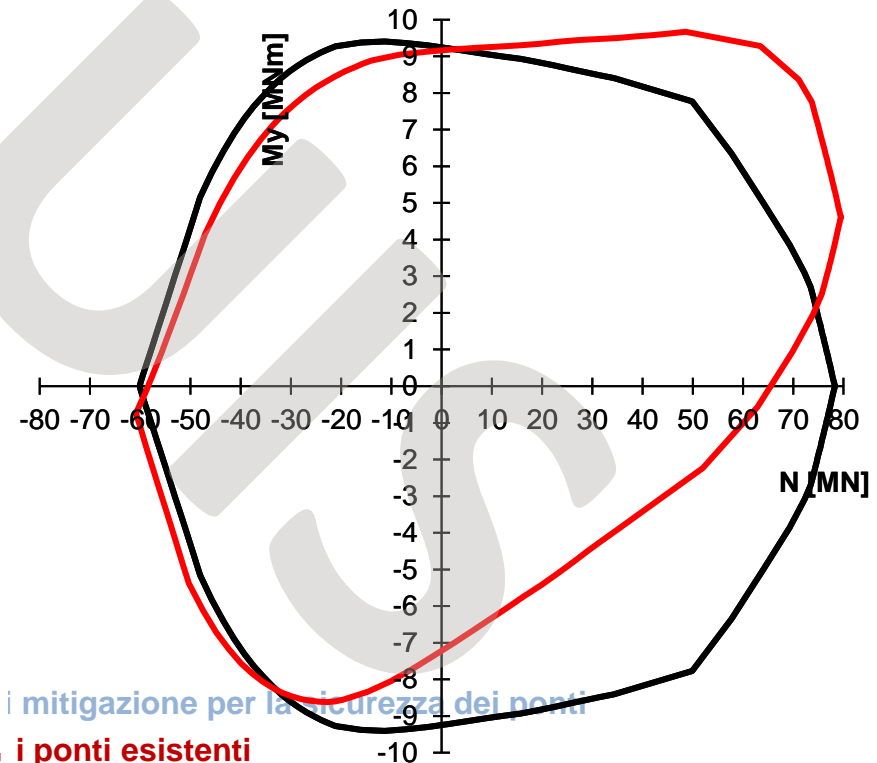
CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

La sezione con la reale distribuzione dei cavi

- Φ6/30 cm alternating stirrups
- 4 1/2" prestressing strands
- 12 1/2" cables (main)
- 16 1/2" cables (main)



- Φ6/30 cm stirrups
- Φ20 ordinary rebars
- Φ6/15 cm stirrups



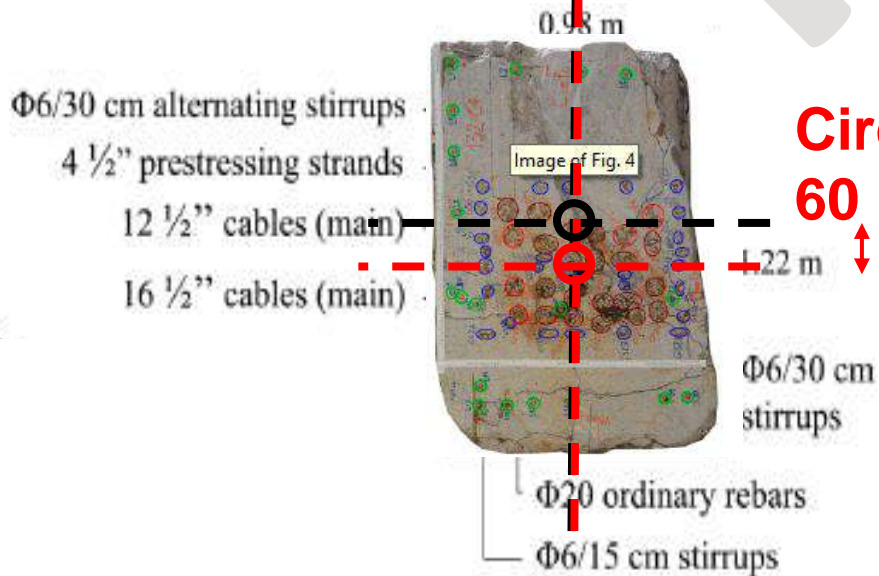
mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti

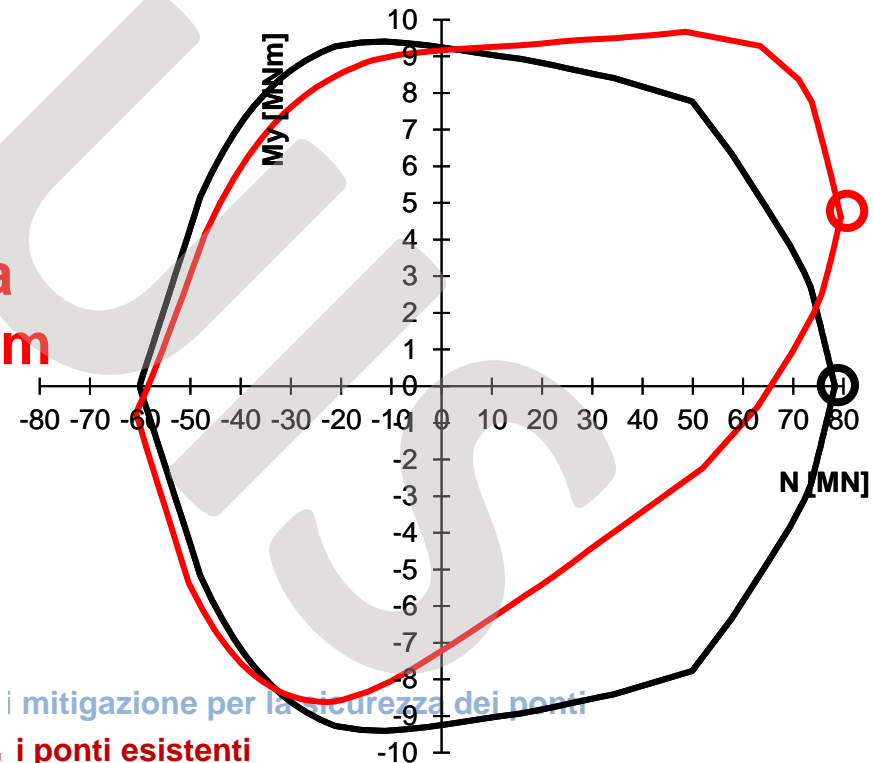
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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

La sezione con la reale distribuzione dei cavi



Circa 60 mm



mitigazione per la sicurezza dei ponti

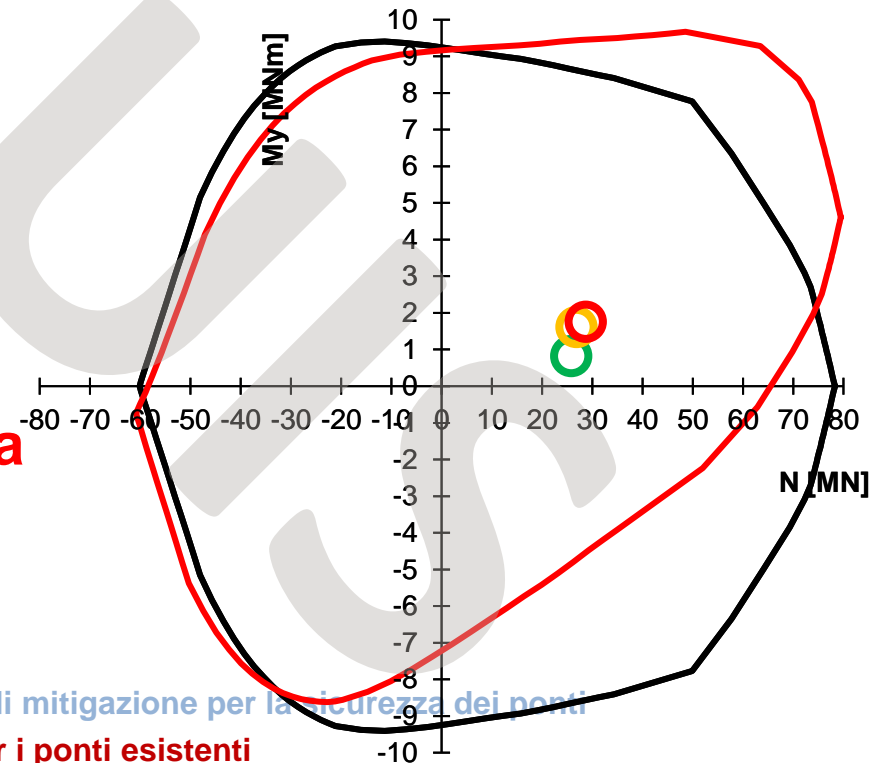
Le linee guida per i ponti esistenti

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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Questa variazione del dominio di resistenza (capacità) avrebbe avuto rilevanza in assenza di corrosione?

**Assolutamente no.
La distanza tra livelli di
domanda e livelli di capacità era
enorme.**

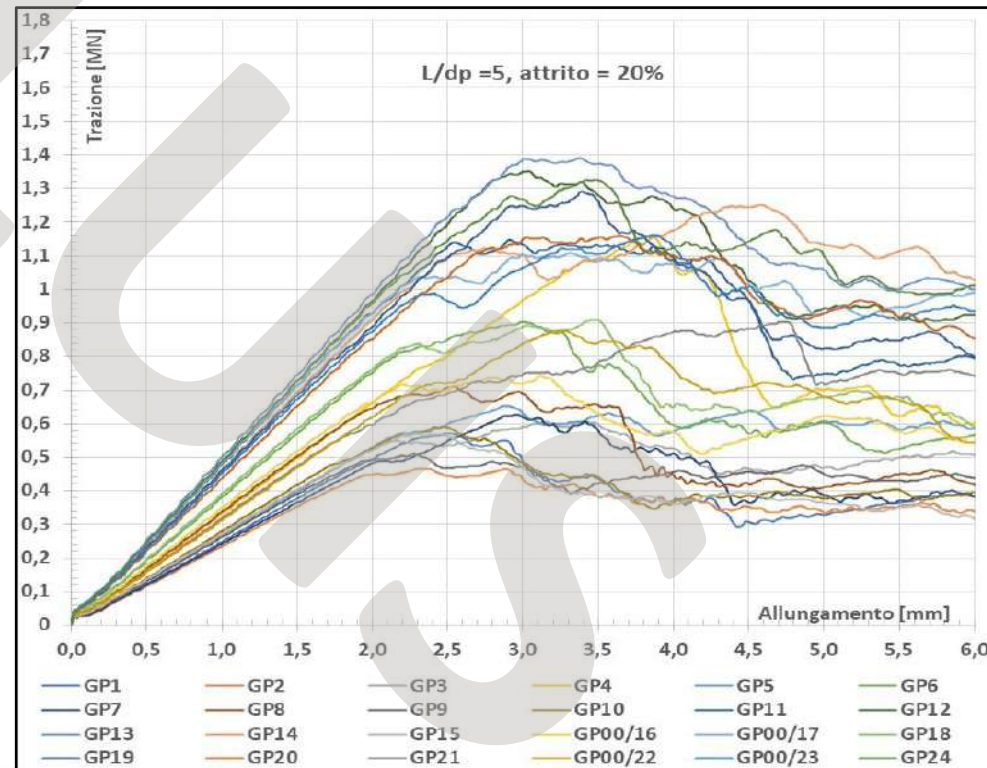
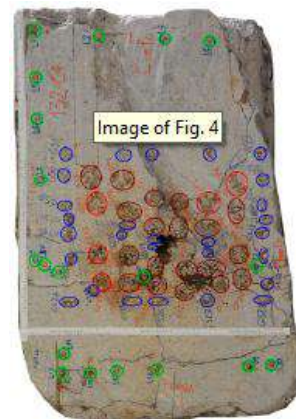


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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come cambia il dominio di resistenza per la corrosione?

Occorre associare ad ogni fibra la migliore stima delle sue caratteristiche di resistenza e capacità deformativa



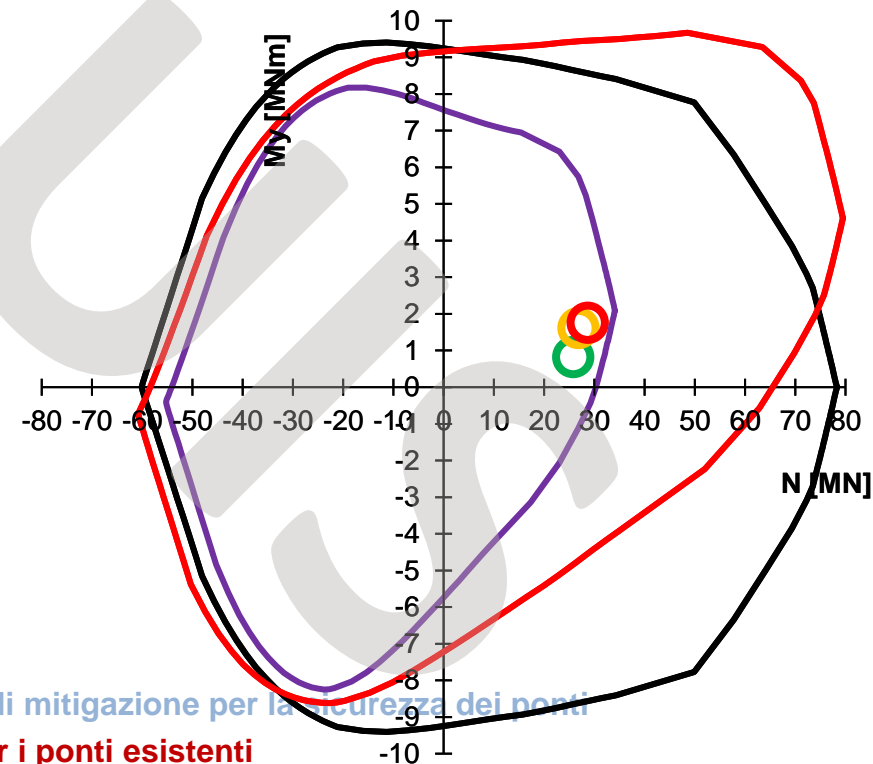
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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come cambia il dominio di resistenza per effetto della corrosione?

Occorre associare ad ogni fibra la migliore stima delle sue caratteristiche di resistenza e capacità deformativa

La distanza tra capacità e domanda è radicalmente diminuita



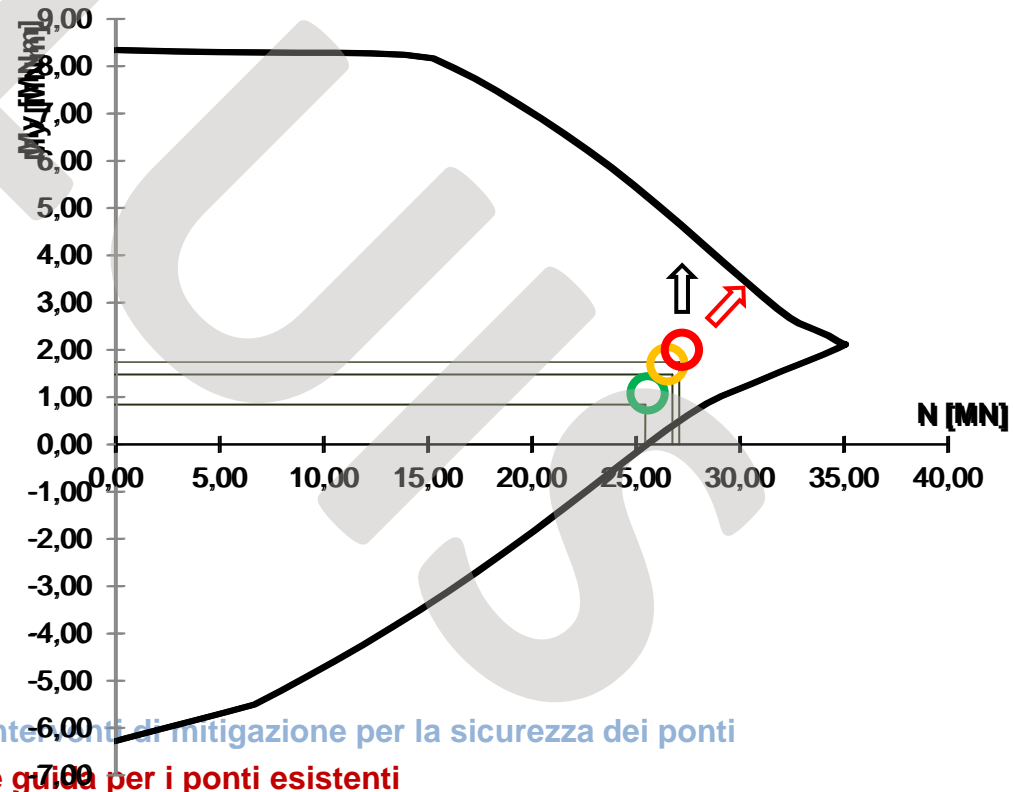
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Vediamo meglio

Se crescono i carichi sul ponte

Se cresce il solo momento (vento verticale – downburst)

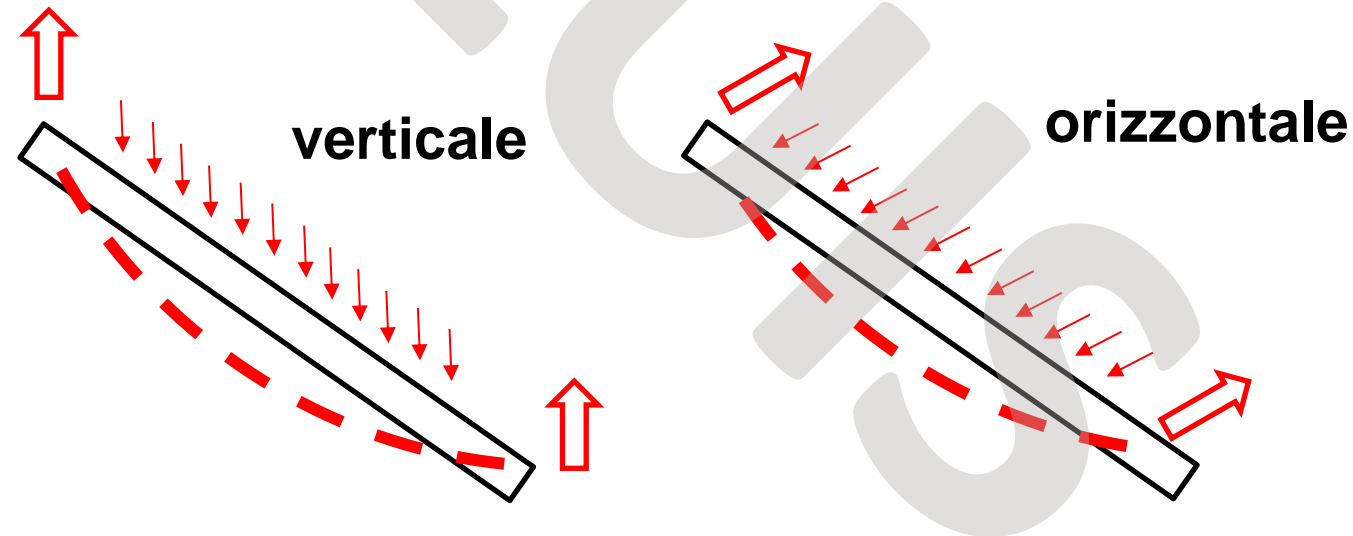


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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

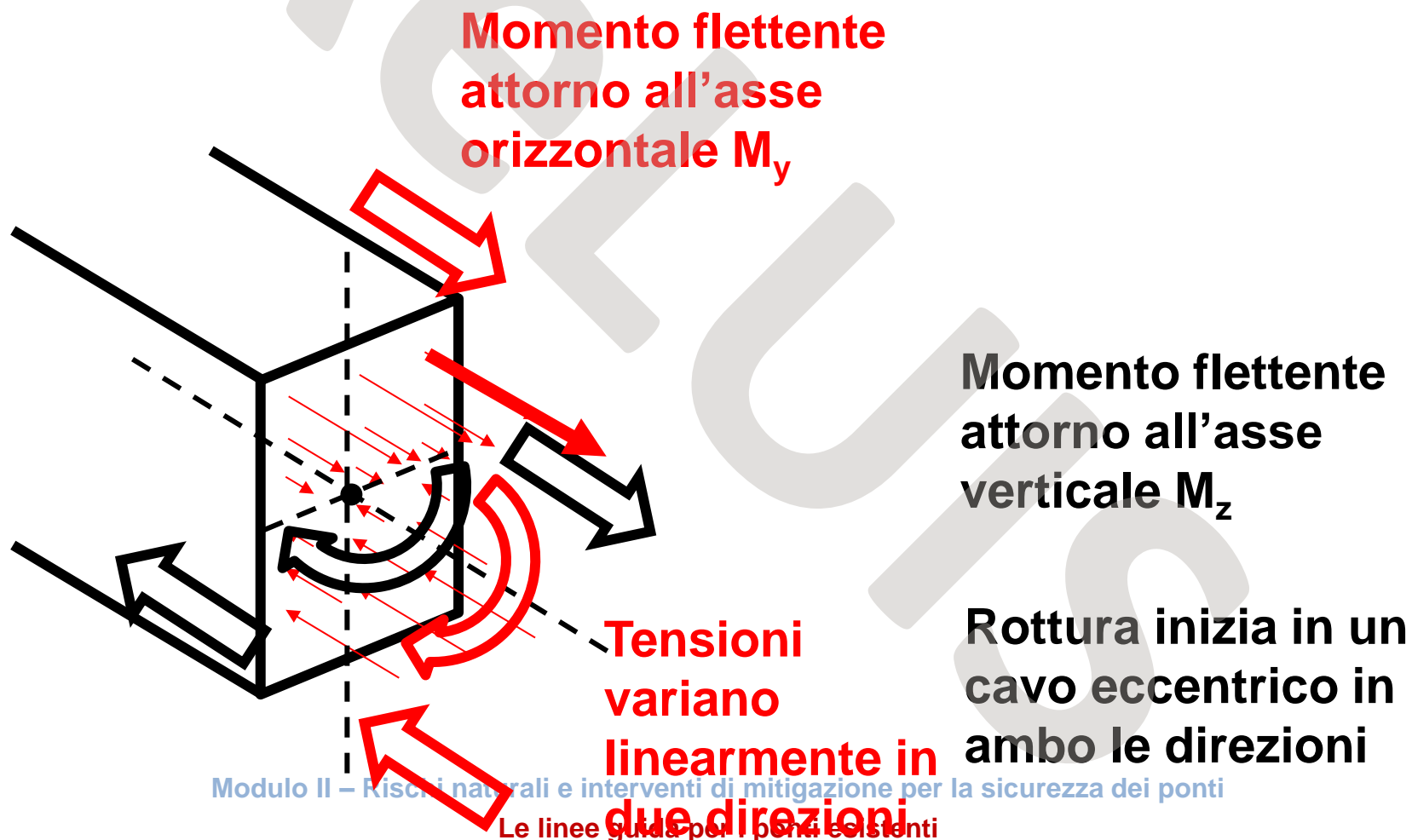
Cosa succede se aggiungo un forte vento orizzontale?

Lo strallo viene inflesso: sia nella direzione verticale, sia nella direzione orizzontale.



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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI



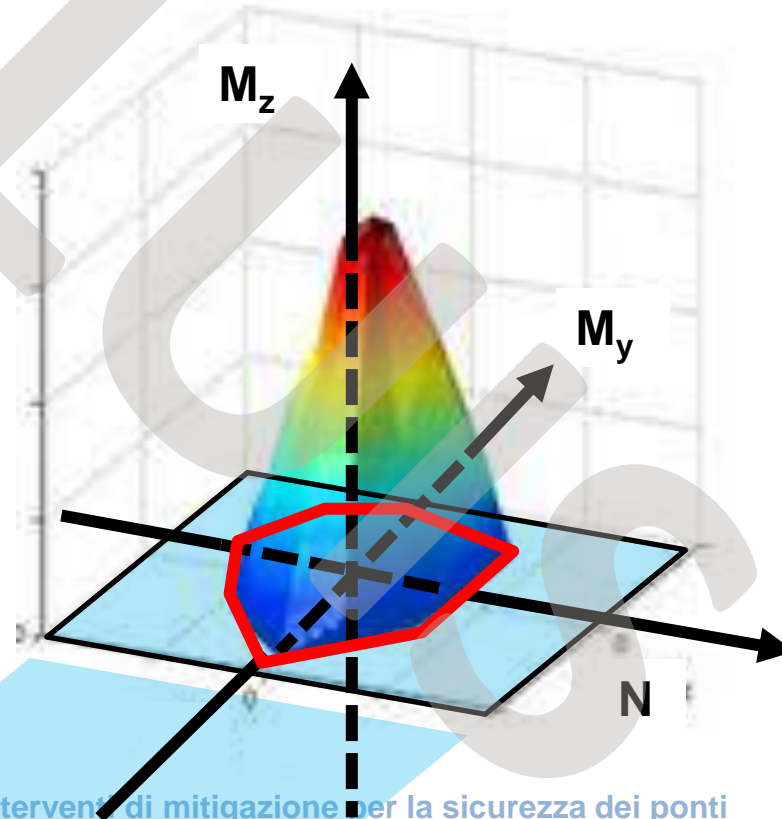
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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Come si calcola la capacità in termini pluriassiali?

$M_z = 0$

Dominio di rottura N - M_y

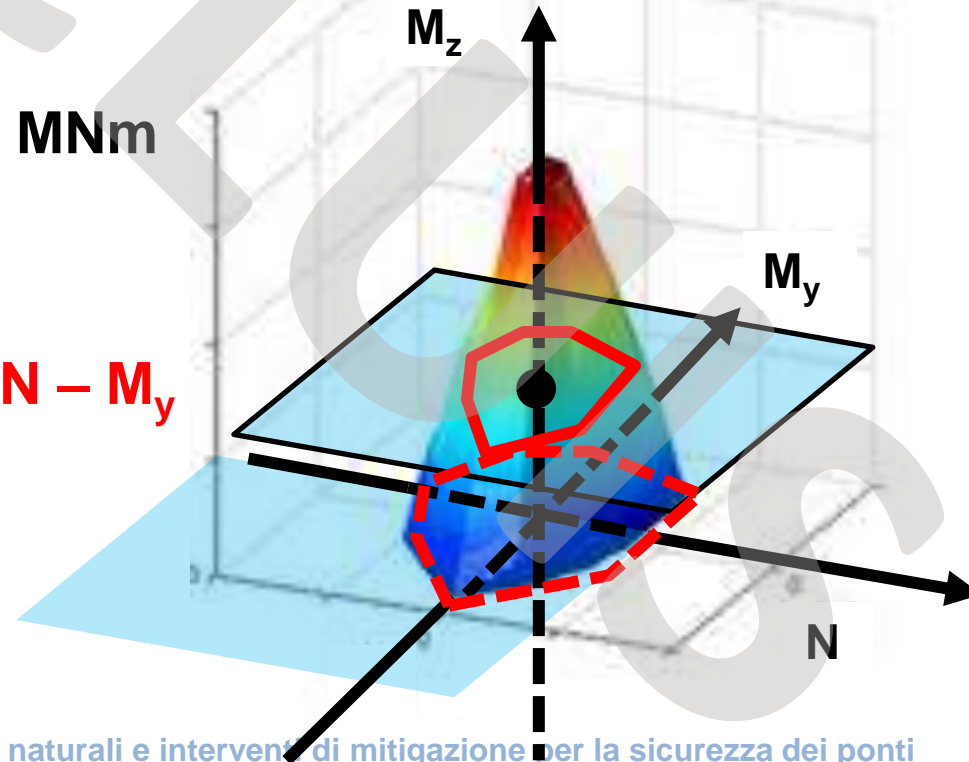


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$M_z \neq 0$ e.g.: $M_z = 0,9 \text{ MNm}$

**Il dominio di rottura $N - M_y$
rimpicciolisce**



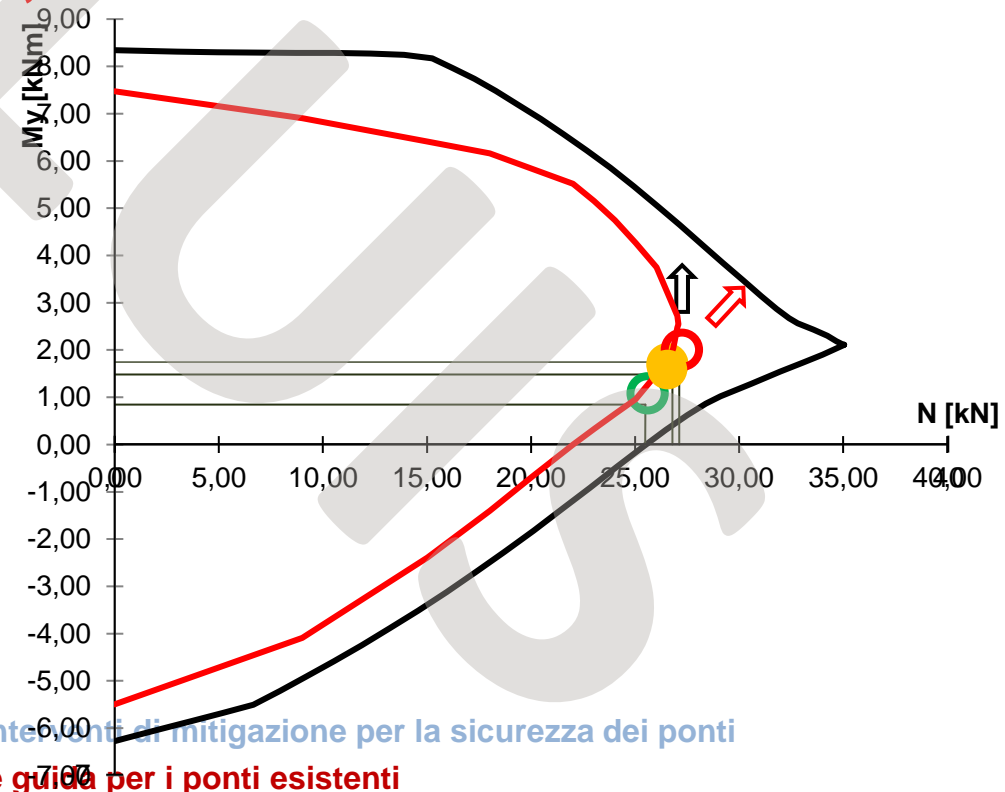
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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Il dominio $N - M_y$ per $M_z = 0$

Il dominio $N - M_y$ per $M_z = 0,9 \text{ MN}$

Il caso peso proprio e traffico del 14 agosto è sul confine



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CONFRONTO DOMANDA CAPACITÀ IN TERMINI DI CARATTERISTICHE DI SOLLECITAZIONE PLURIASSIALI

Un momento flettente orizzontale di 0.9 MNm nella zona della sezione critica corrisponde a velocità del vento dell'ordine di 70 km/h

L'innescò del crollo può avere avuto origine da una forte componente di vento orizzontale

La possibile combinazione con vento verticale e variazioni termiche non è nota, ma può avere contribuito all'innescò, portando a rottura progressivamente i cavi partendo da uno più esterno

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CONCLUSIONI AL CAPITOLO 10

Confrontando domanda e capacità considerando la sola azione assiale

- a) **La domanda del 18 luglio è stata certamente decisamente maggiore di quella del 14 agosto – di almeno 280 kN o 28 tonnellate**
- b) **Il crollo non può essere stato innescato dal procedere della corrosione in 27 giorni – la velocità era troppo bassa**
- c) **Domande superiori (almeno 27 MN) a quella del 28 luglio si sono ripetute con elevata frequenza in condizioni di traffico intenso**
- d) **La capacità media è da stimarsi in circa 31,5 MN**

prof. ing. **Gianmichele Calvi**

CONCLUSIONI AL CAPITOLO 10

Confrontando domanda e capacità considerando il momento attorno all'asse orizzontale


- a) **La distanza tra i punti che identificano la domanda ed il dominio della capacità aumenta, cioè aumenta la sicurezza**
- b) **Ciò è dovuto allo spostamento verso il basso delle armature nella zona critica, per cui la presenza di un momento flettente è favorevole**
- c) **Il collasso non poteva essere innescato da maggiori carichi verticali**

prof. ing. **Gianmichele Calvi**

CONCLUSIONI AL CAPITOLO 10

Confrontando domanda e capacità considerando il momento attorno all'asse orizzontale ed il momento attorno all'asse verticale

- a) La presenza di forte vento ha l'effetto di ridurre il dominio della capacità
- b) Il punto caratterizzante l'azione indotta dai carichi verticali è in prossimità del dominio ridotto da un momento compatibile con una velocità del vento di circa 70 km/h
- c) In tali condizioni la presenza di altre azioni (vento verticale, variazione di temperatura, azioni torsionali dovuti alla collocazione dei carichi) e modestissime variazioni di capacità potevano indurre l'inizio del crollo

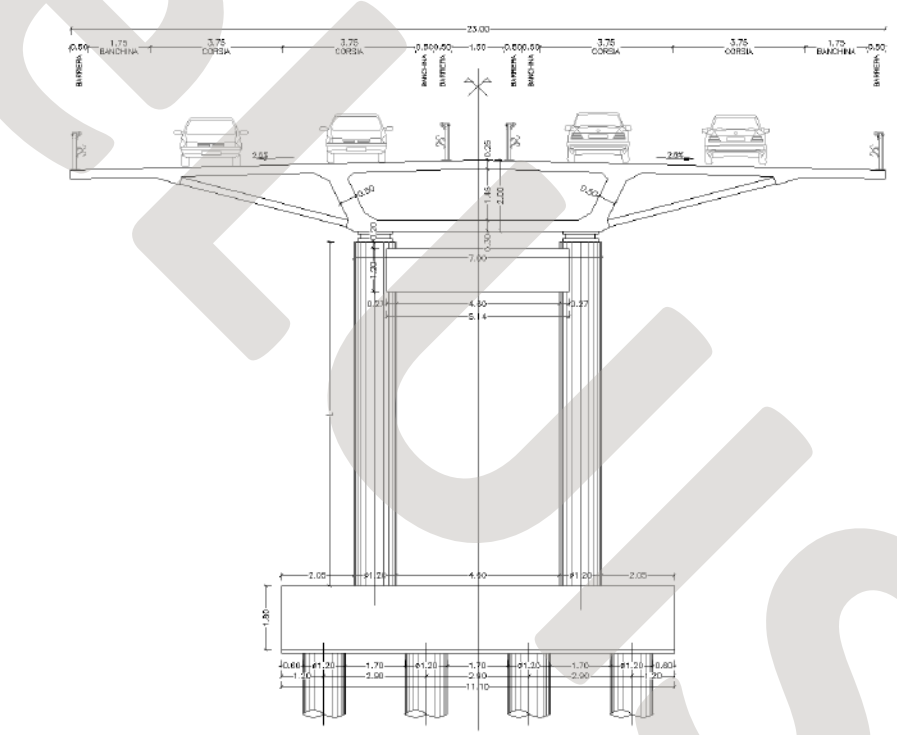
 **Identificazione
dinamica del
ponte Morandi**

Cosa avrebbe potuto dire

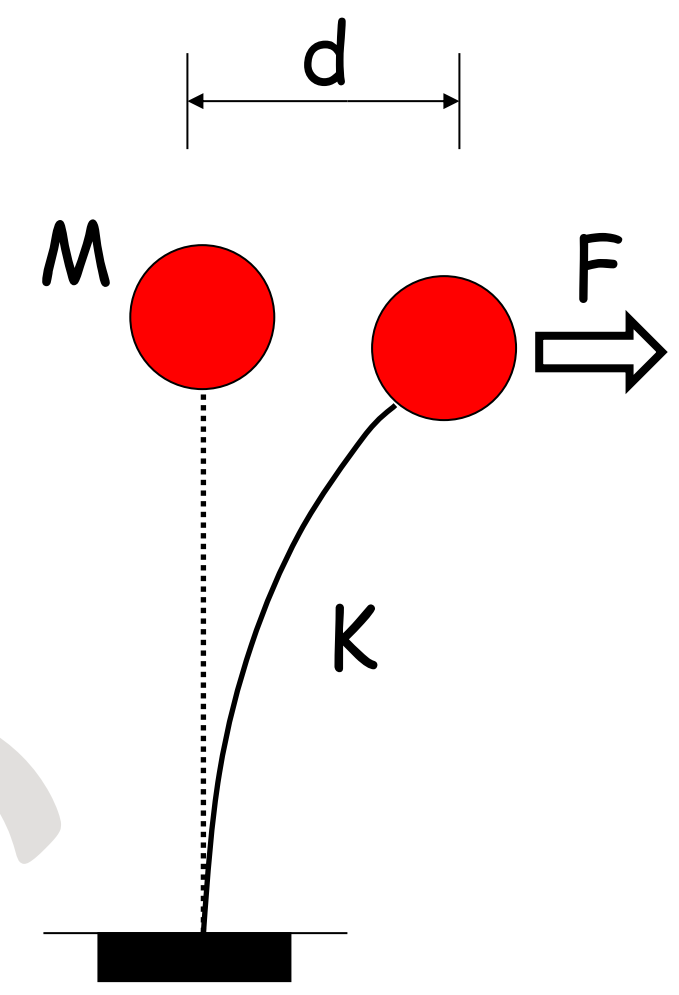
Cosa ha detto allora

Cosa dice oggi

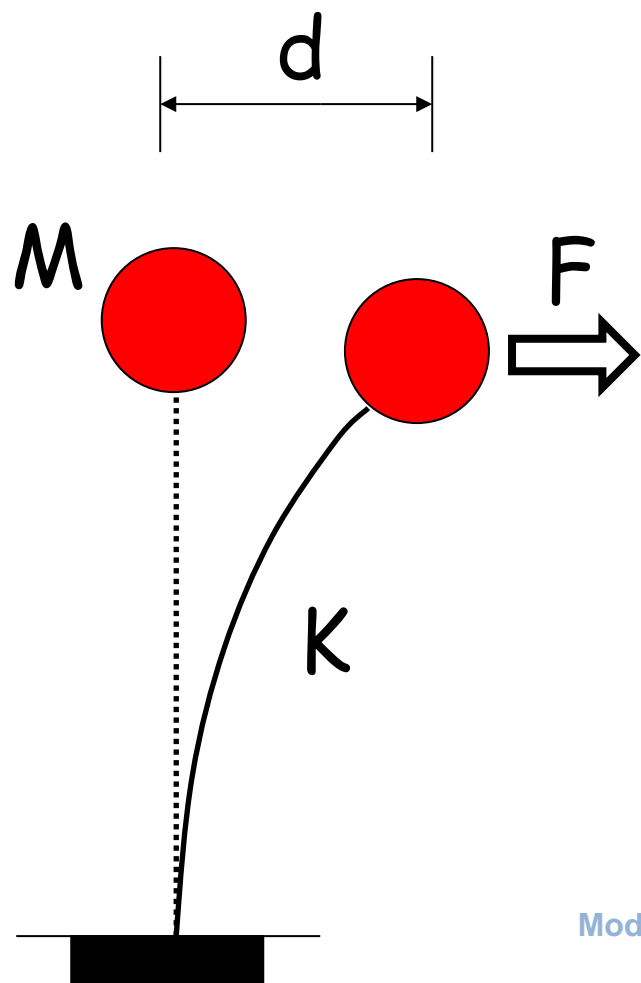
Cenni di dinamica delle strutture



Struttura e modello



Cenni di dinamica delle strutture



$$M = \frac{W}{g}$$

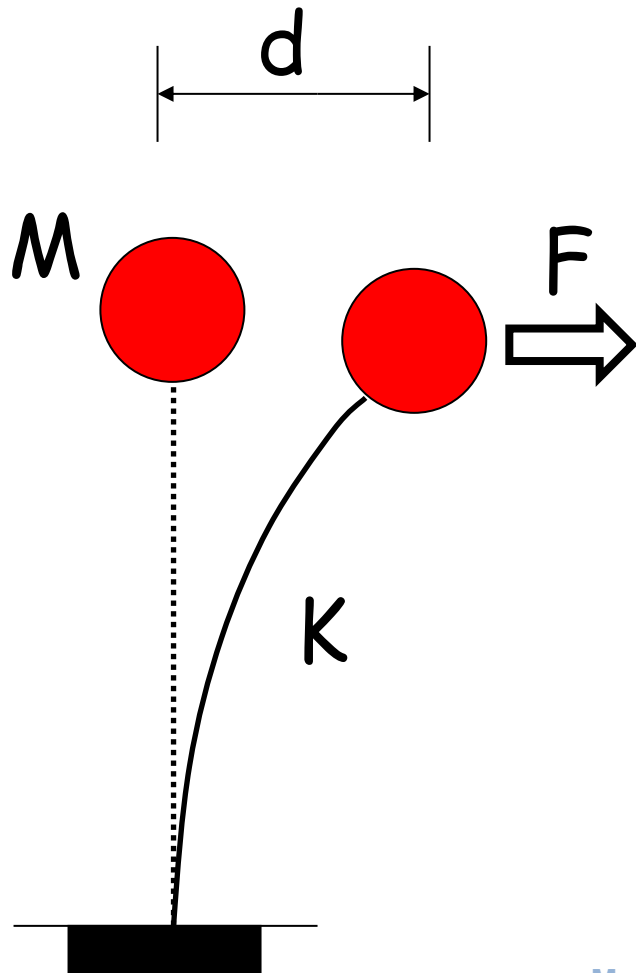
Cos'è la massa M

$W = \text{peso}$

$g = \text{costante di gravità}$

$g \approx 10 \text{ m/s}^2$

Cenni
di dinamica
delle strutture



Forza F necessaria
per indurre uno
spostamento d

$$K = \frac{F}{d}$$

Cos'è
la rigidezza K

K dipende da:

- proprietà del materiale
- geometria della struttura

Cos'è
la rigidezza K

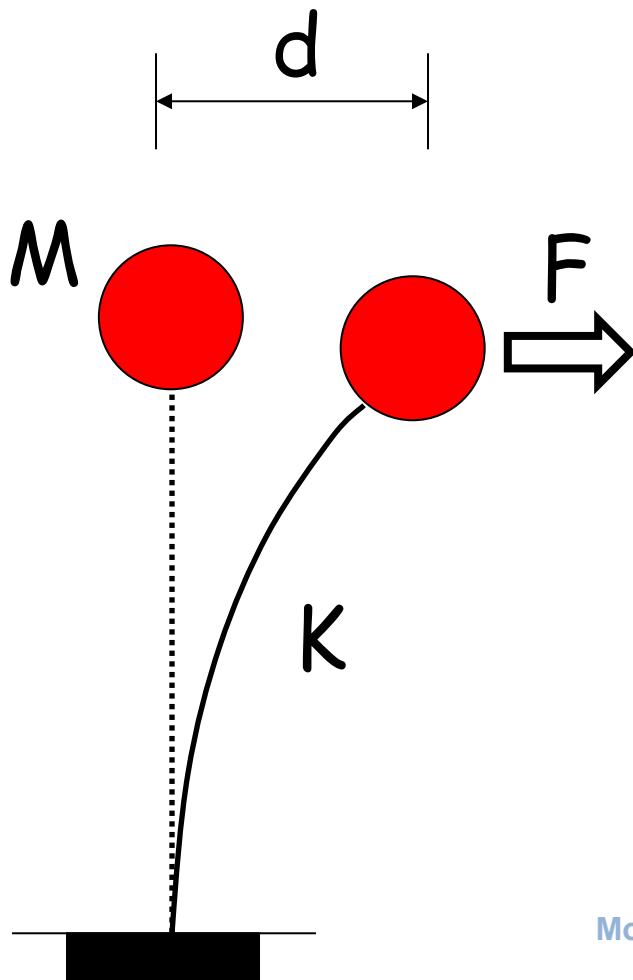
104

proprietà del materiale:
modulo elastico E

geometria della struttura:
altezza H

momento di inerzia della sezione
 $J = f(\text{larghezza } b, \text{ profondità } d)$

e.g.: $J = b d^3 / 12$

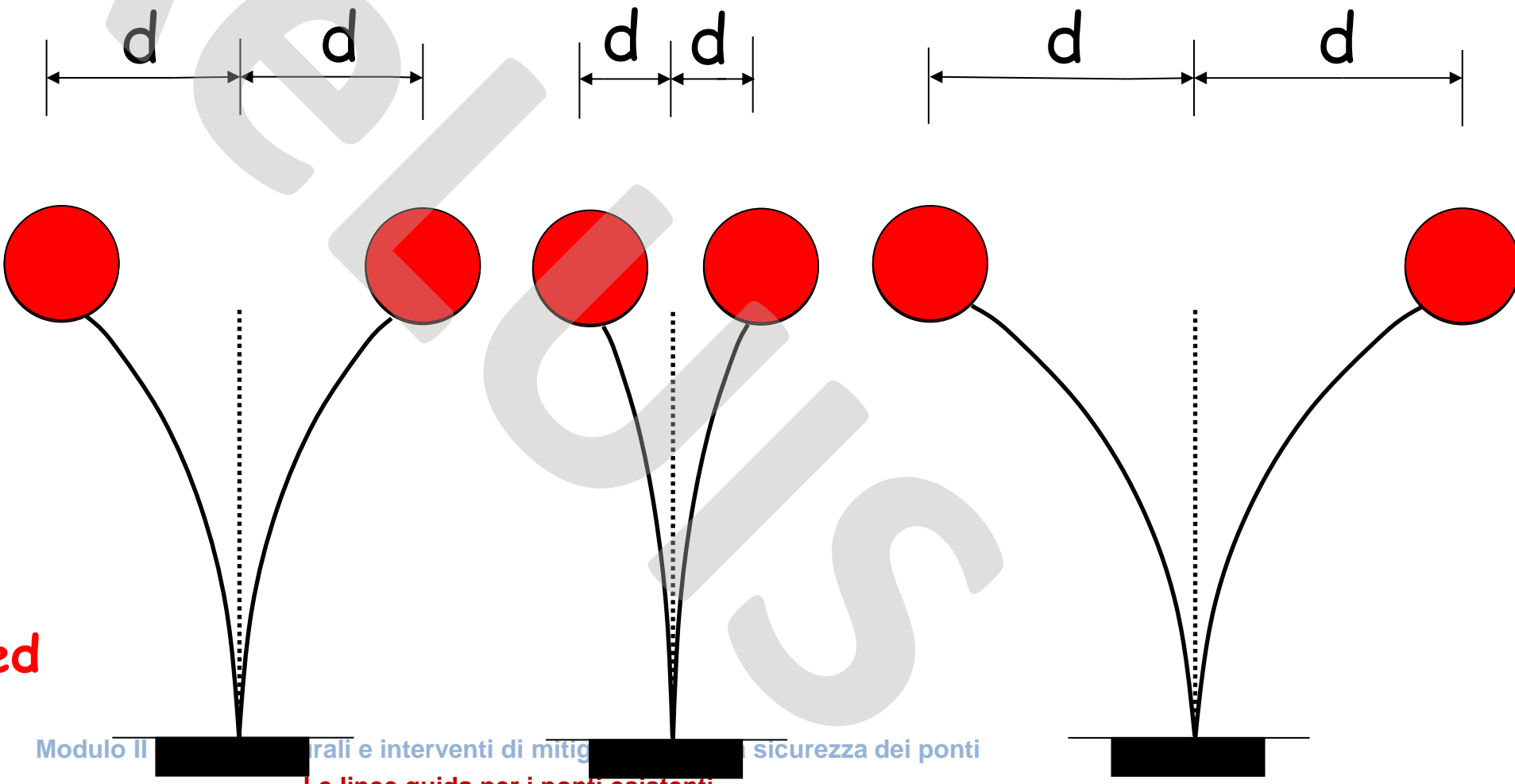


Cenni di dinamica delle strutture

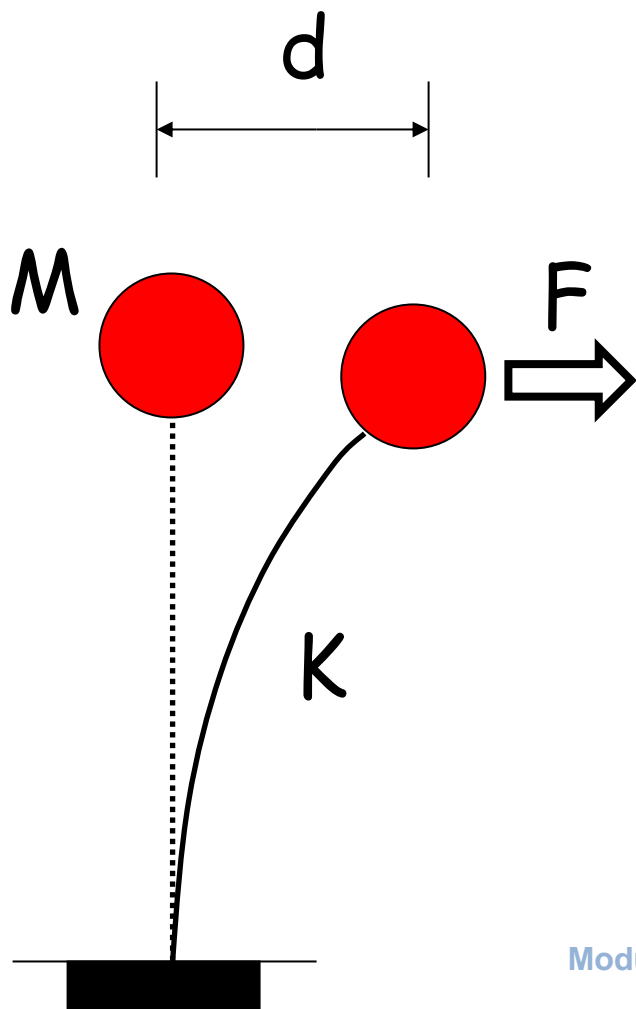
T è il tempo che la massa impiega ad andare da un'estremità all'altra ed a tornare al punto di partenza

T non dipende da d: la massa, spostata più lontano, si muove più velocemente ed impiega lo stesso tempo

Cos'è il periodo di vibrazione T



Cenni di dinamica delle strutture



M = massa

K = rigidezza

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

Cos'è il periodo
di vibrazione T

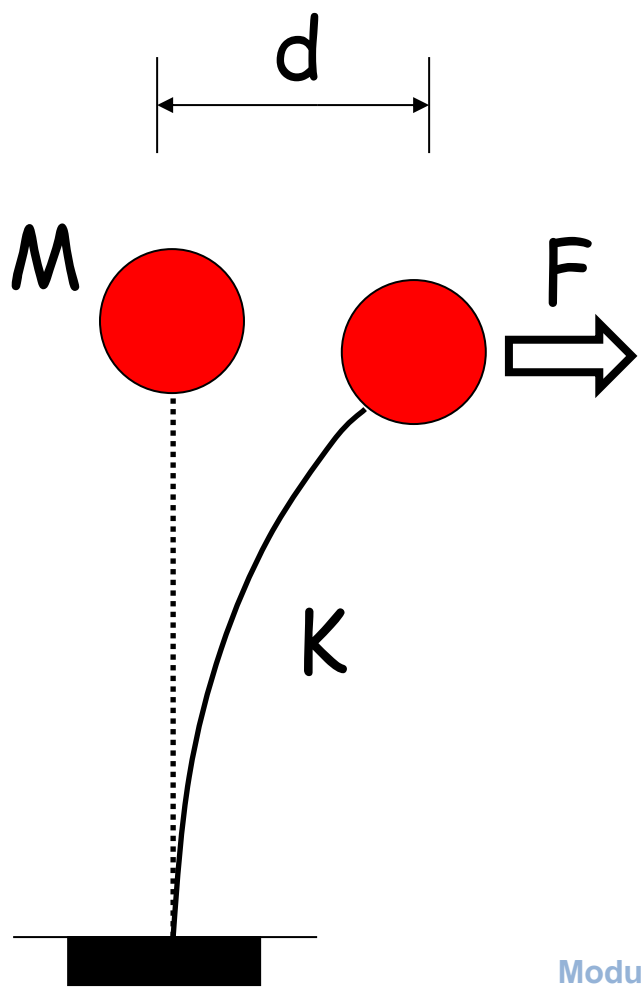
T non dipende
da F né da d

Frequenza

$$f = \frac{1}{T} \quad [\text{Hertz} = 1/\text{s}]$$

Numero di
periodi in 1 s

Risposta lineare elastica



$M =$ è costante

La risposta è lineare elastica se K
non varia

i.e.:

- Se E non cambia (il materiale non si deteriora)
- Se J non cambia (non c'è fessurazione)

Cenni di dinamica delle strutture

Identificazione
dinamica

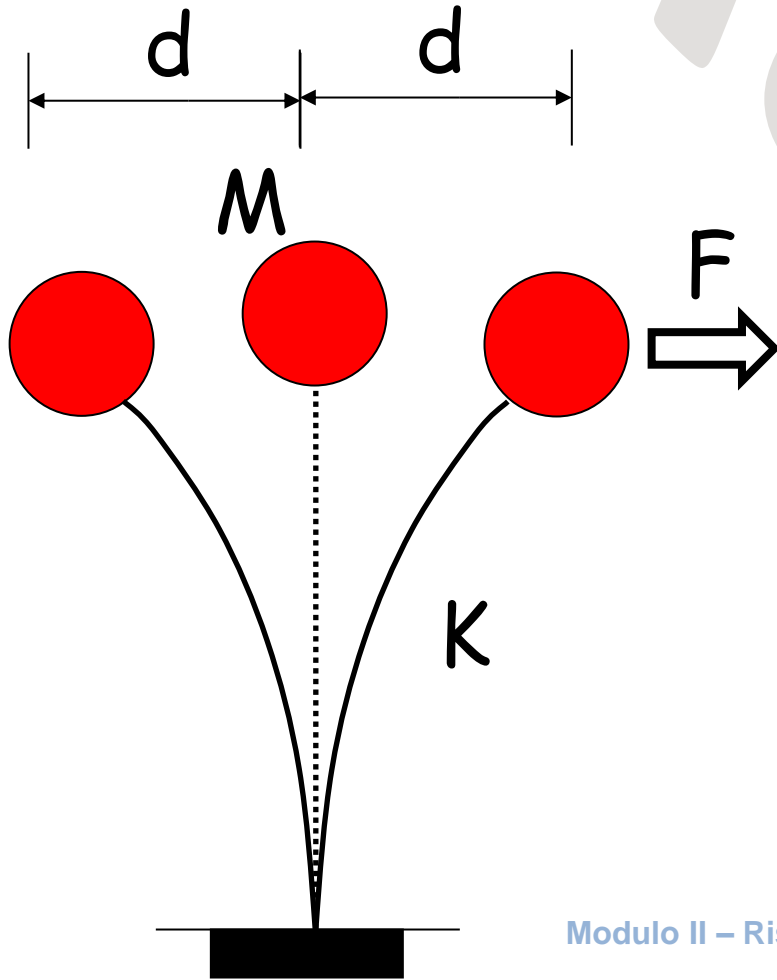


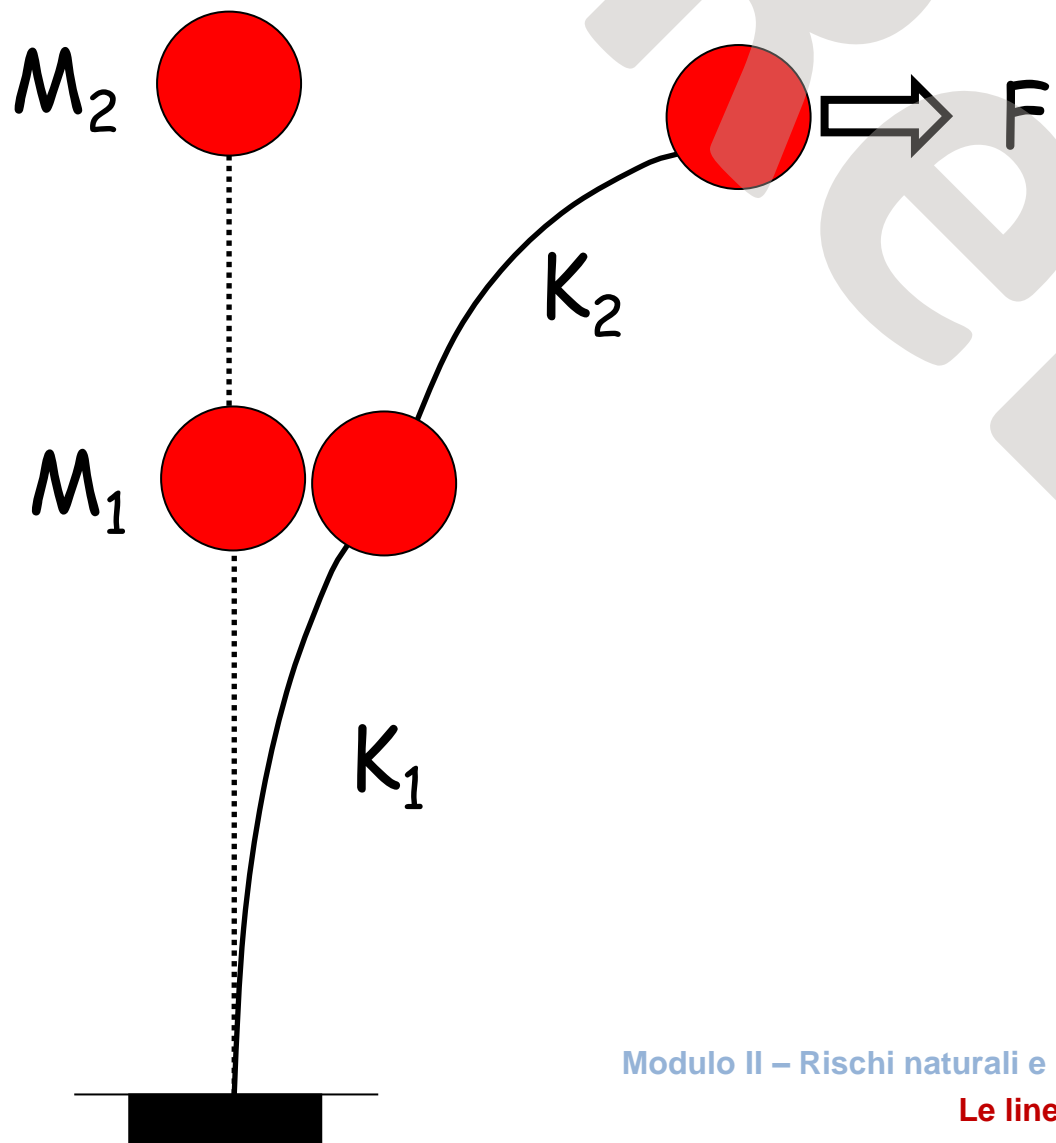
Anziché determinare T dai dati della struttura

Determinare i dati della struttura misurando sperimentalmente T

e.g.:

stimare la presenza delle fessure



di dinamica
delle strutture

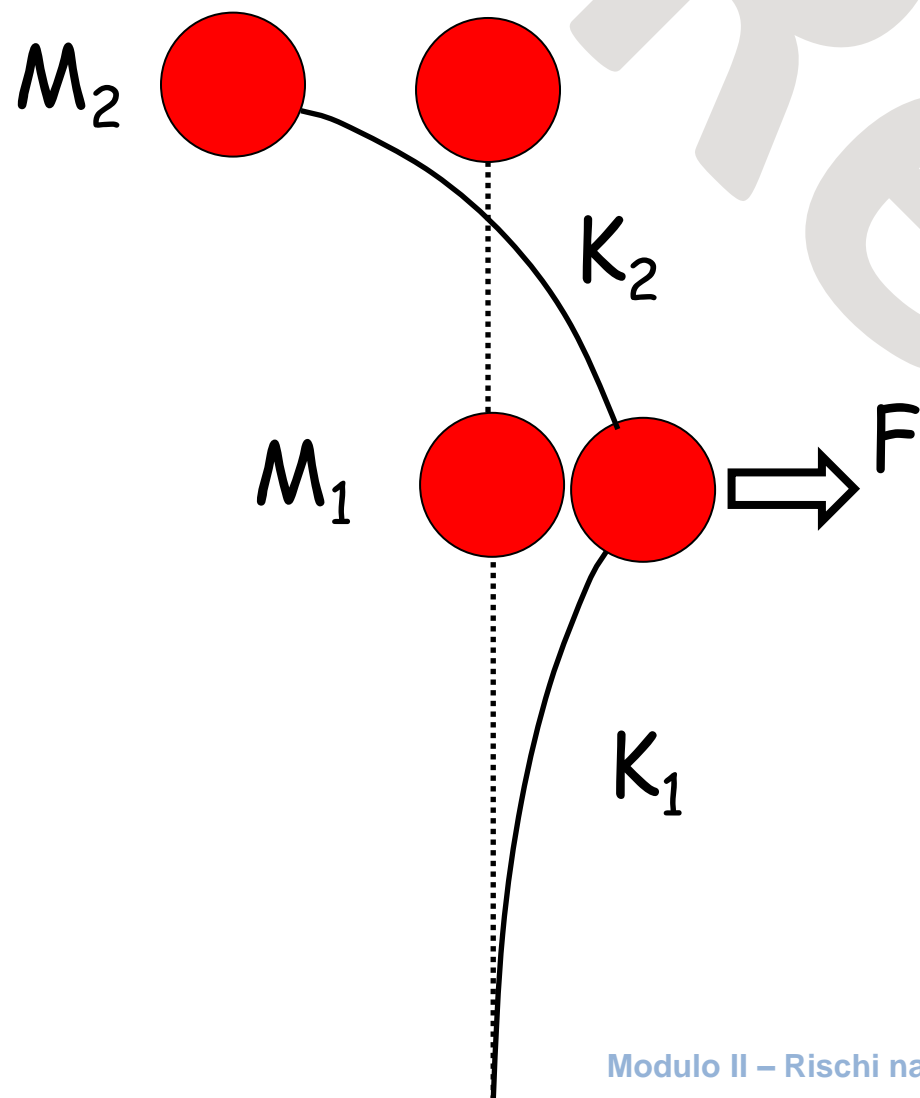
Strutture più complesse:
Due gradi di libertà



Primo modo di
vibrare

Primo periodo T_1

Cenni
di dinamica
delle strutture



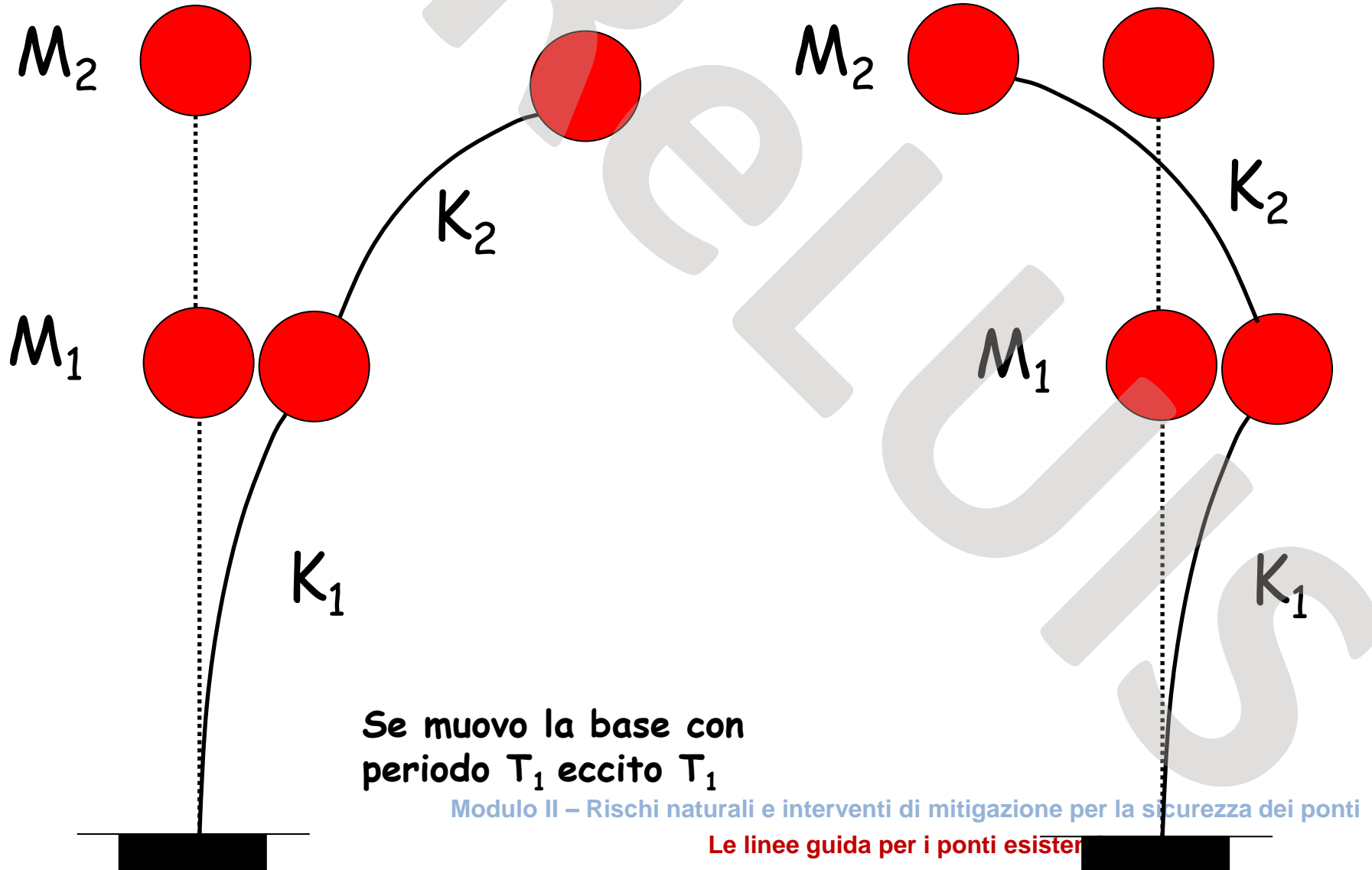
Strutture più complesse:
Due gradi di libertà

Secondo modo di vibrare
Secondo periodo T_2

Esistono entrambi i modi,
ma la forza applicata può
eccitarne prevalentemente
uno

cluis
**Cenni
di dinamica
delle strutture**

**Eccitazione alla
base e risonanza**



Se muovo la base con
periodo T_1 eccito T_1

Se muovo la base con
periodo T_2 eccito T_2

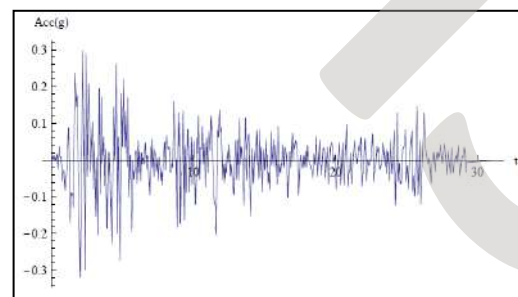
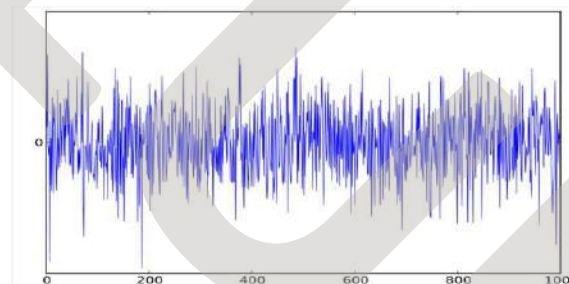
Identificazione dinamica

Se non uso una eccitazione specifica, la struttura vibra per l'eccitazione indotta dal rumore ambientale

e.g.:

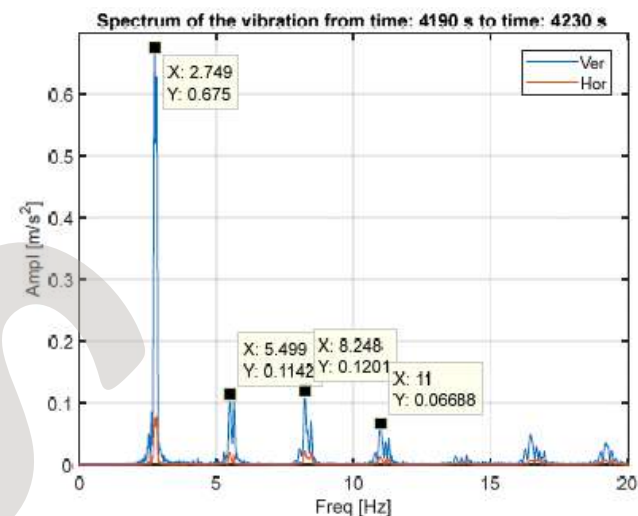
- Passaggio del traffico sul ponte
- Vento
- Vibrazioni per il passaggio dei treni

Input: rumore bianco (tutte le frequenze) o rosa (molte, non tutte)



Rumori bianchi e rumori rosa

Output: analisi spettrale - le frequenze proprie della struttura



Identificazione dinamica

Prime conclusioni

Non ha senso identificare una struttura senza disporre di un modello

Una eccitazione diversa può eccitare in modo maggiore o minore diversi periodi propri, ma non può cambiarne il valore

La deformata corrispondente ad un periodo («forma modale») non dipende dall'eccitazione

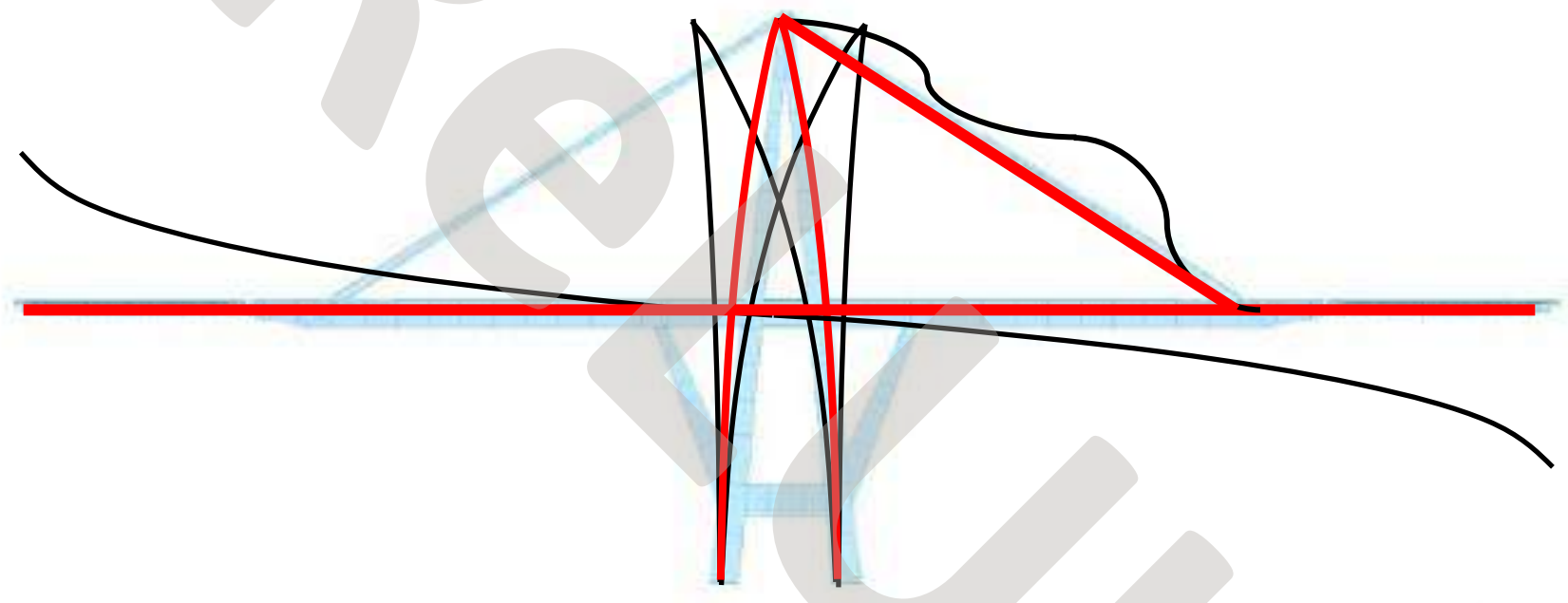
Una eccitazione più forte può indurre ampiezze maggiori di oscillazione

È utile avere indicazioni sull'eccitazione (e.g. disporre di un anemometro per vento forte)

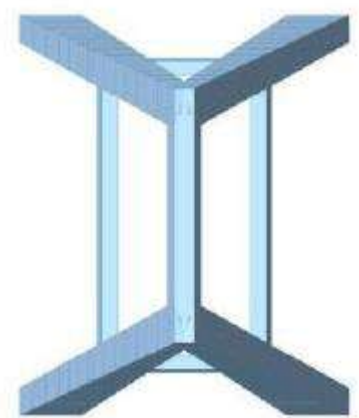
È fondamentale sapere cosa si sta cercando

Il ponte Morandi

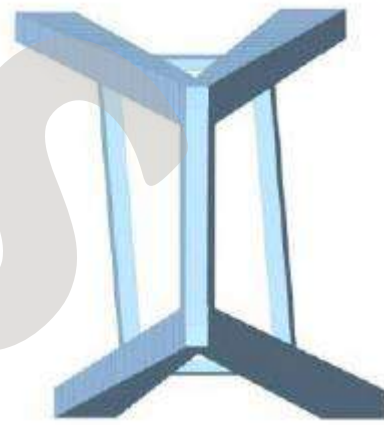
Com'era fatto



La pila 9
progettata



La pila 9
costruita



Le linee guida per i ponti esistenti

Prove sperimentali

Cosa cercare e perché

CESI 2015

OGGETTO DEL CONTRATTO

Costituiscono oggetto del presente contratto i servizi specialistici per l'assessment, la validazione del sistema di sorveglianza e le verifiche strutturali del Viadotto Polcevera dell'autostrada A10.

Nel dettaglio le attività consistono in:

1. Esame delle procedure di sorveglianza strutturale in atto e due diligence del sistema di monitoraggio installato;
2. Progettazione dei rilievi dinamici Pila 9 e Pila 10;
3. Esecuzione e assistenza rilievi dinamici;
4. Processamento delle misure;
5. Validazione del sistema di sorveglianza strutturale, eventuali proposte di miglioramento e identificazione per ulteriori determinazioni sperimentali.

Le attività sopracitate verranno svolte dalla Contraente secondo "Descrizione Tecnica" - Paragrafo 1.

"Descrizione delle Attività" presente nell'Offerta n. B5009998- AN15SCE019 del 08/05/2015 allegata al presente contratto.

- a) Progettazione ed esecuzione di prove dinamiche
- b) Validazione del sistema di sorveglianza
- c) Proposte di miglioramento del sistema di sorveglianza
- d) Proposta di altre prove

Prove sperimentali

Politecnico
di Milano
(Gentile)

Cosa cercare e
perché



che accetta, l'incarico di ricerca specialistica avente ad oggetto: "*Caratterizzazione dinamica dei sistemi bilanciati n. 9-10 del viadotto Polcevera in Genova e studio preliminare del sistema di monitoraggio dell'opera*".

Tale programma di ricerca si articola in n. 3 fasi:

- a) **Esecuzione di indagini** in condizioni operative (in presenza di traffico marittimo) sui sistemi bilanciati n. 9-10 del Viadotto Polcevera.
 - b) **Identificazione dinamica: parametri per calibrazione di modelli numerici**
 - c) **Identificazione dinamica: caratterizzazione dinamica completa per confronti successivi**
 - d) **Studio di un sistema di monitoraggio permanente**
- (2) Analisi e selezione di accenti di interesse in termini di identificazione di modelli numerici e di confronto con i dati sperimentali per la caratterizzazione del comportamento dinamico. Per la caratterizzazione di modelli numerici, sia a partire da una caratterizzazione completa delle opere indagate, sia per confronti successivi (ad esempio, durante gli eventi di risanamento).
- (3) Studio preliminare del sistema di monitoraggio permanente (statico/dinamico) da porsi in opera sui sistemi bilanciati n. 9-10 del Viadotto Polcevera in base a specifiche funzionali che verranno fornite da Spea.

Prove sperimentali

Cosa misurare



CESI 2015

Pila strumentata	Lato carreggiata	Posizioni di misura non installate
9	Sud	<ul style="list-style-type: none"> • Pos. 4 e 6 (stralli) • Pos. 5, 17, 18 sull'antenna • Pos. 13 sull'impalcato
9	Nord	<ul style="list-style-type: none"> • Pos. 4 e 6 (stralli) • Pos. 5 sull'antenna • Pos. 13 sull'impalcato
10	Sud	<ul style="list-style-type: none"> • Pos. 4 e 6 (stralli)
10	Nord	<ul style="list-style-type: none"> • Pos. 4 e 6 (stralli)

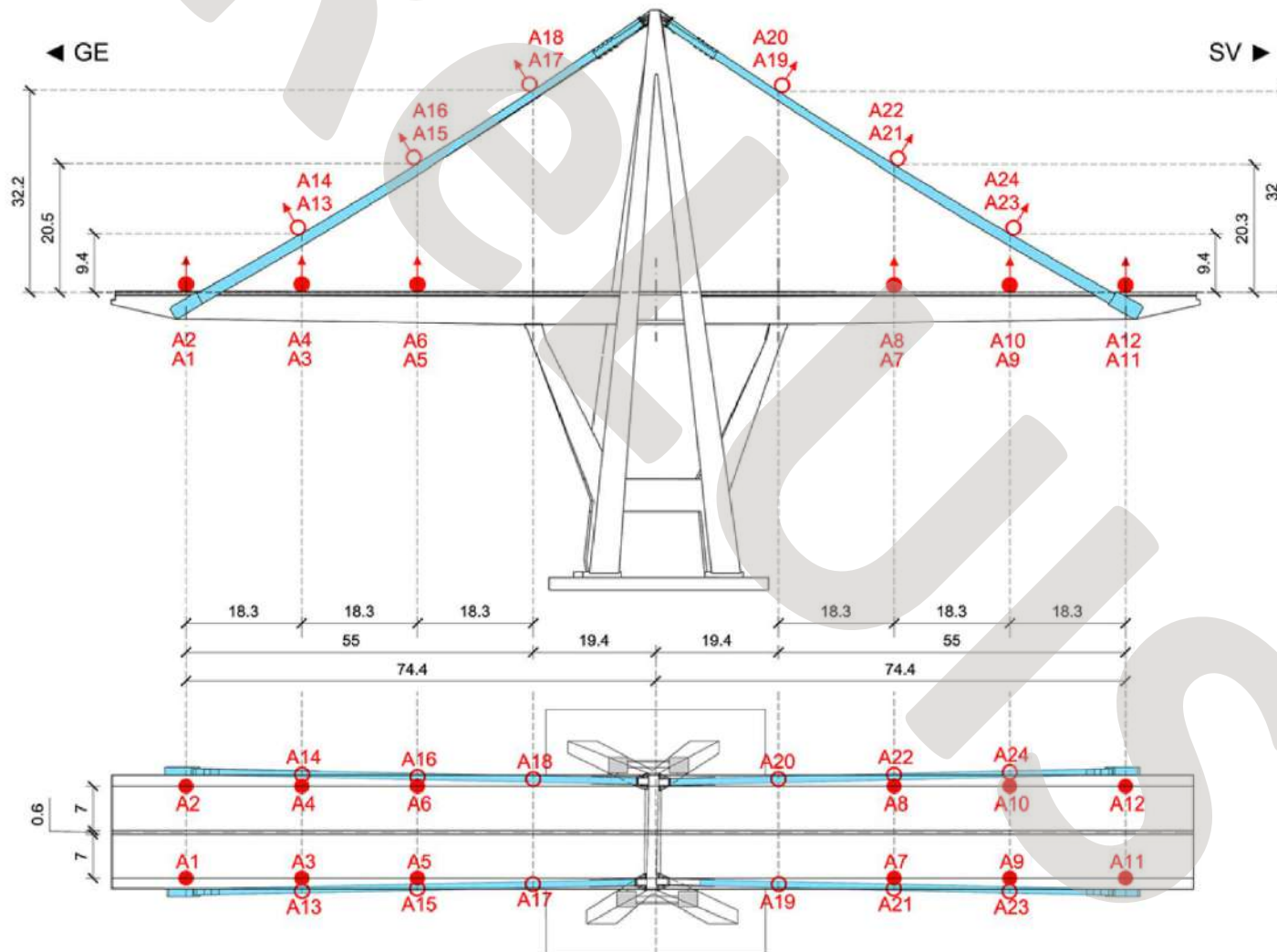


«Si evidenzia che, in ogni caso, nonostante la mancanza delle posizioni di misura indicate, l'assieme dei trasduttori era tale da consentire comunque l'identificazione completa delle caratteristiche dinamiche dell'infrastruttura»

Prove sperimentali

Cosa misurare

Politecnico
di Milano
(Gentile)



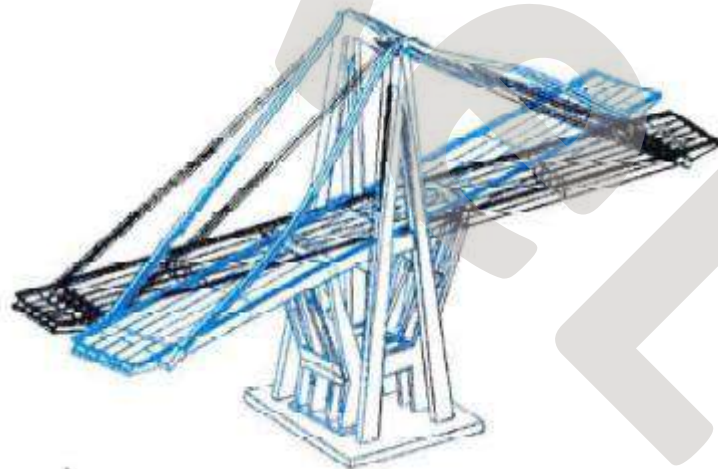
Le linee guida per i ponti esistenti

Pila
completamente
priva di
strumenti,
incluso il punto
di sommità

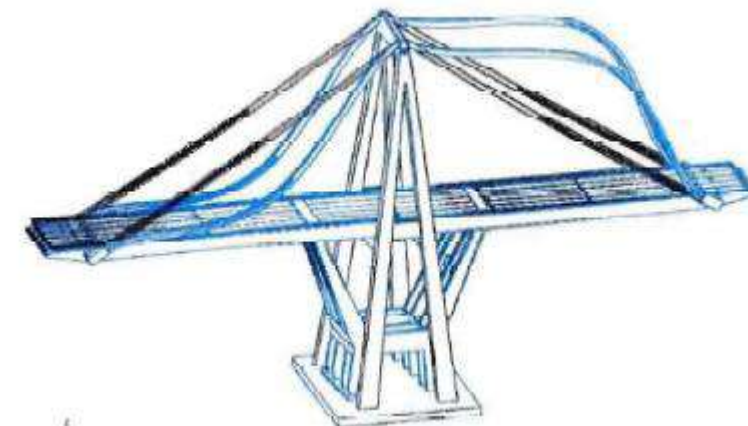
Prove sperimentali

CESI 2015

Modo 1
 $f = 0,49 \text{ Hz}$
 $T = 2,04 \text{ s}$



Modo 5
 $f = 0,72 \text{ Hz}$
 $T = 1,39 \text{ s}$



Quale modello

Perfetta corrispondenza sperimentale dopo gli aggiustamenti del modello

Asimmetria nella risposta degli stralli N e S di pila 10, non spiegata da CESI

Prove sperimentali

Quale modello

Politecnico
di Milano
(Gentile)

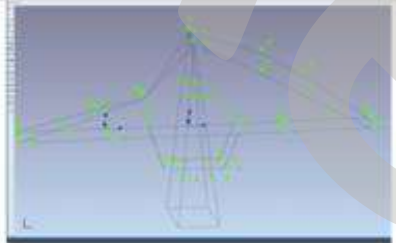


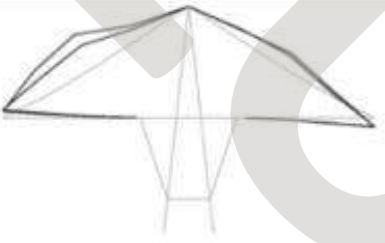

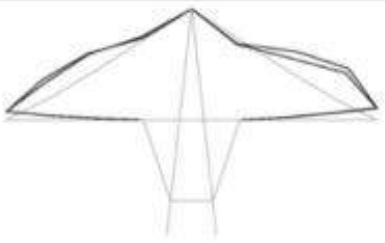
Non c'è un modello Polimi

C'è un modello numerico SPEA, oggetto di discussioni tra le parti, per una asserita mancanza di simmetria in un modo sperimentale definito "anomalo" negli stralli sud di Pila 9, con frequenza più alta rispetto al modo corrispondente di Pila 10

Identificazione dinamica

Confronti CESI POLIMI



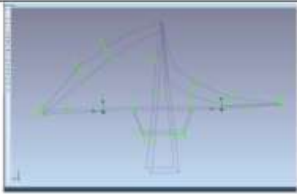
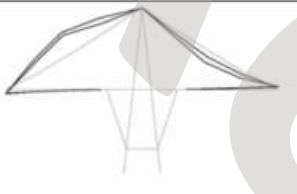
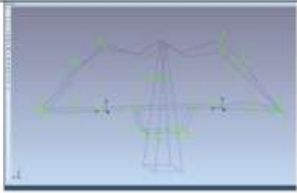
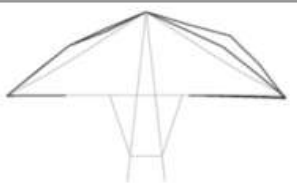
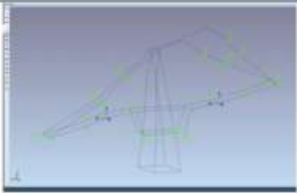
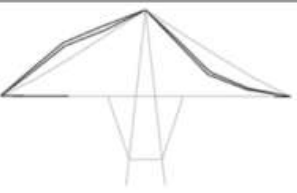
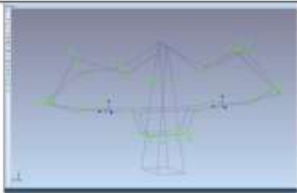
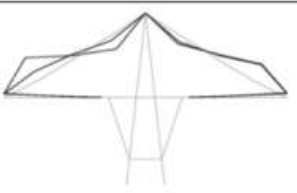
Modo	CESI 2015	GENTILE 2017	Δ [%]
Flessionale impalcato + stralli			0
	0.732 Hz	0.728 Hz	
Stralli nel piano verticale			0
	0.811 Hz	0.815 Hz	
Flessionale impalcato + stralli			+2
	1.416 Hz	1.445 Hz	

Pila 10

Identificazione dinamica

Confronti CESI POLIMI



Modo	CESI 2015	GENTILE 2017	Δ [%]
Flessionale impalcato + stralli			-1
	0.713 Hz	0.703 Hz	
Stralli nel piano verticale			0
	0.723 Hz	0.728 Hz	
Flessionale impalcato + stralli			-7
	0.790 Hz	0.737 Hz	
Flessionale impalcato + stralli			0
	1.563 Hz	1.567 Hz	

Pila 9

Ottima corrispondenza in tutti i casi confrontabili, in entrambe le pile

(non sono ovviamente confrontabili modi che coinvolgano l'antenna)

ali e interventi di mitigazione per la sicurezza dei ponti

e linee guida per i ponti esistenti

Identificazione dinamica

CESI

Pila 10

Il livello di RMS (Root Mean Square) degli spostamenti registrati nella risposta degli stralli sud era superiore a quello registrato nella risposta degli stralli nord

Se cambiano i valori di RMS ma non cambiano i periodi, allora la diversa ampiezza del segnale deve dipendere dall'azione diversa

Dai dati meteo dei giorni di prova sembrerebbe emergere come il vento fosse più forte il 29 ottobre (con raffiche a 37 km/h) rispetto al 5 novembre (raffiche a 16 km/h)

Identificazione dinamica

"le deformate dei modi globali osservate in ambedue i sistemi bilanciati presentano un'evidente mancanza di simmetria longitudinale (lato Nord vs. lato Sud) e trasversale (lato Genova vs. lato Savona), relativamente al comportamento degli stralli in c.a.p. **Tale mancanza di simmetria**, che si palesa attraverso spostamenti modali abbastanza diversi dei 4 stralli nell'ambito della medesima deformata modale **è certamente da ascrivere a differenze nelle caratteristiche meccaniche e nell'azione di tiro degli stralli.**"

"**appare probabile - a parere di chi scrive - che le differenze osservate siano riconducibili ad una differente pre-sollecitazione residua nei 4 tiranti** in c.a.p. generata, ad esempio, da possibili fenomeni di corrosione nei cavi secondari, difetti di iniezione, ecc."

"per gli stralli del sistema bilanciato n. 9 è stato possibile identificare con confidenza solo 4 modi globali e 2 di essi **si presentano con deformata modale non del tutto conforme alle attese** e certamente meritevole di approfondimenti teorico-sperimentali."

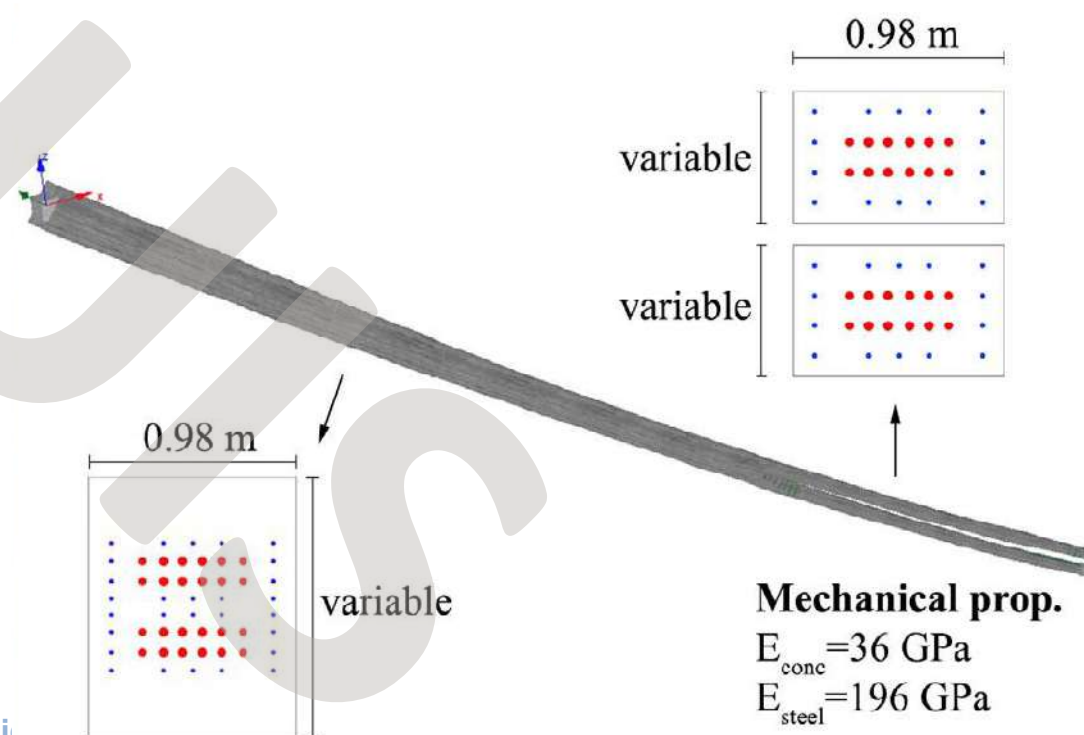
Il ponte Morandi

Gli stralli



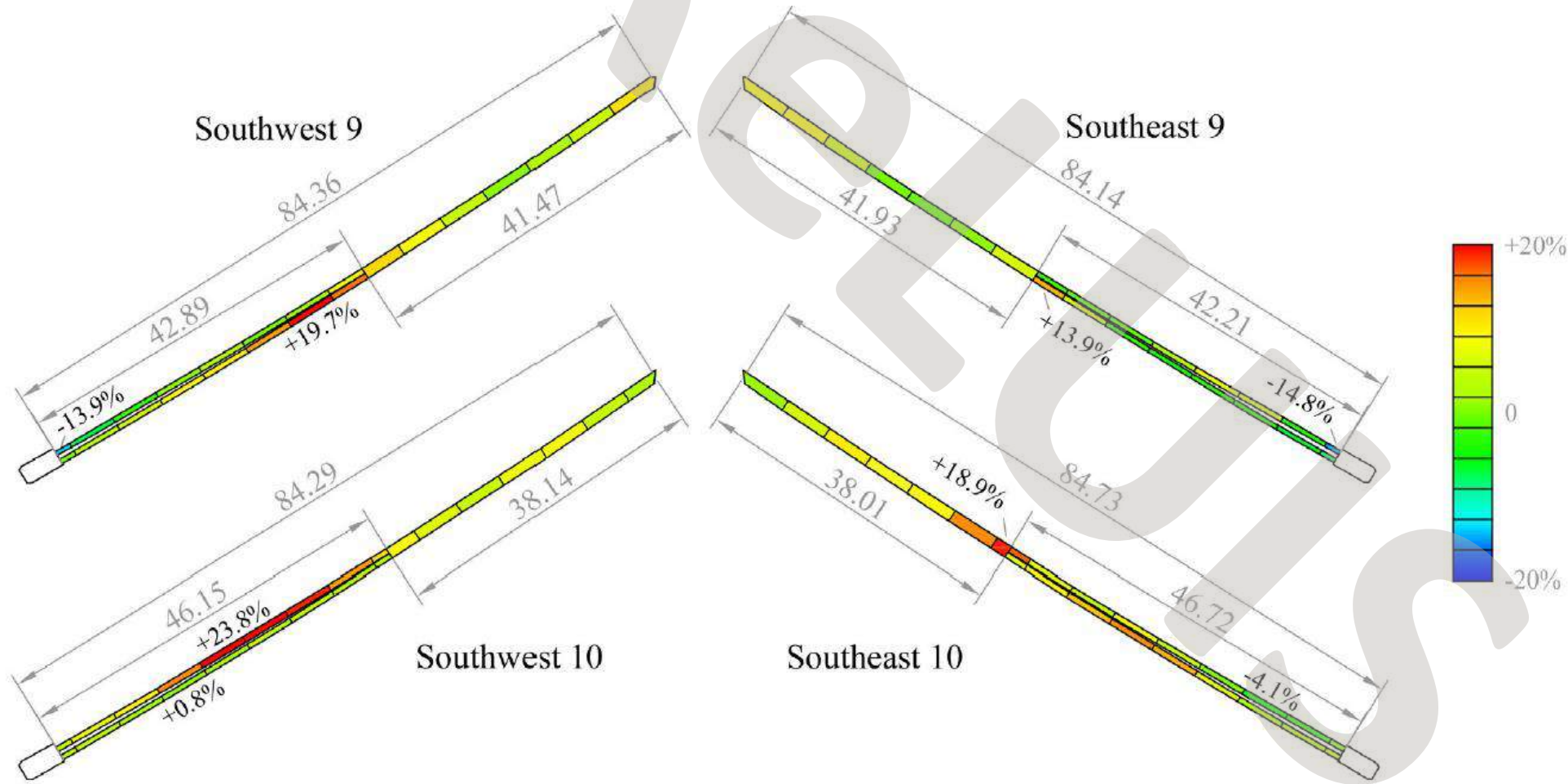
«Sagging»

Biforcazione



Il ponte Morandi

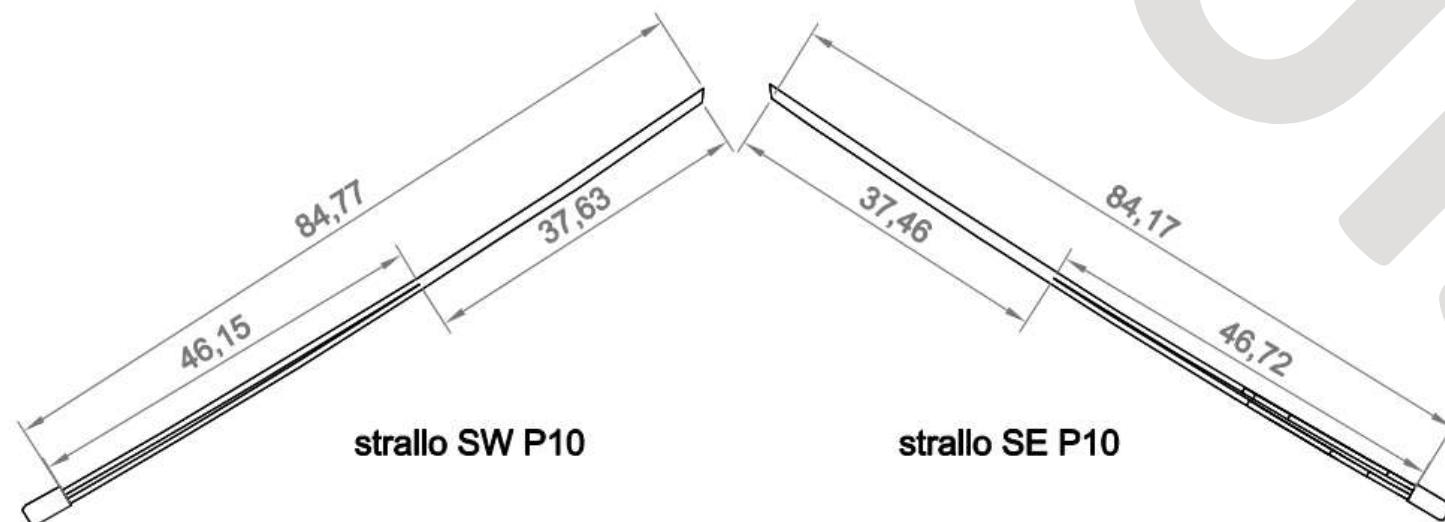
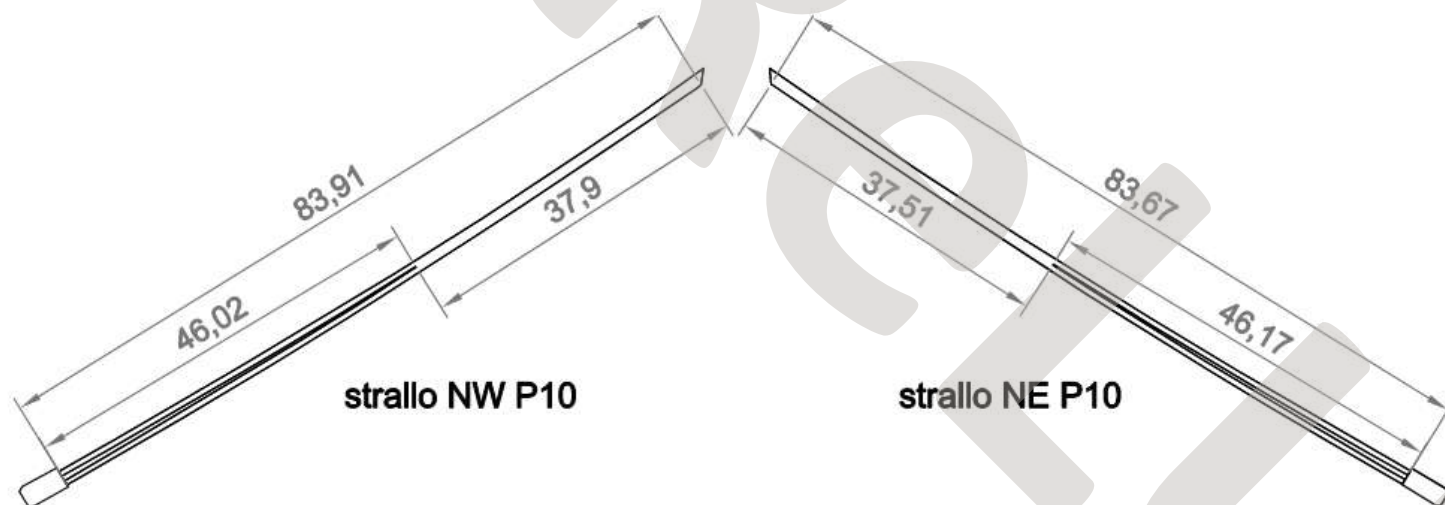
Gli stralli



Variazione percentuale dell'altezza della sezione degli stralli reali rispetto al progetto

Il ponte Morandi

Gli stralli



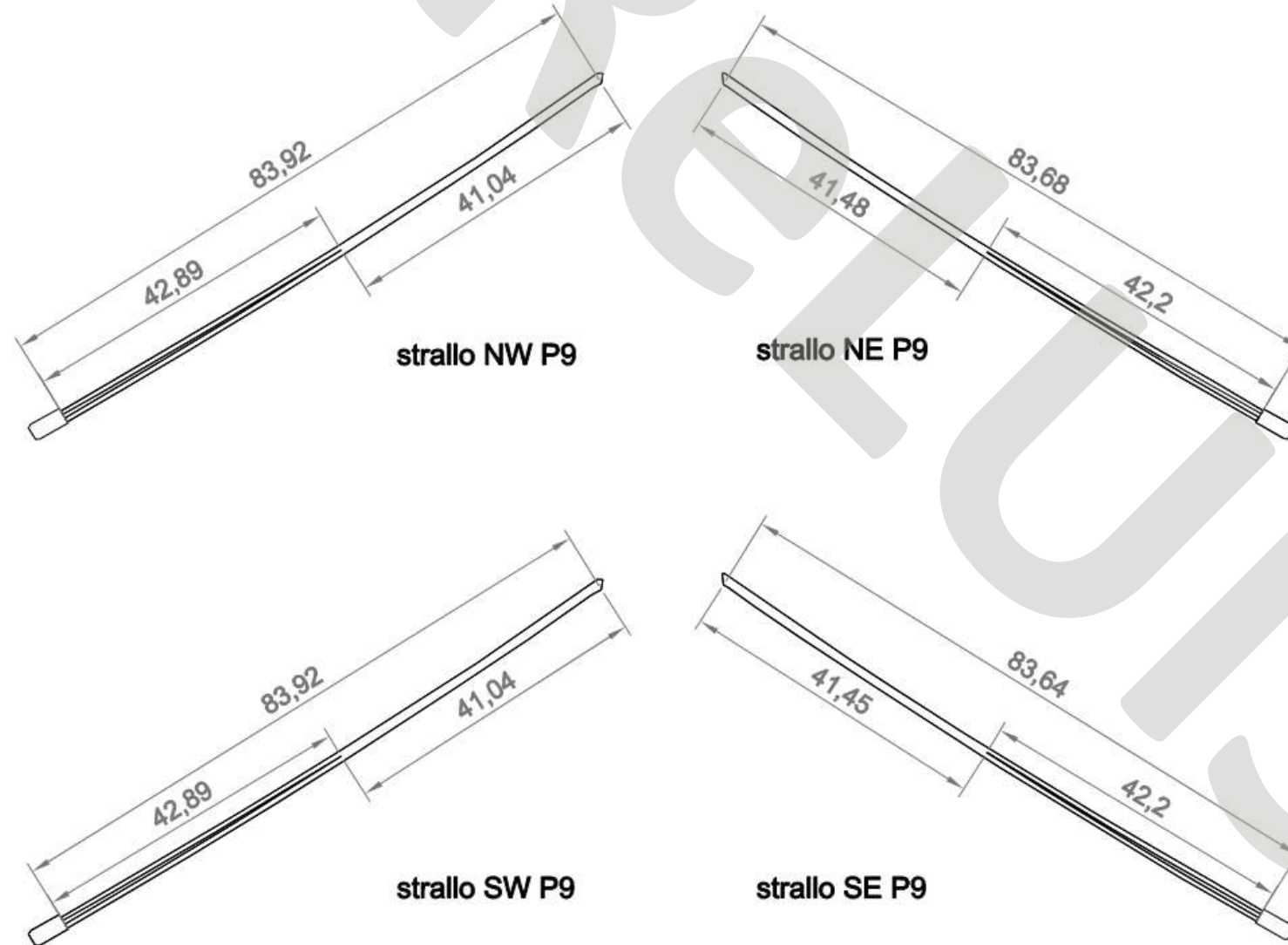
Pila 10

icurezza dei ponti

Il ponte Morandi

Gli stralli

Pila 9



La parte biforcata è
circa 4 m più corta

dei ponti

Identificazione dinamica

Confronto tra dati sperimentali ed un modello globale



Caratteristiche dinamiche				
# modo Polimi	# modo Midas	Polimi f [Hz]	Midas f [Hz]	Differenza [%]
1	2	0.703	0.640	-9
2	4	0.723	0.718	-1
3	6	0.737	0.722	-2
4	5	0.811	0.719	-11
5	19	1.567	1.571	< 1
6	20	1.660	1.657	< 1
7	36	2.778	2.835	+2
8	40	3.047	2.919	-4

Identificazione dinamica

Confronto tra dati sperimentali ed un modello globale

Pila 10

Stay	Mode	Experimental data f (Hz)	Numerical data f (Hz)	Δ [%]
SE10	1 st Local bifurcation	1.514	1.570	+3.7
SW10	1 st Local bifurcation	1.499	1.582	+5.5
SE10	2 nd Local bifurcation	1.675	1.712	+2.2
SW10	2 nd Local bifurcation	1.621	1.749	+7.9
SE10	3 rd	2.793	2.954	+5.6
SW10	3 rd	2.666	3.018	+13

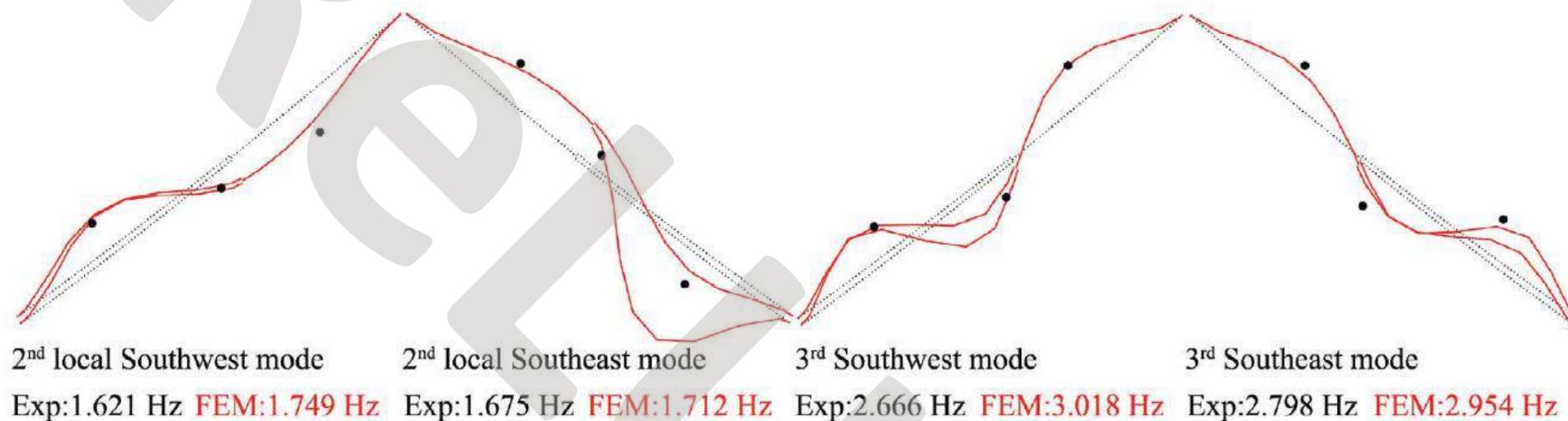
Pila 9

Stay	Mode	Experimental data f (Hz)	Numerical data f (Hz)	Δ [%]
SE9	2 nd Local bifurcation	1.831	1.828	-0.2
SW9	2 nd Local bifurcation	1.904	1.840	-3.4
SE9	3 rd	2.695	2.929	+8.7
SW9	3 rd	2.891	2.978	+3.0

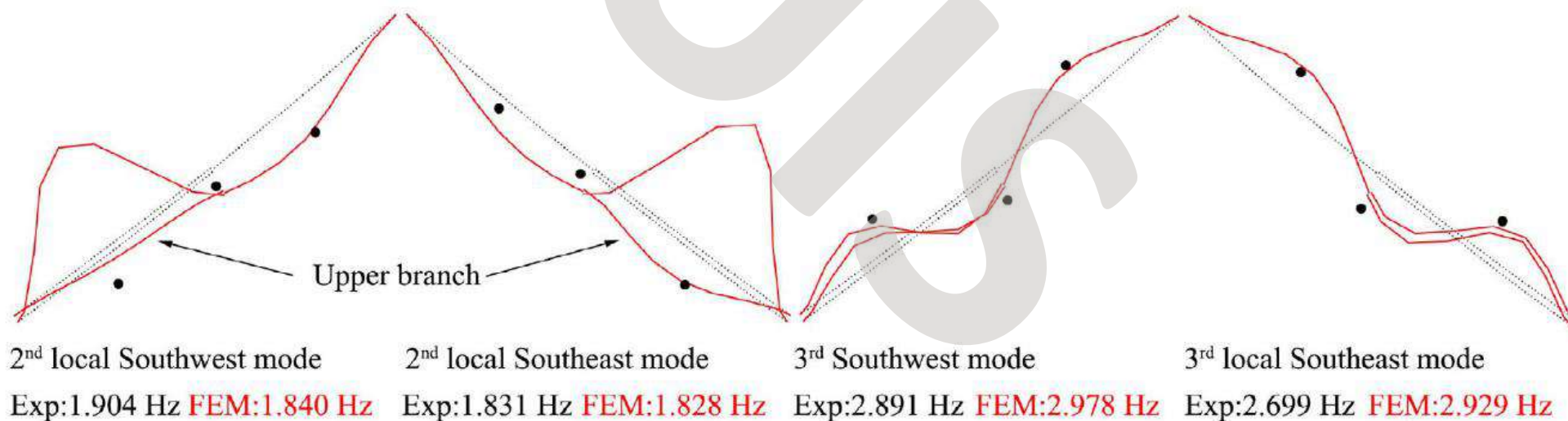
Identificazione dinamica

Confronto tra dati sperimentali ed un modello globale

Pila 10



Pila 9



**Identificazione
dinamica**

**Asimmetrie e
deformate inattese**

**Un modello che tenga conto della
effettiva geometria spiega le
apparenti incongruenze evidenziate**

Identificazione dinamica

Si poteva capire se lo strallo era compresso?

Una perdita completa della post-compressione avrebbe comportato una situazione di fessurazione estesa e sistematica, con conseguente variazione del momento di inerzia (J)

La variazione di rigidità avrebbe indotto una significativa variazione di T (o f), immediatamente identificabile dai dati sperimentali

I dati sperimentali confermano con certezza che permaneva uno stato di coazione, con calcestruzzo compresso e cavi tesi

Identificazione dinamica

$$f_n = \frac{n}{2L_{beam}} \sqrt{\frac{1}{m_{beam}} \left[EI \left(\frac{n\pi}{L_{beam}} \right)^2 - C \right]}$$

$$f_n = \frac{n}{2L_{cable}} \sqrt{\frac{T}{m_{cable}}}$$

Si poteva valutare il livello di compressione?

Trave:
se aumenta C diminuisce f

Cavo:
se aumenta T aumenta f

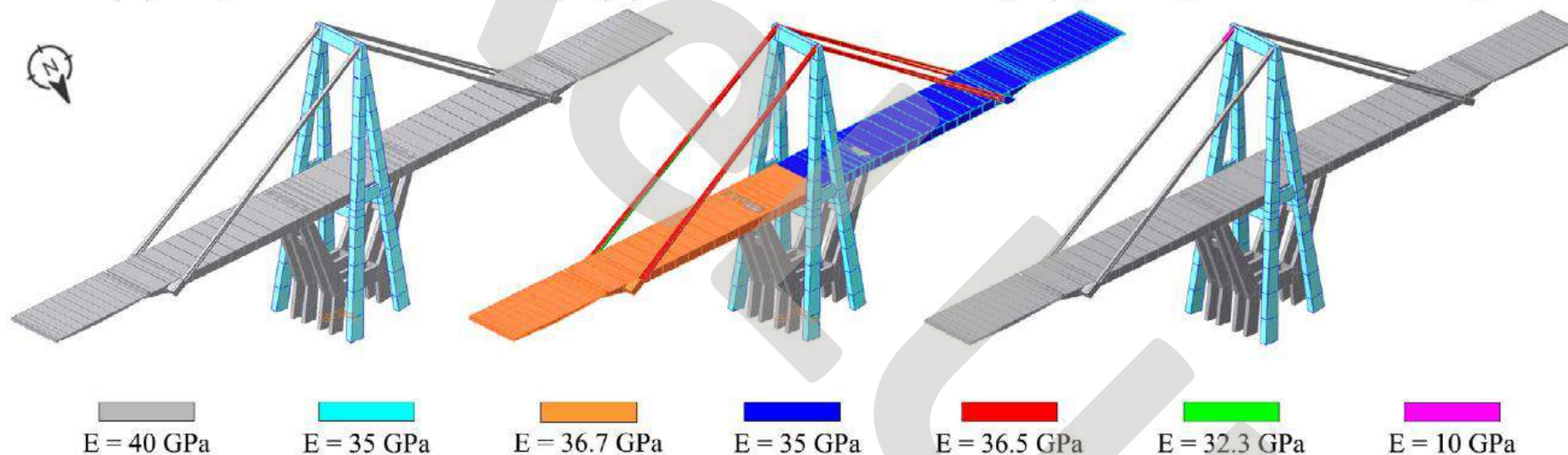
In un elemento precompresso i due sistemi coesistono e $C = T$

Non è concettualmente né numericamente possibile identificare una perdita di postcompressione, a meno di una perdita completa con conseguente fessurazione sistematica

Identificazione dinamica

Influenza del difetto su periodi e forme di vibrazione

(A) "Simplified" numerical modelling (B) "Detailed" numerical modelling (C) "Damaged" numerical modelling



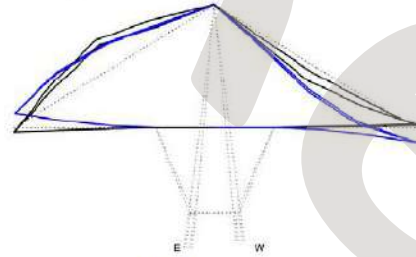
Experimental frequency f (Hz)	'Simplified' model f (Hz)	'Detailed' model f (Hz)	'Damaged' model f (Hz)
0.703	0.640	0.620	0.632
0.723	0.718	0.704	0.718
0.737	0.722	0.707	— ^a
0.811	0.719	— ^a	— ^a
1.567	1.571	1.674	1.557
1.660	1.657	1.504	1.678
2.778	2.835	2.575	2.835
3.047	2.919	2.809	2.904

a sicurezza dei ponti

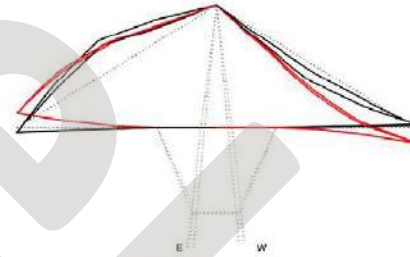
Identificazione dinamica

Influenza del difetto su periodi e forme di vibrazione

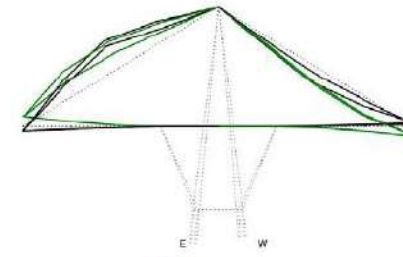
1st experimental mode: 0.703 Hz



FEM: 0.640 Hz

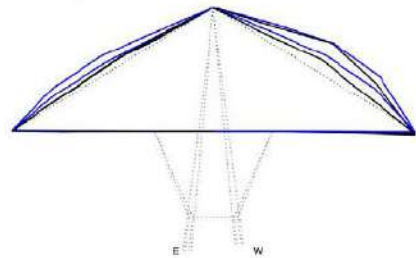


FEM: 0.620 Hz

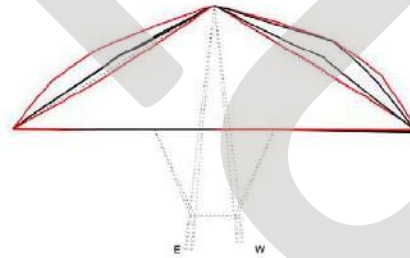


FEM: 0.632 Hz

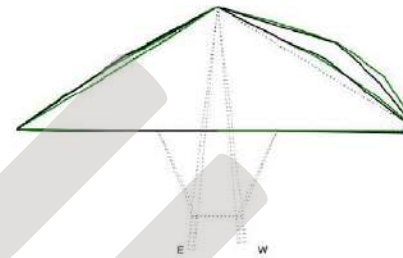
2nd experimental mode: 0.723 Hz



FEM: 0.718 Hz

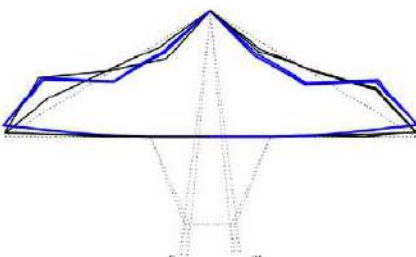


FEM: 0.704 Hz

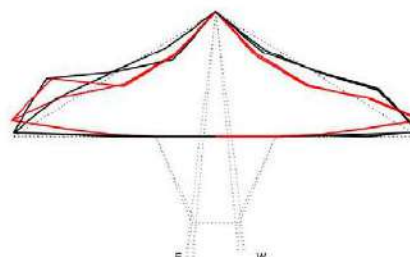


FEM: 0.718 Hz

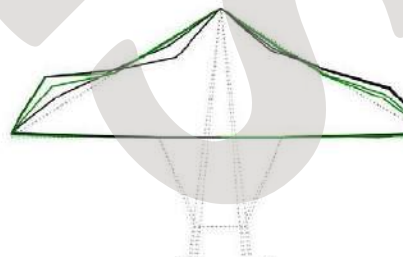
5th experimental mode: 1.567 Hz



FEM: 1.571 Hz



FEM: 1.674 Hz



FEM: 1.557 Hz

Identificazione dinamica

Conclusioni (1)

L'assenza di qualsiasi strumento sulla pila nelle prove effettuate da PoliMI escludeva la possibilità di tarare modelli globali del sistema bilanciato e di identificare modi in cui partecipasse la risposta dell'antenna.

La forma modale del modo "locale" ritenuto anomalo risulta spiegata considerando la biforcazione degli stralli.

Le differenze di frequenza tra i modi locali ritenuti anomali negli stralli sud del sistema 9, rispetto a quelli analoghi del 10, sono identificate dai modelli, purché vengano riprodotte le effettive geometrie, con sezioni diverse e diverse lunghezze della parte biforcata degli stralli e rotazione delle pile attorno all'asse verticale.

La stessa considerazione vale per le modeste asimmetrie nella risposta sperimentale del sistema 9.

Identificazione dinamica

Conclusioni (3)

La variazione dello stato di coazione negli stralli non induce significativi cambiamenti nella risposta, né dal punto di vista teorico né da quello numerico, a meno che si arrivi ad uno stato di trazione generalizzato nel calcestruzzo.

I dati sperimentali consentono di escludere tale situazione, senza consentire alcuna considerazione in merito al valore della post-compressione residua.

Affermare che una diversa frequenza può essere dovuta ad un diverso valore della coazione residua è concettualmente errato.

Identificazione dinamica

Conclusioni (4)

L'introduzione di uno stato di danno locale nella parte sommitale dello strallo non conduce a risultati numerici più prossimi a quelli sperimentali, pertanto non sarebbe stato ragionevole ipotizzarne la presenza.

Identificazione dinamica

Conclusioni (5)

Le prove sperimentali effettuate, non avrebbero in alcun modo consentito di identificare uno stato di danno localizzato, né di stimare il valore dello stato di post-compressione residuo negli stralli.

Il difetto non avrebbe potuto essere identificato mediante l'analisi di dati dinamici sperimentali

La presenza dei due sistemi di monitoraggio non avrebbe consentito di rilevare l'imminenza del crollo

Identificazione dinamica

Conclusioni (8)

I dati ottenuti confermano una risposta fisiologica della struttura, identificano valori ragionevoli delle proprietà meccaniche, assicurano la presenza di uno stato di coazione residuo, senza consentire di stimarne l'entità.

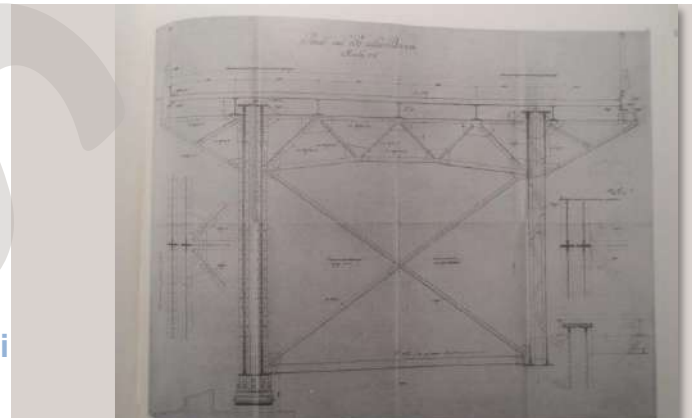
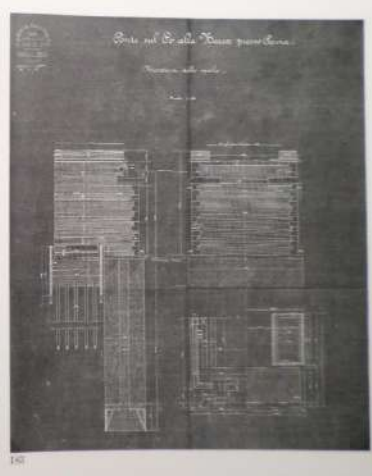
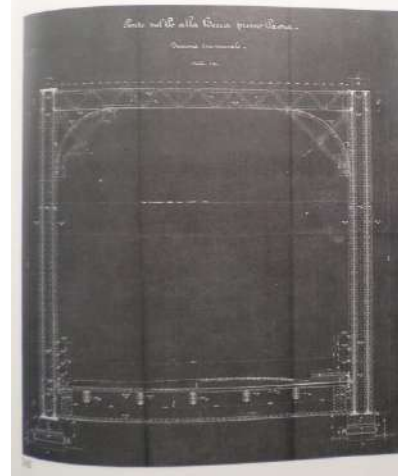
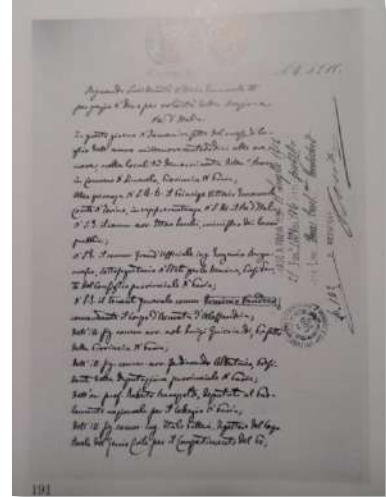
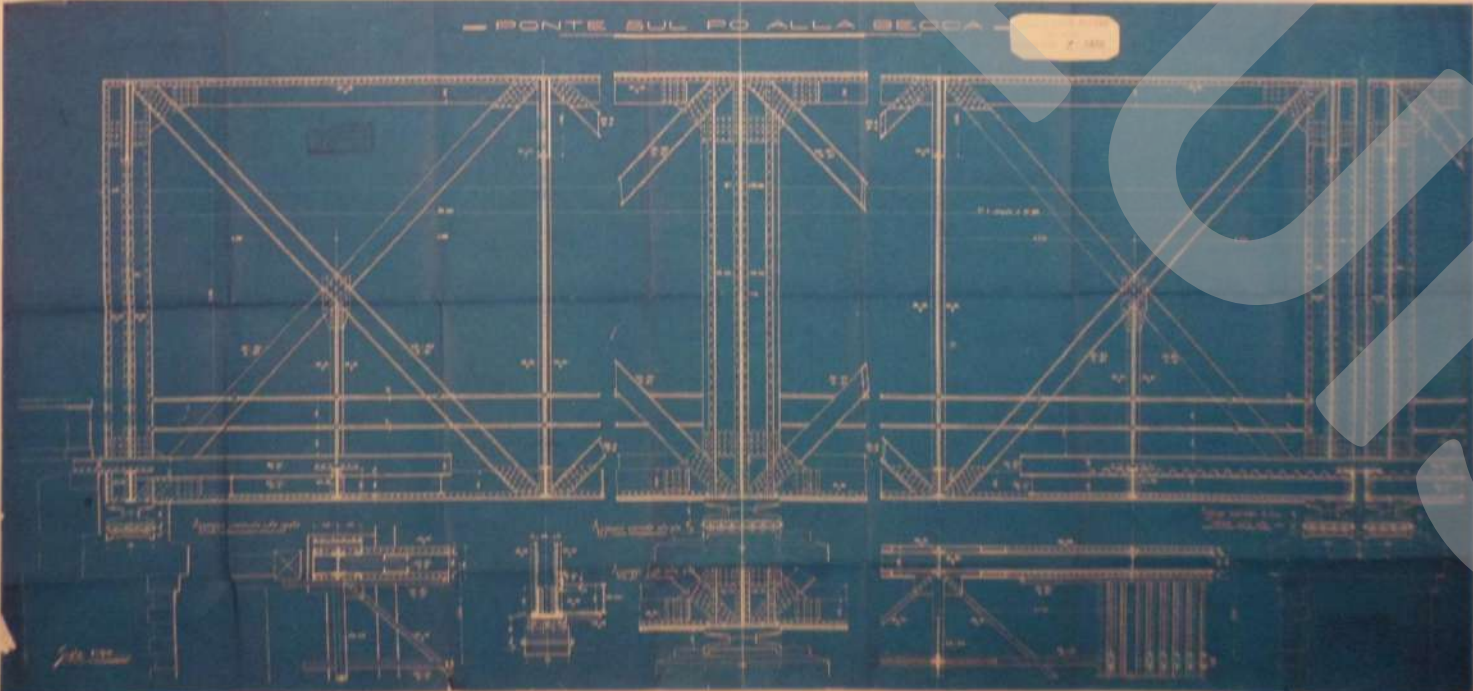
The time of steel

Il ponte della Becca alla confluenza di Po e Ticino

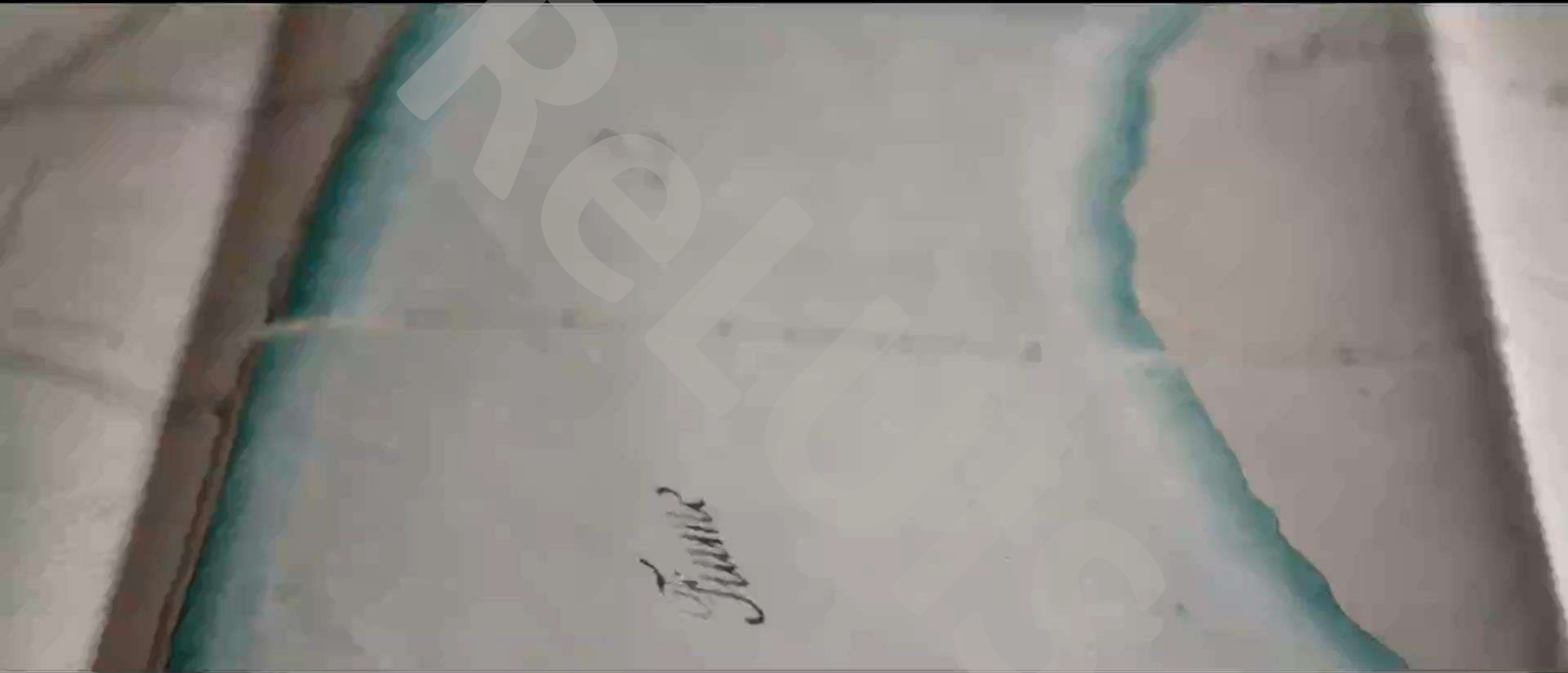
RELUMS

IL PONTE ATTUALE E LA SUA STORIA

IL PROGETTO



IL COSTO DELL'OPERA

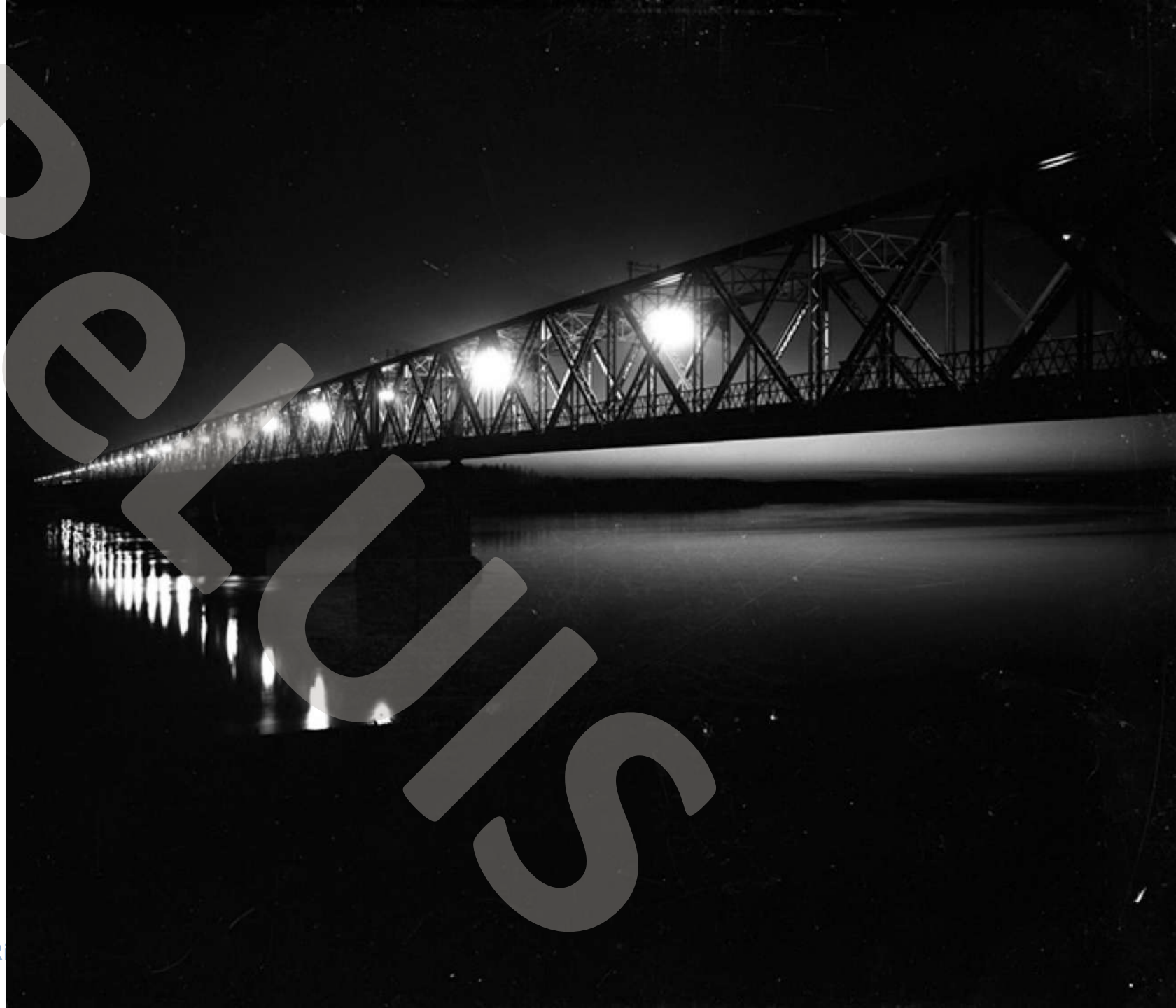


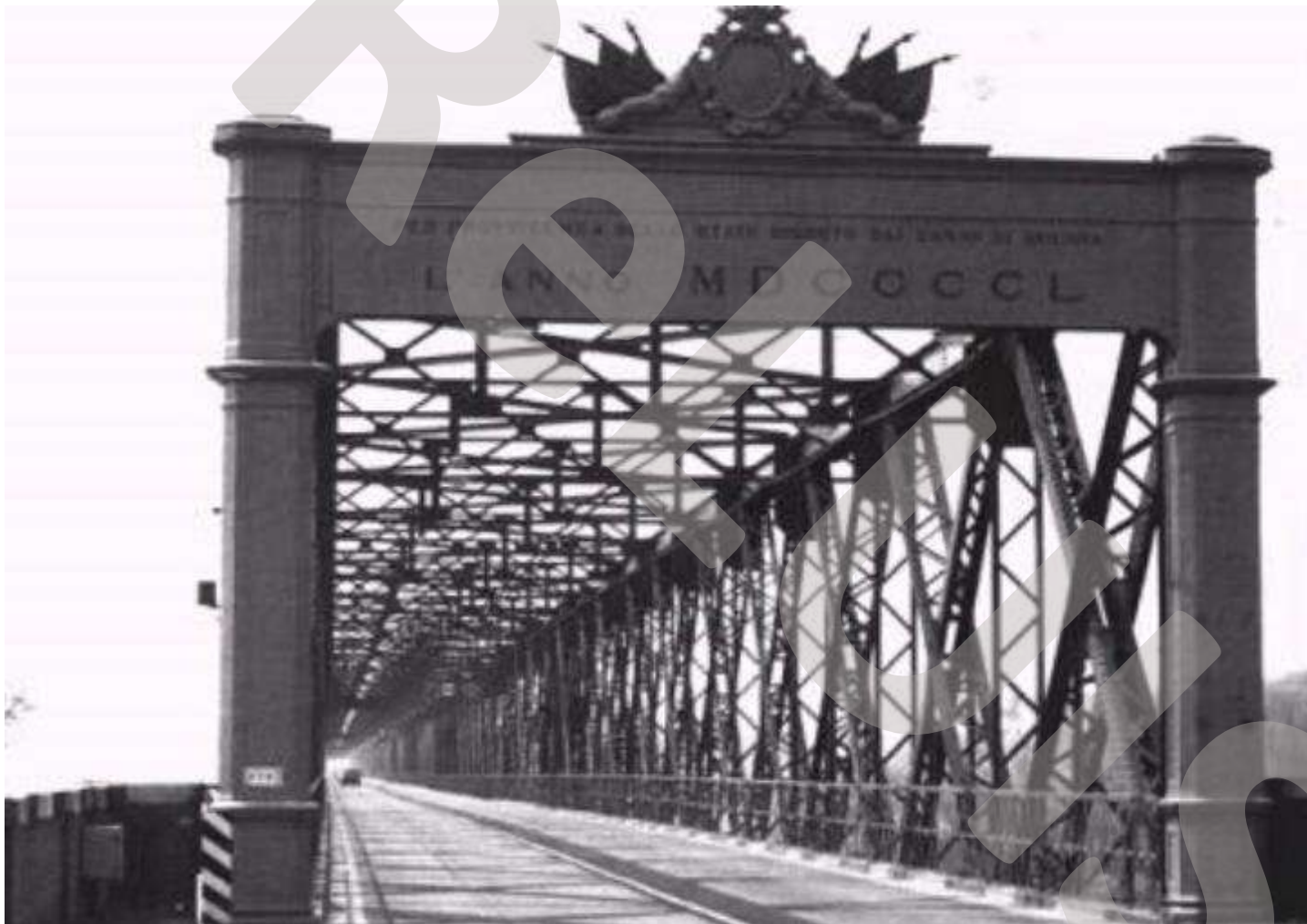
Fiume

IL PONTE ATTUALE E LA SUA STORIA

MANUTENZIONE



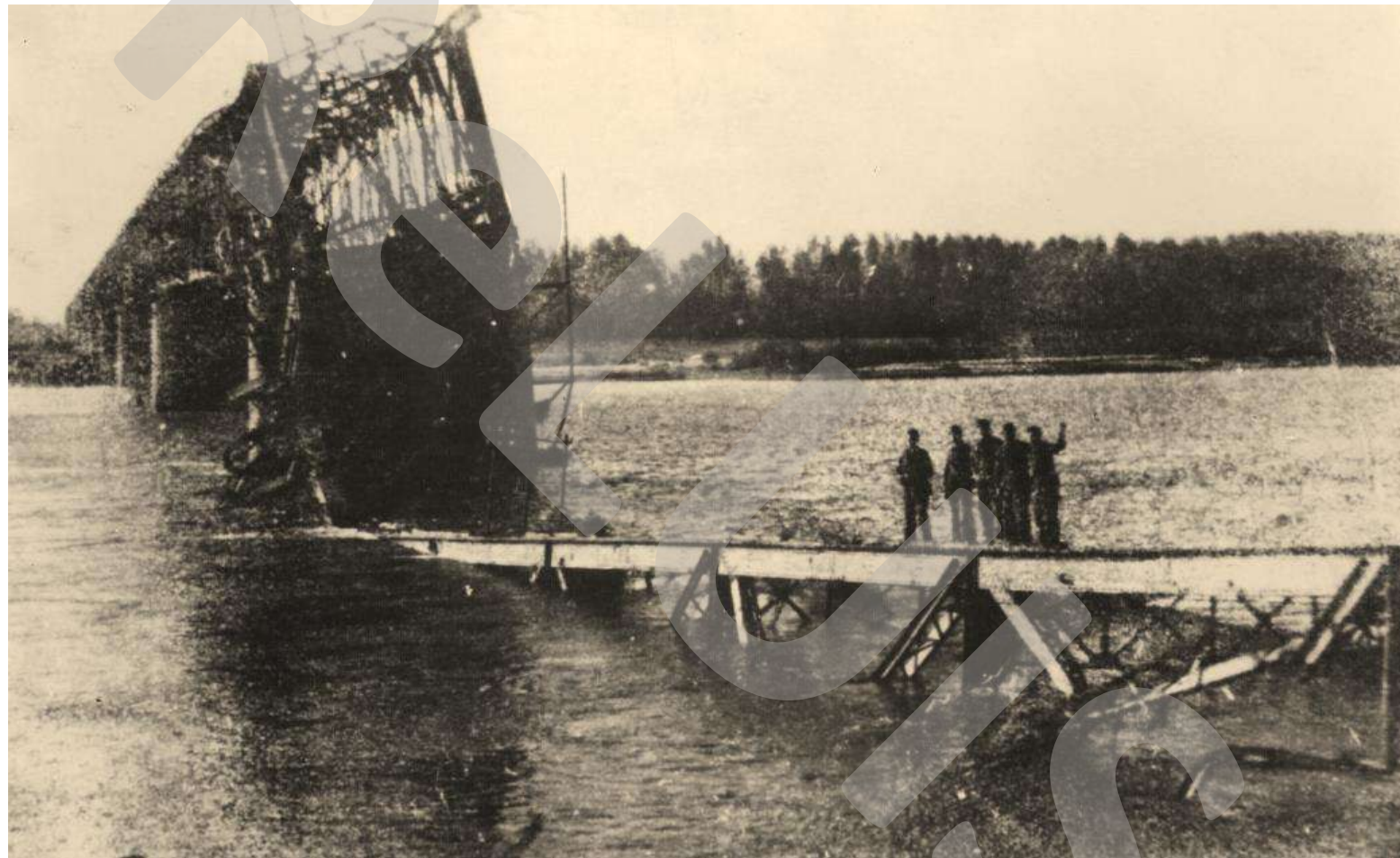




1912

Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti



Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti



Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti



upstream



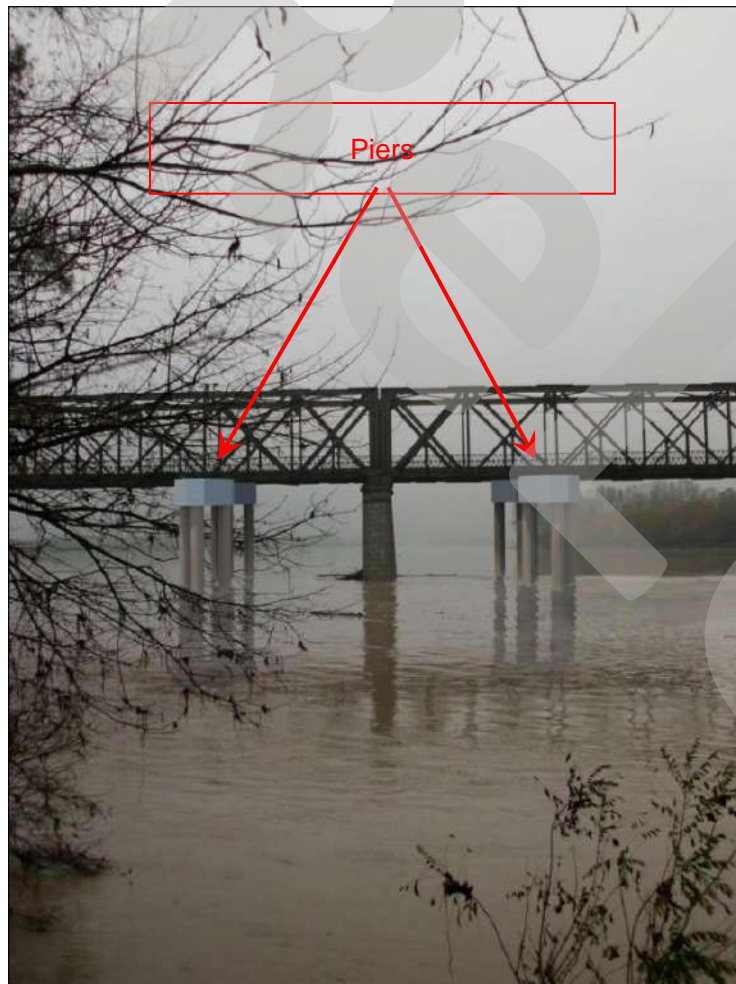
downstream

**Bearing on Pier 9
(upstream)**



**Bearing on Pier 9
(downstream)**





Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti

IL PONTE ATTUALE E LA SUA STORIA

PILA 9

2010
-2012



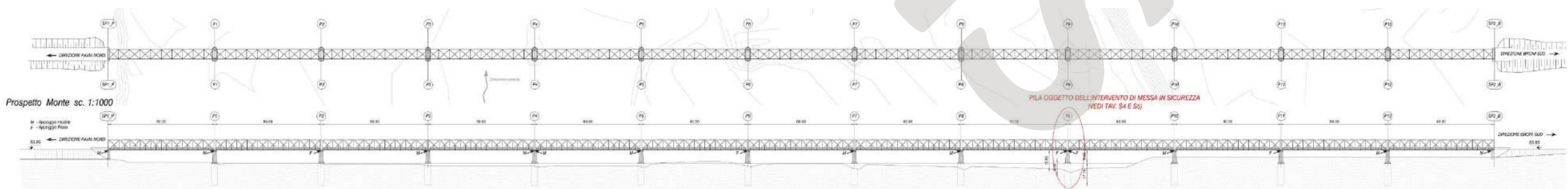
pe
en





Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

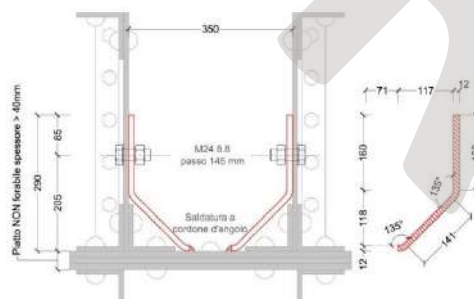
Le linee guida per i ponti esistenti



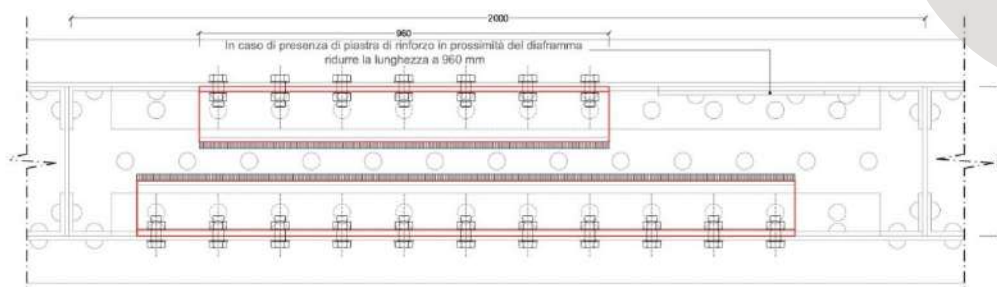
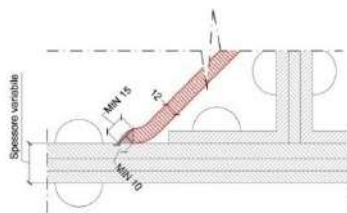


Modulo II – Rischi naturali e interventi di mitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti



DETTAGLIO SALDATURA
SCALA 1:2



The Viaduct 1 Anatolian Motorway (Bolu Viaduct)

Duzce Earthquake, Nov. 1999







Two parallel viaducts

58/59 spans

Span lengths 39.4m

Continuous deck slab

Movement joints at 10 spans, 400m

7 Precast V-Beams

In situ deck slab

Pier heights 10m to 50m

14 pot bearings (200 mm displ. capacity) per pier

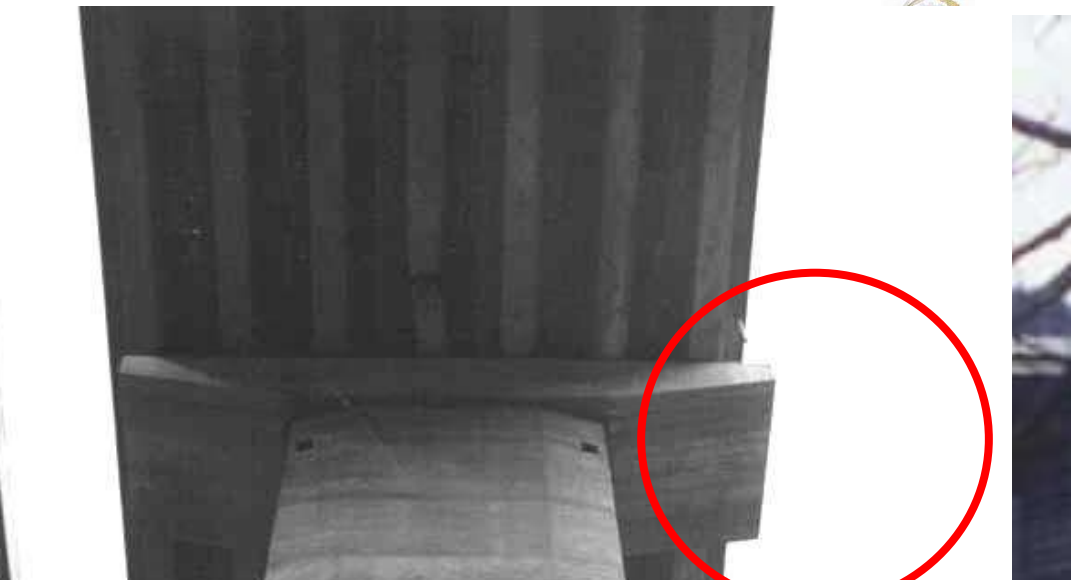
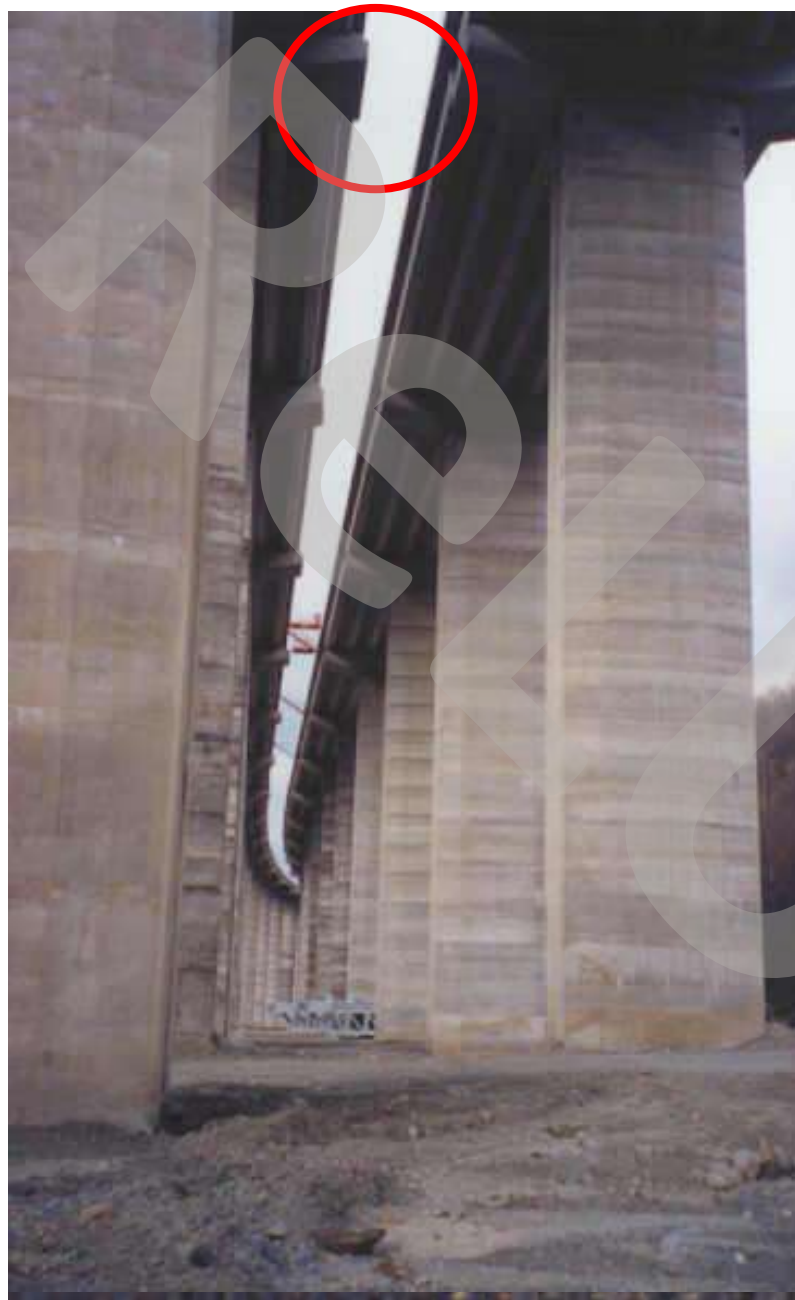
1 EDU (360 mm) per pier

Pier hollow section ~ 8.2 x 4.5 m

Adequate confinement and shear reinforcement

Good detailing

**Relative
displacement
~ 1.5m**



**Part of bridge being driven towards Istanbul by 1.5m
Pot bearings ejected and destroyed
Beam end supported on support blocks, or unsupported**

**Severe cracking to link slabs [up to 300mm
vertical deflection over 1.8m (20% drift)]**

2002/ 9/27 10:37am



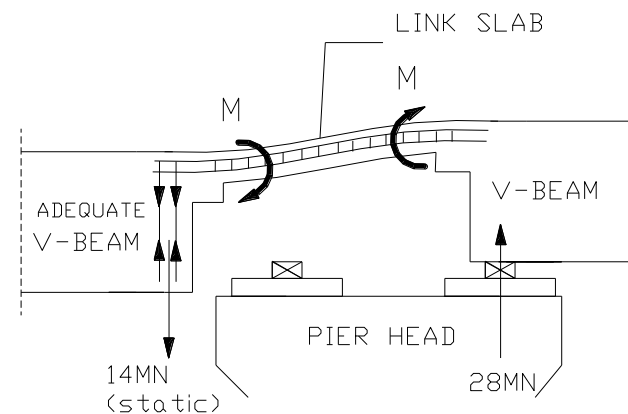
EDU destroyed



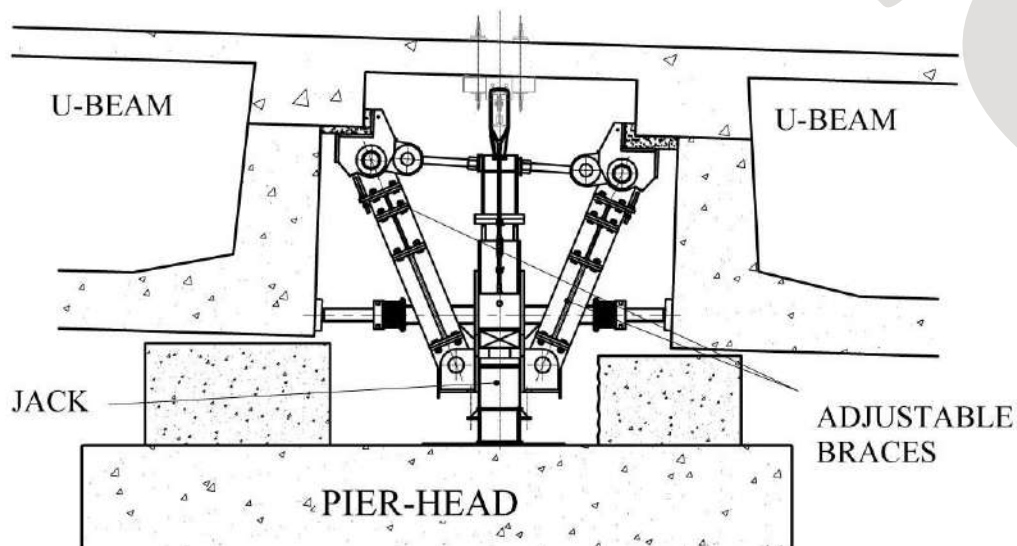
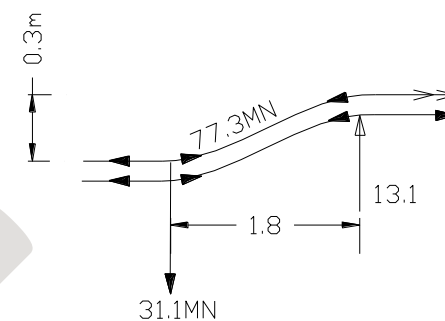
7 pile caps damaged



P 47 L - Pile 1122 south side



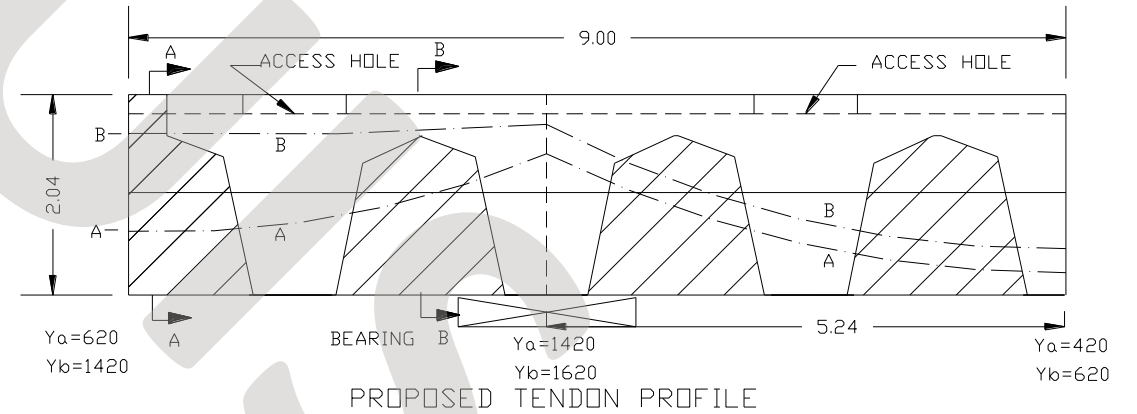
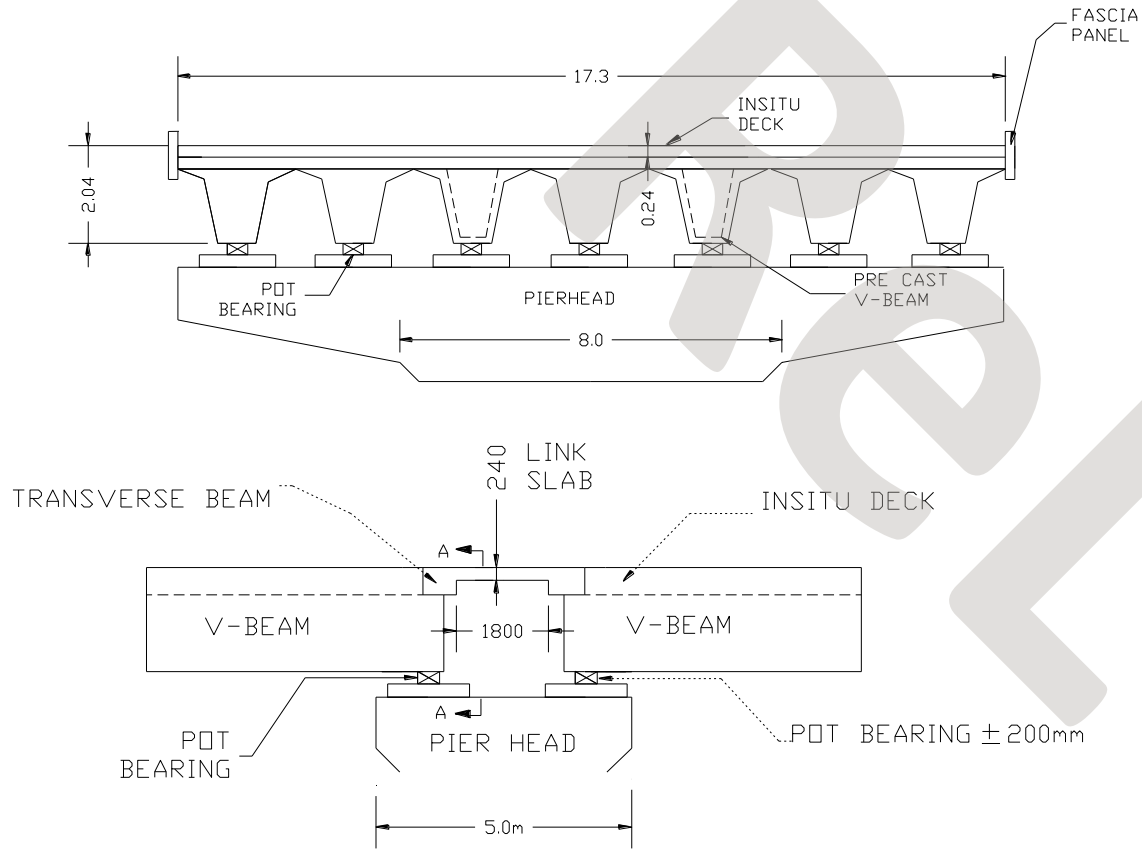
Capacity from linkslab moments = 12.7MN (Shear \square .K)
Capacity from large deformation:



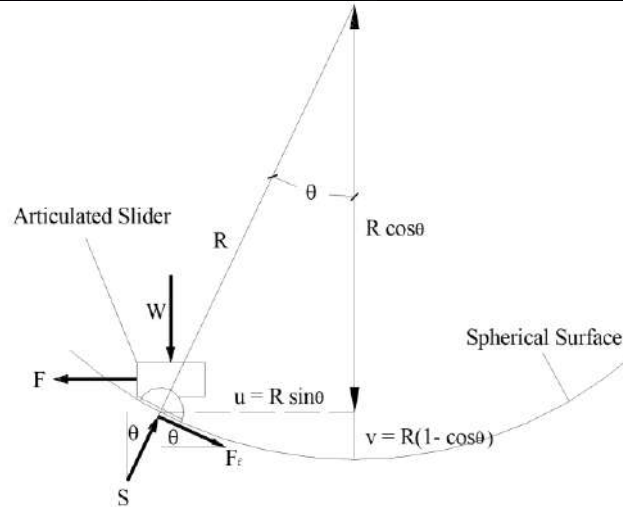
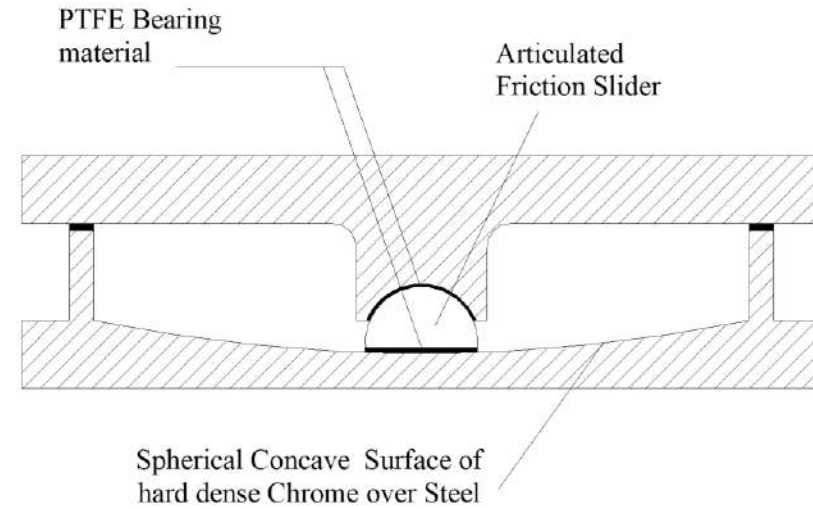
Lifting up and repositioning

nitigazione per la sicurezza dei ponti

Le linee guida per i ponti esistenti

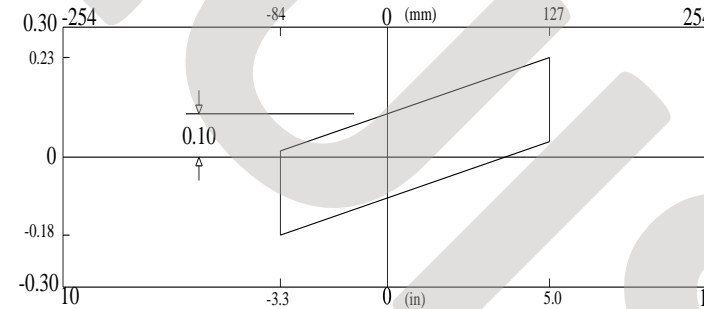


Transformation into continuous deck, transversal post tensioned diaphragm beams



$$T = 2\pi \left(\frac{W}{Kg} \right)^{1/2} = 2\pi \left(\frac{R}{g} \right)^{1/2}$$

$T = 2 \text{ sec. } \mu = 0.10$



$$K = W / R$$

$$V_y = W \times \mu$$

From 14 pot bearings and 1 EDU to 2 isolators



**Deck continuous over 800 m
+/-700/900 mm displacement capacity**



**The Viaduct did not collapse under a
very rare ground shaking.**

An acceptable performance?

Which performances should be met?

**How to correlate probability of
occurrence of ground shaking and
design performances?**

Isolation was used to retrofit the Viaduct

Was isolation needed?

For which purpose?

To what extent?



NOTE:

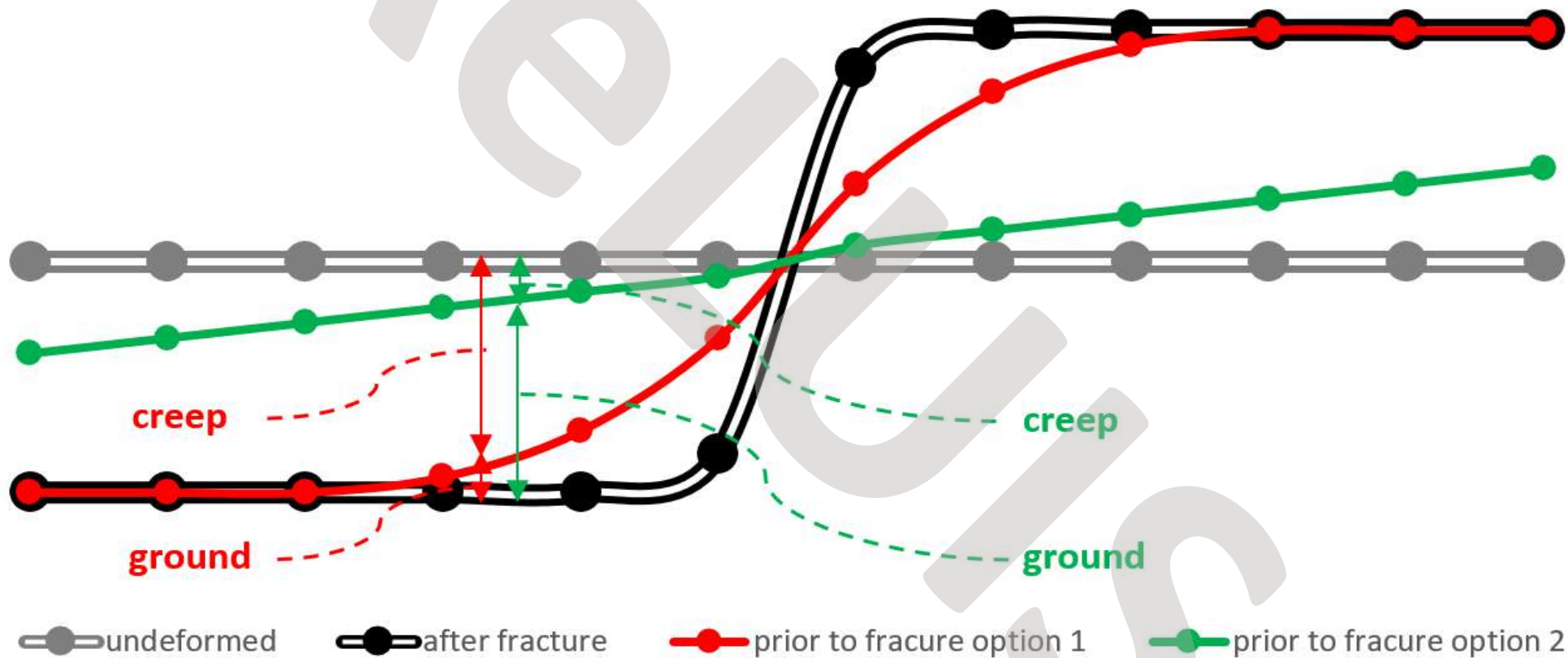
- Displacement capacity of tall piers is very large
- Base shear is dominated by pier mass response
- Displacement demand on bearings is larger than total displacement (second mode)
- For relatively short piers isolation is essential
- Isolation is fundamental for fault displacement,
- Partial isolation to be considered

Appendix

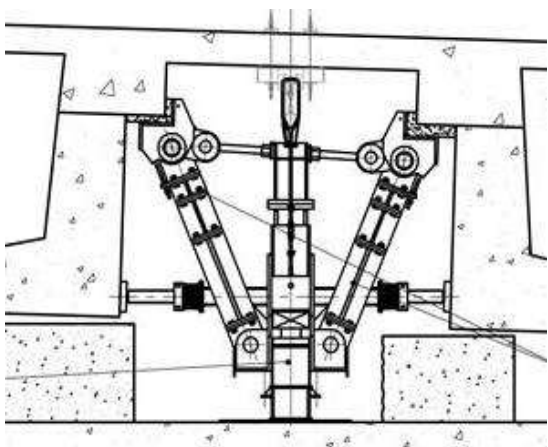
Hand calculations

for assessment, repair and strengthening
of the Anatolian Viaduct,
following the 1999 Düzce earthquake

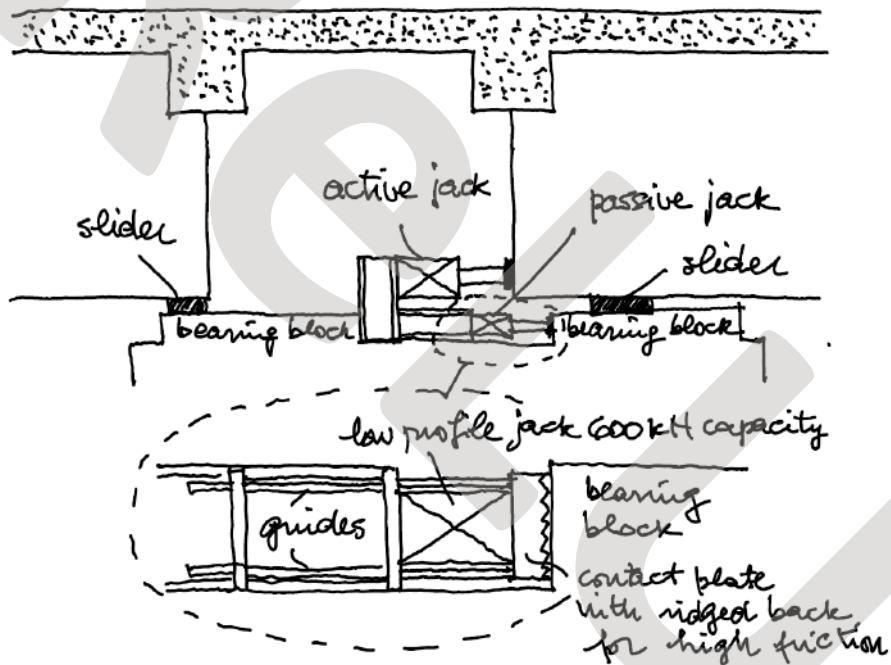
A viaduct like a series of strain gauges



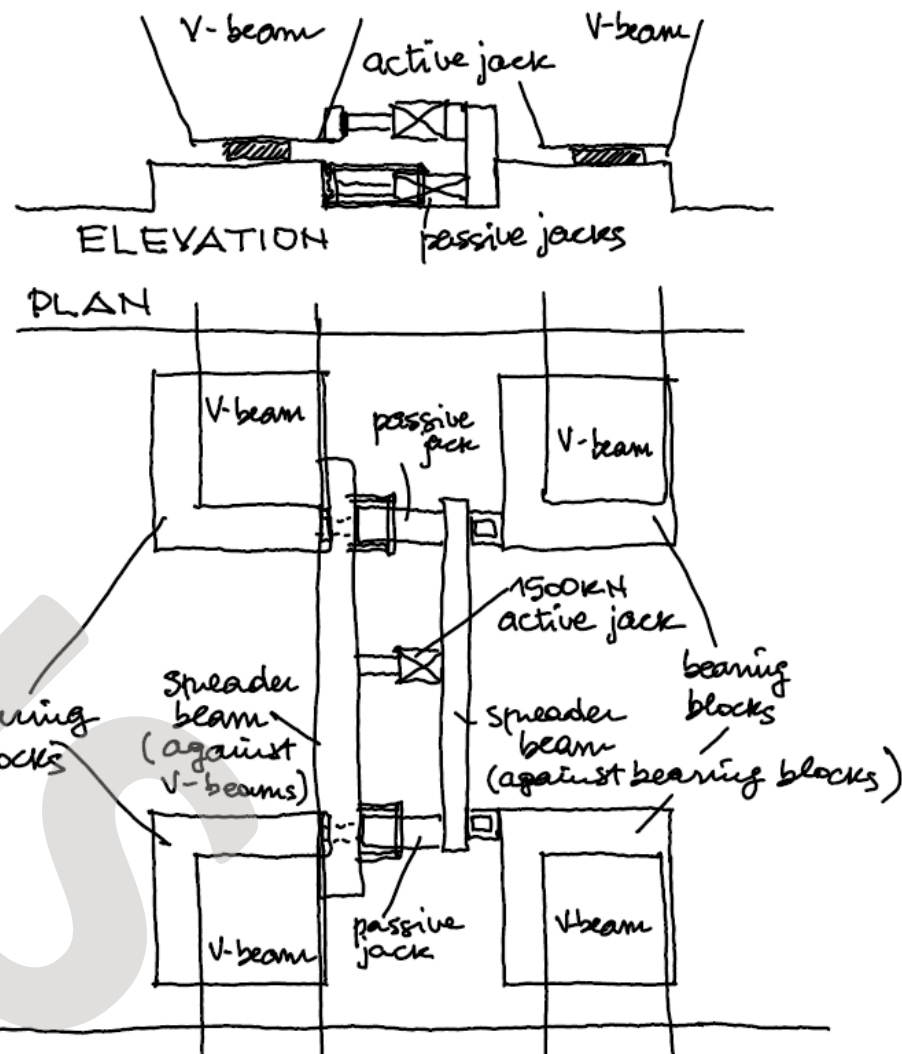
Repositioning the deck



Uplifting

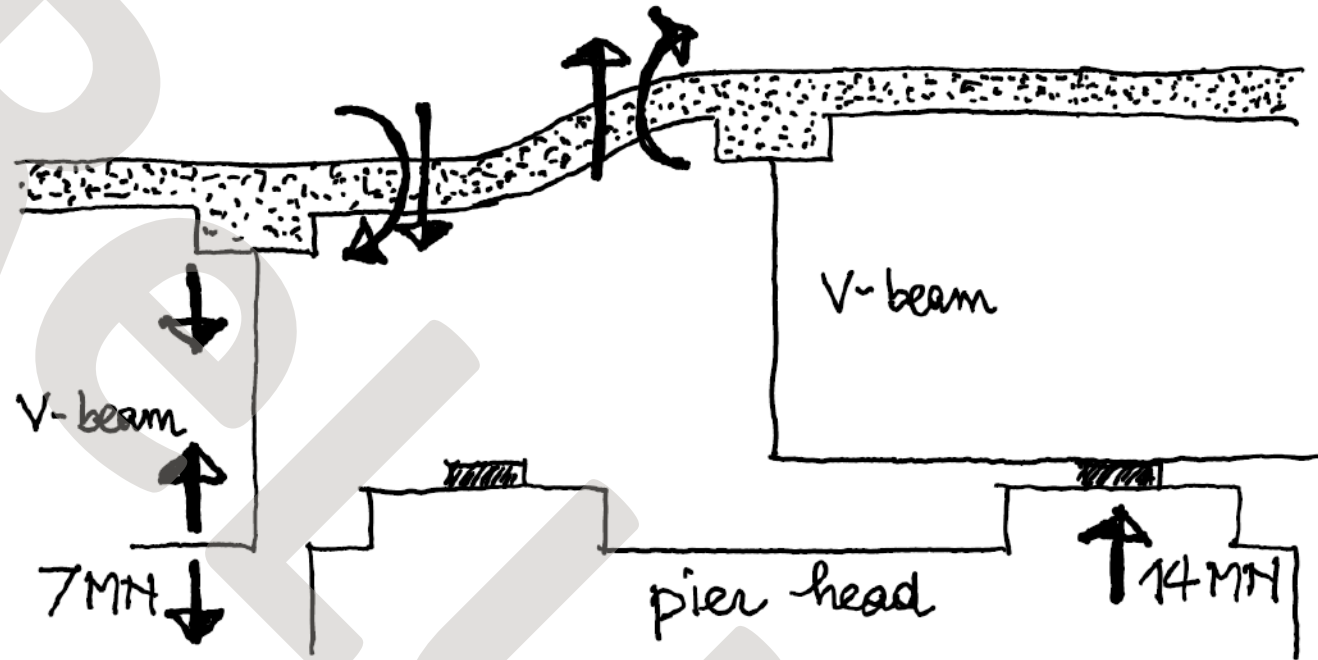


Moving longitudinally

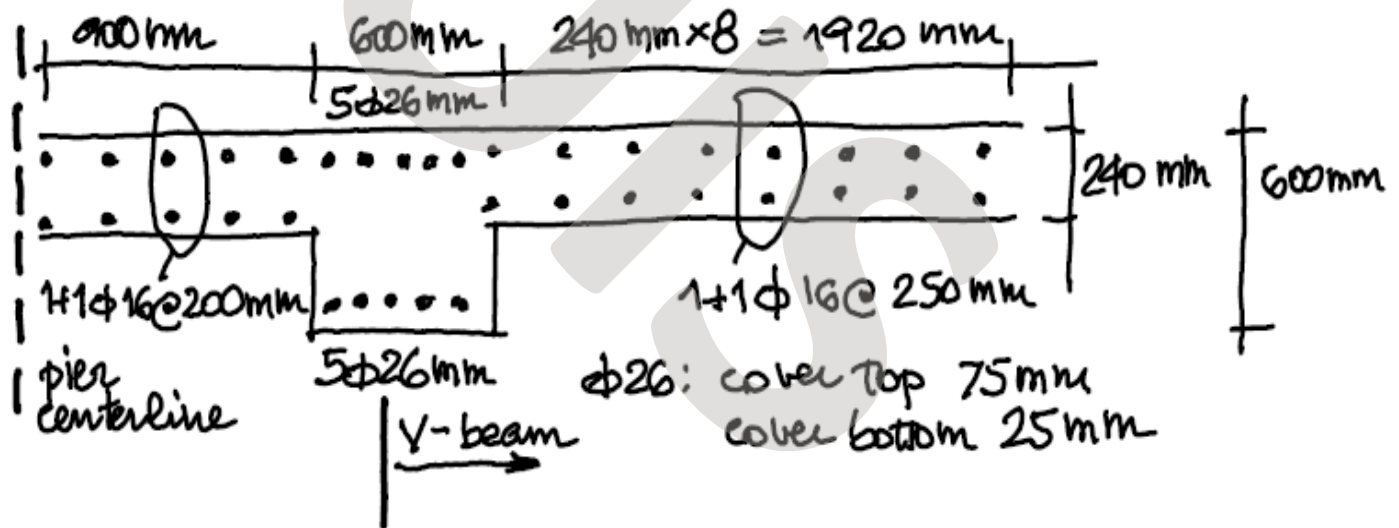


Moving transversally

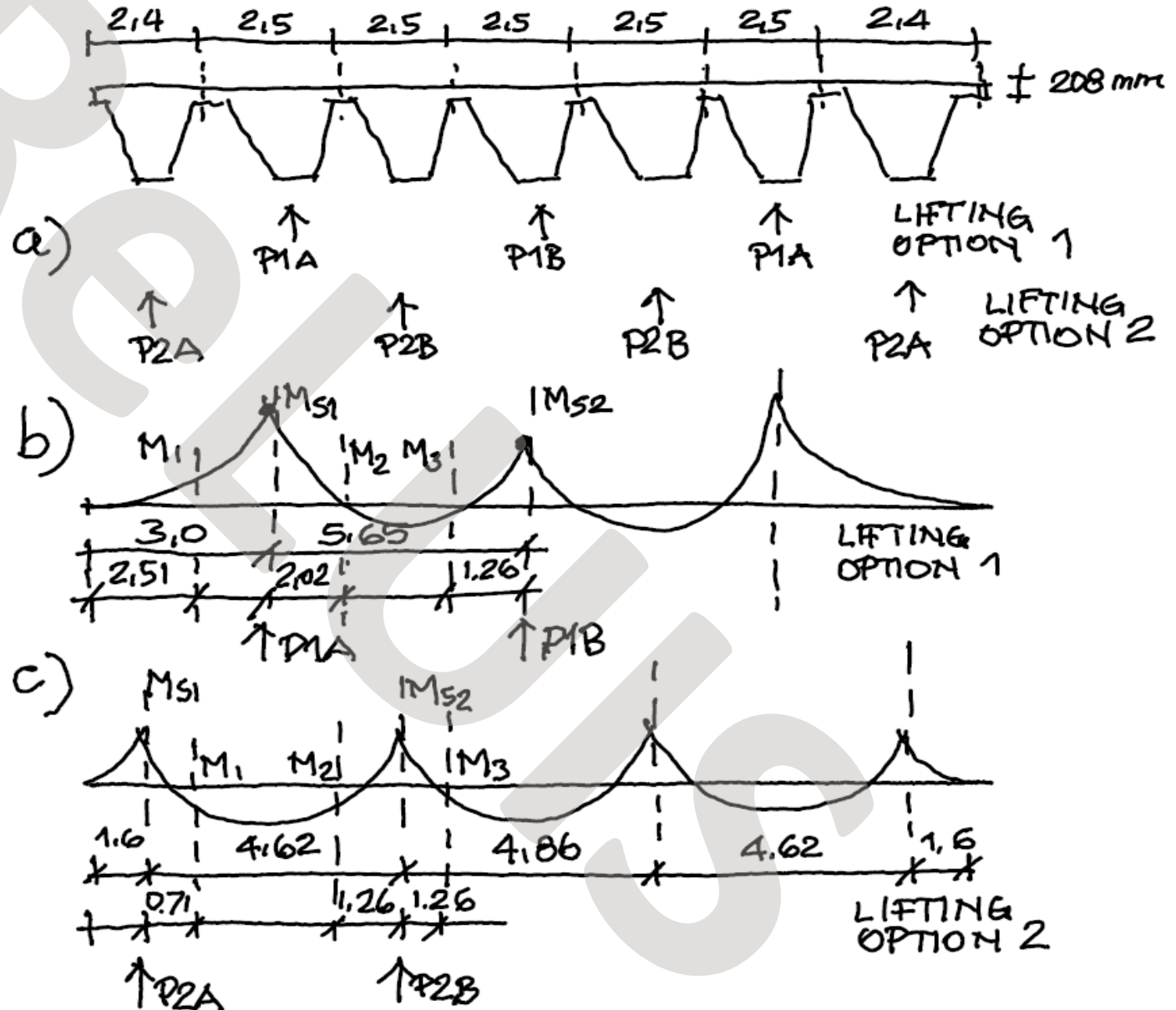
Equilibrium



Strength



Bending moments: Pushing on three or four points?



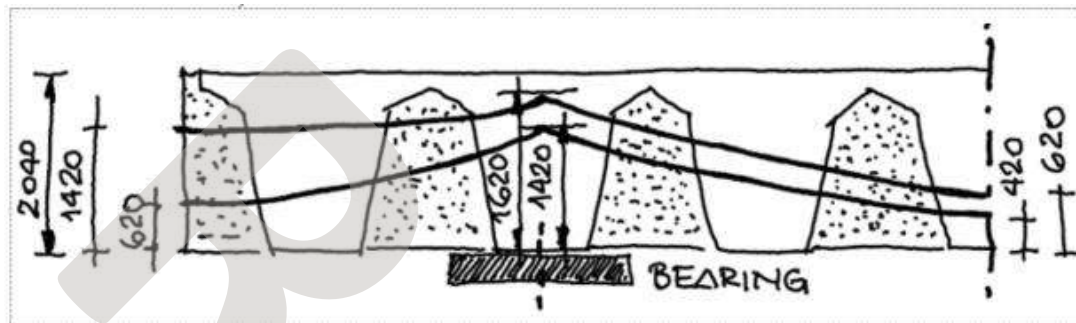


Fig. 5.10A Tendon profiles

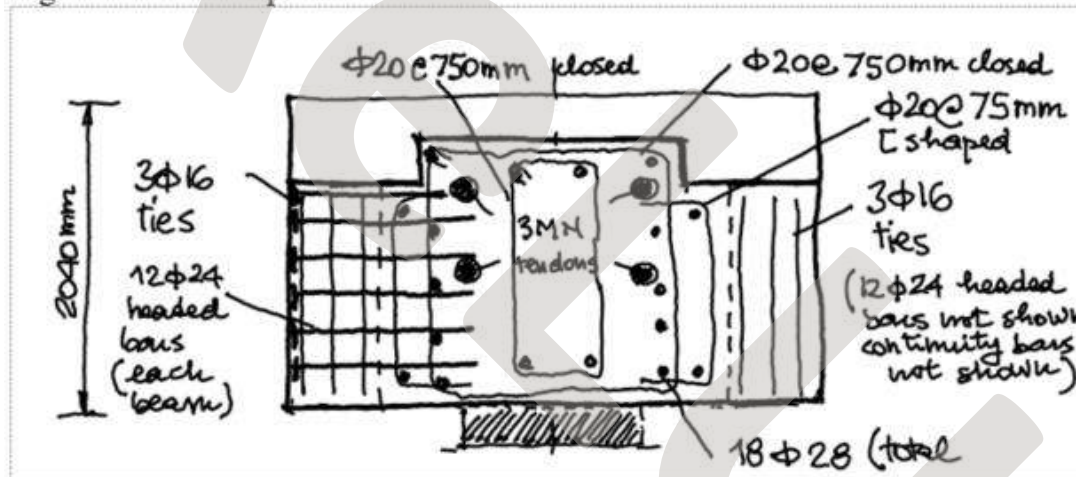
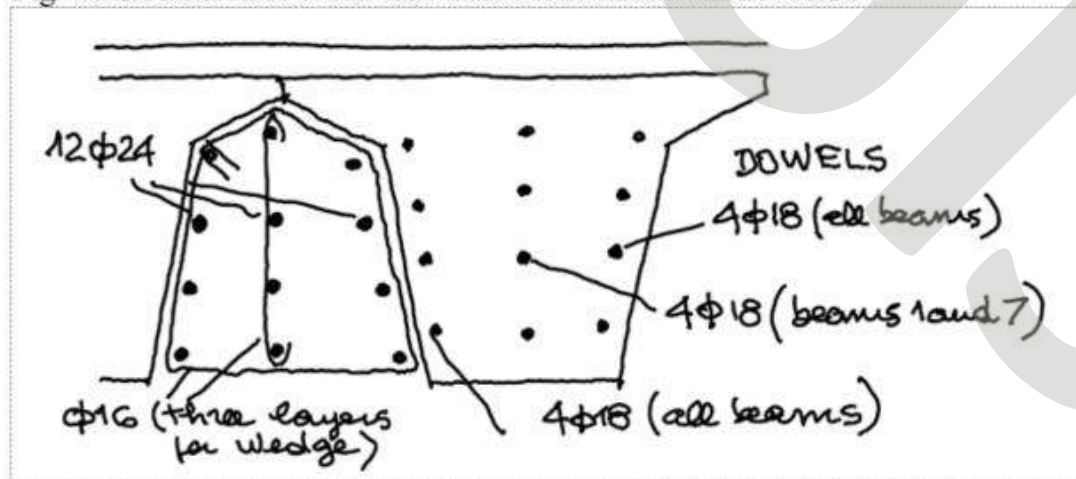
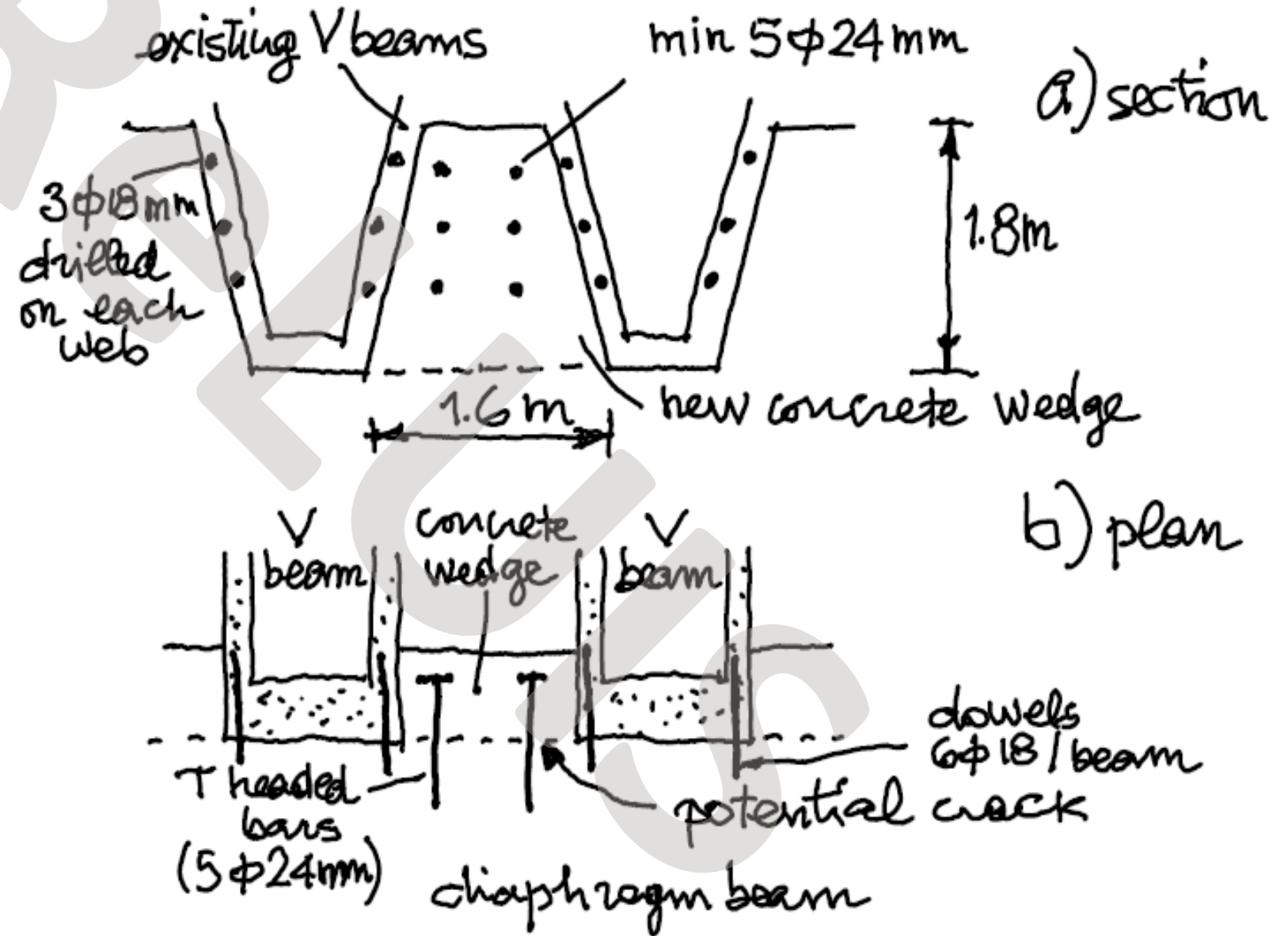


Fig. 5.11A Tendon location and transverse reinforcement details

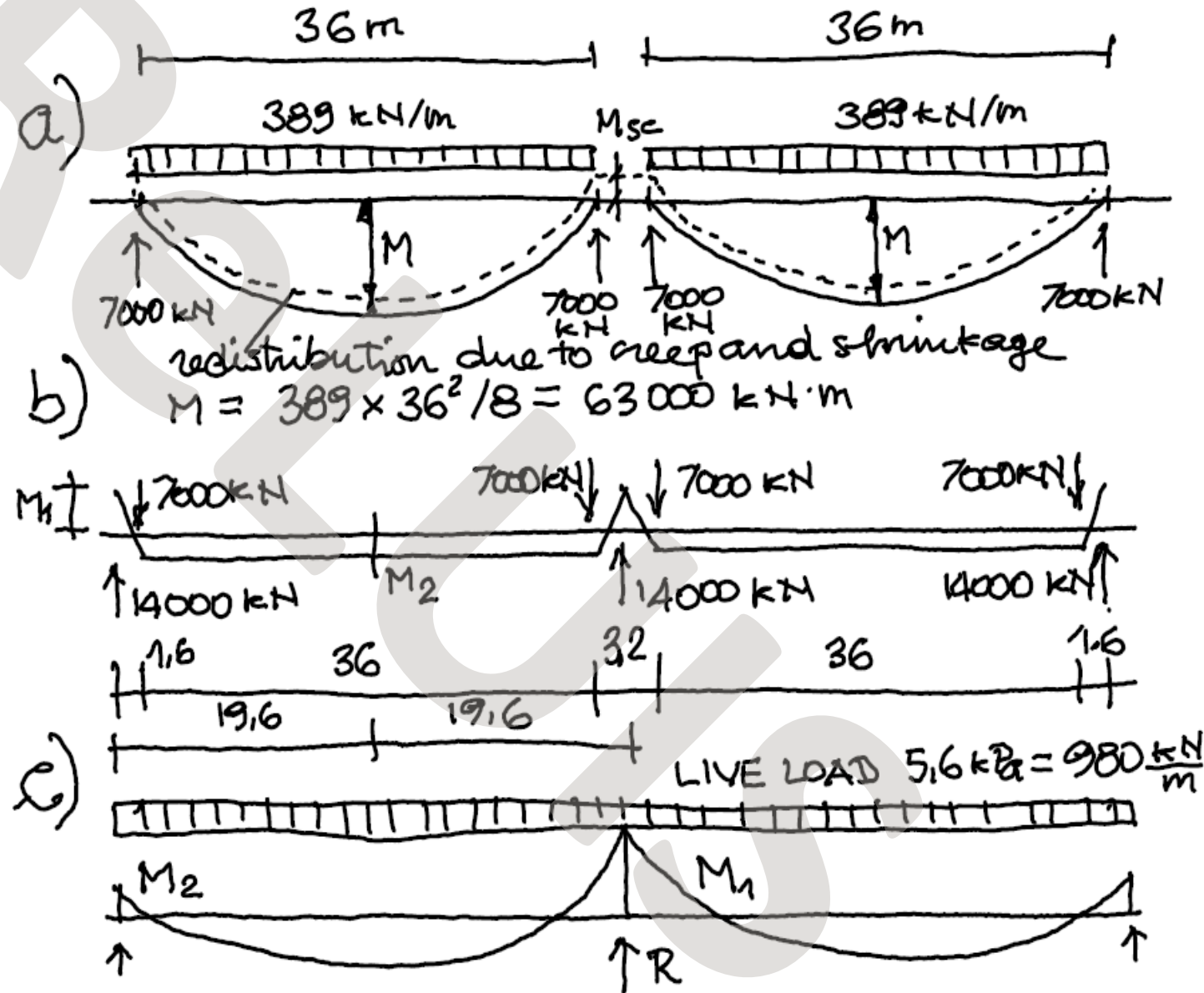


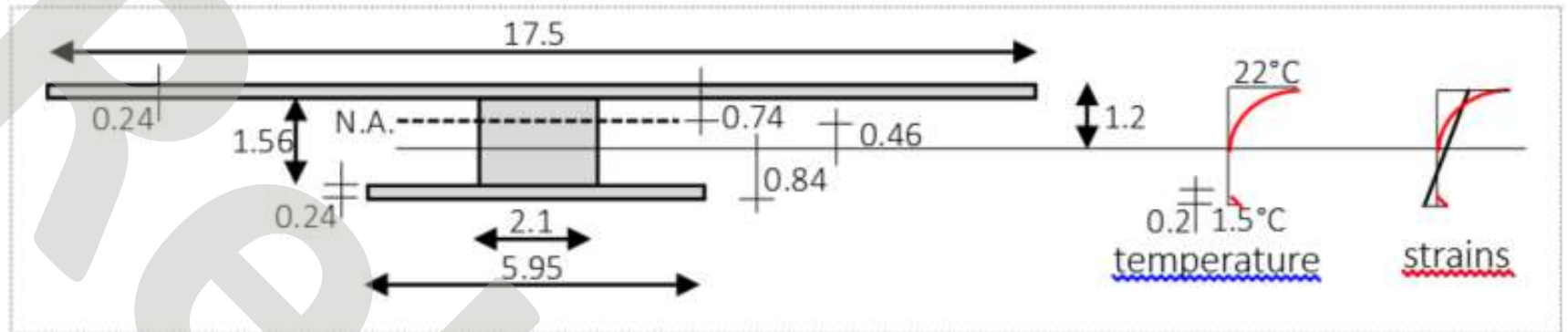
Post-tensioned diaphragm beams

Connection between existing V beams and new diaphragm beam

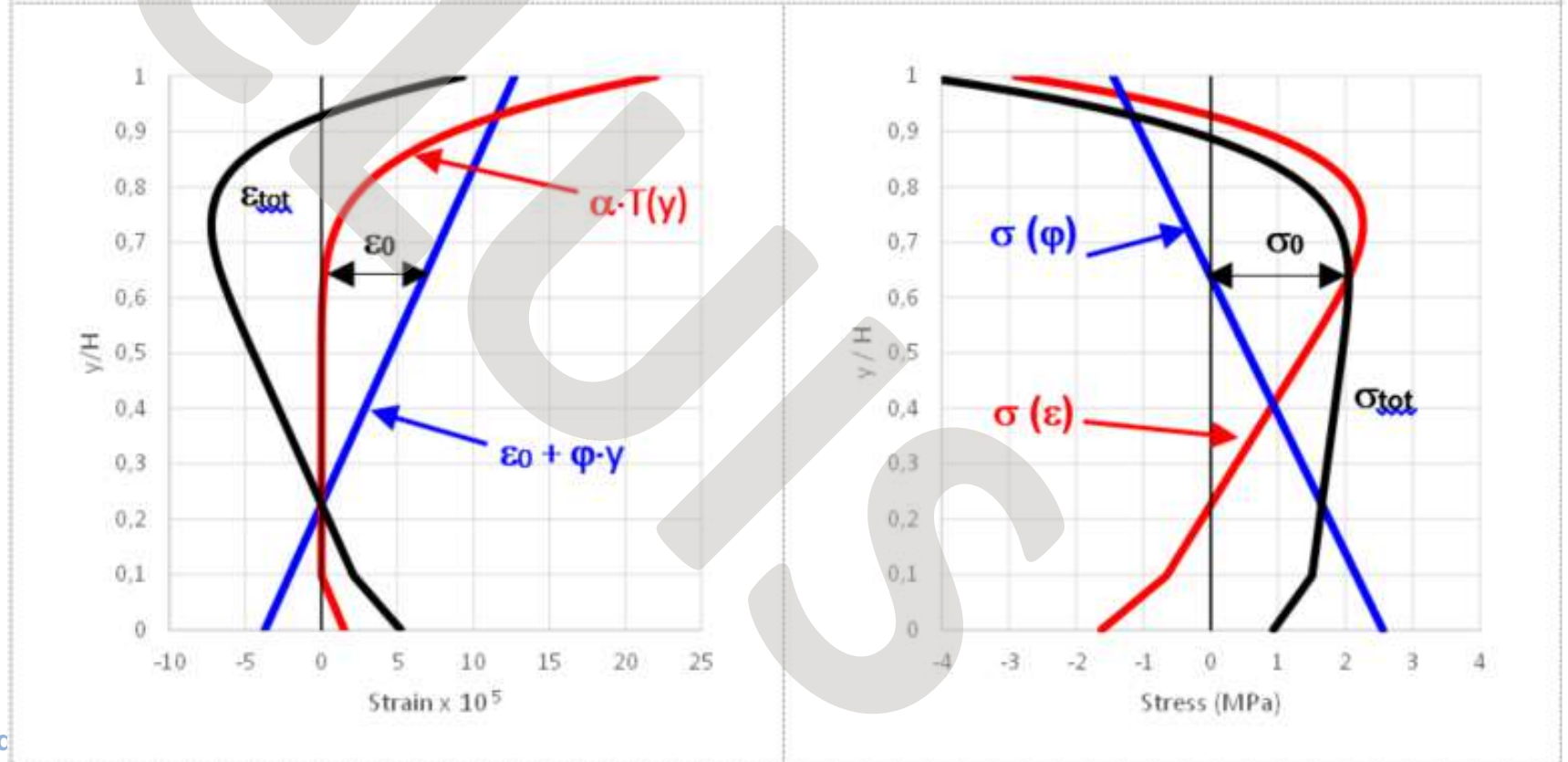


Bending moments due to continuity and support relocation



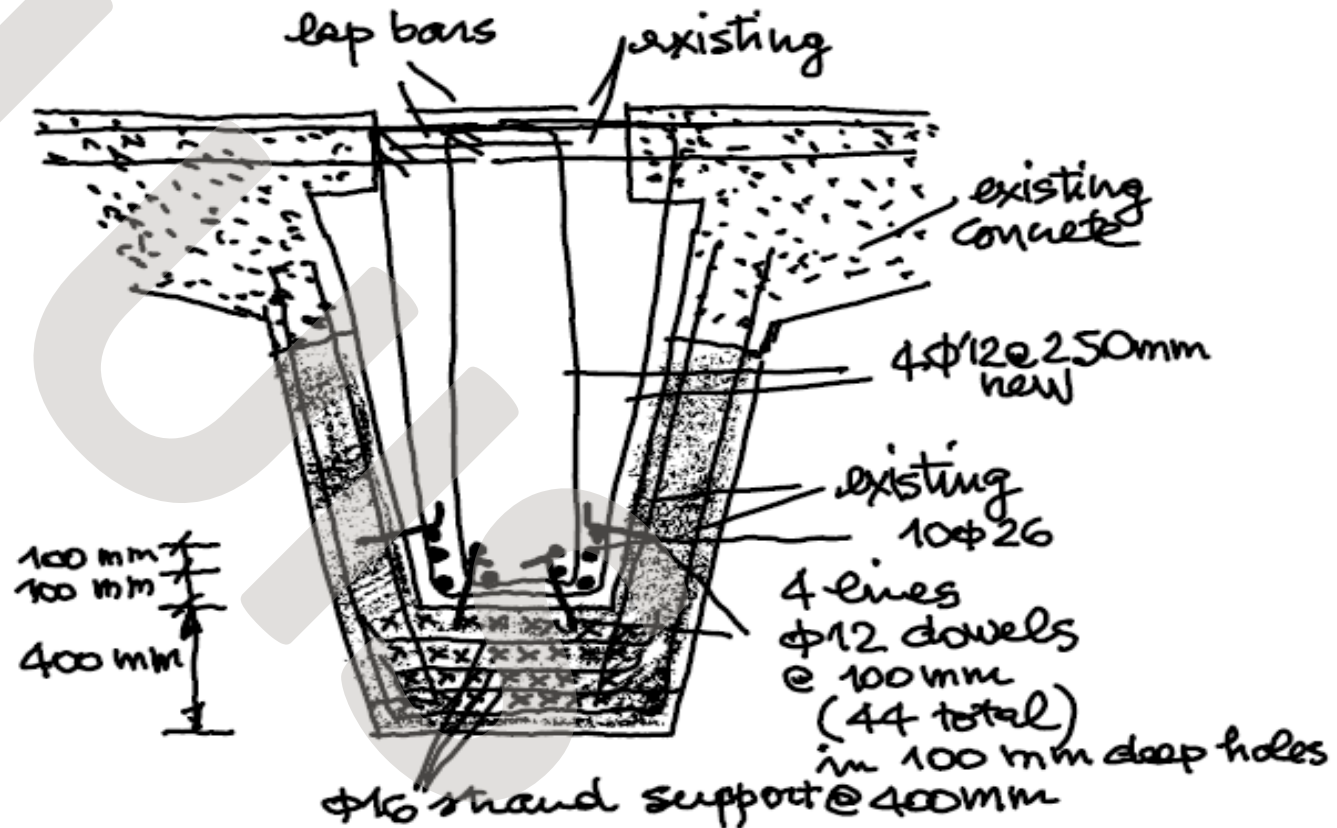
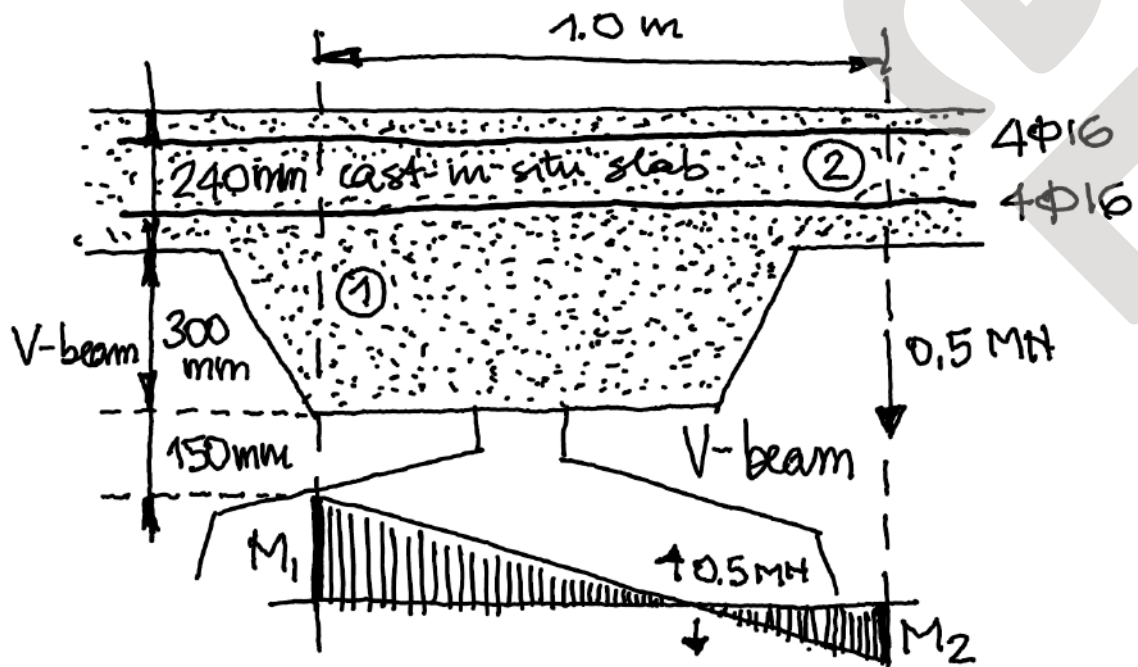


Internal stresses due to temperature and radiation effects

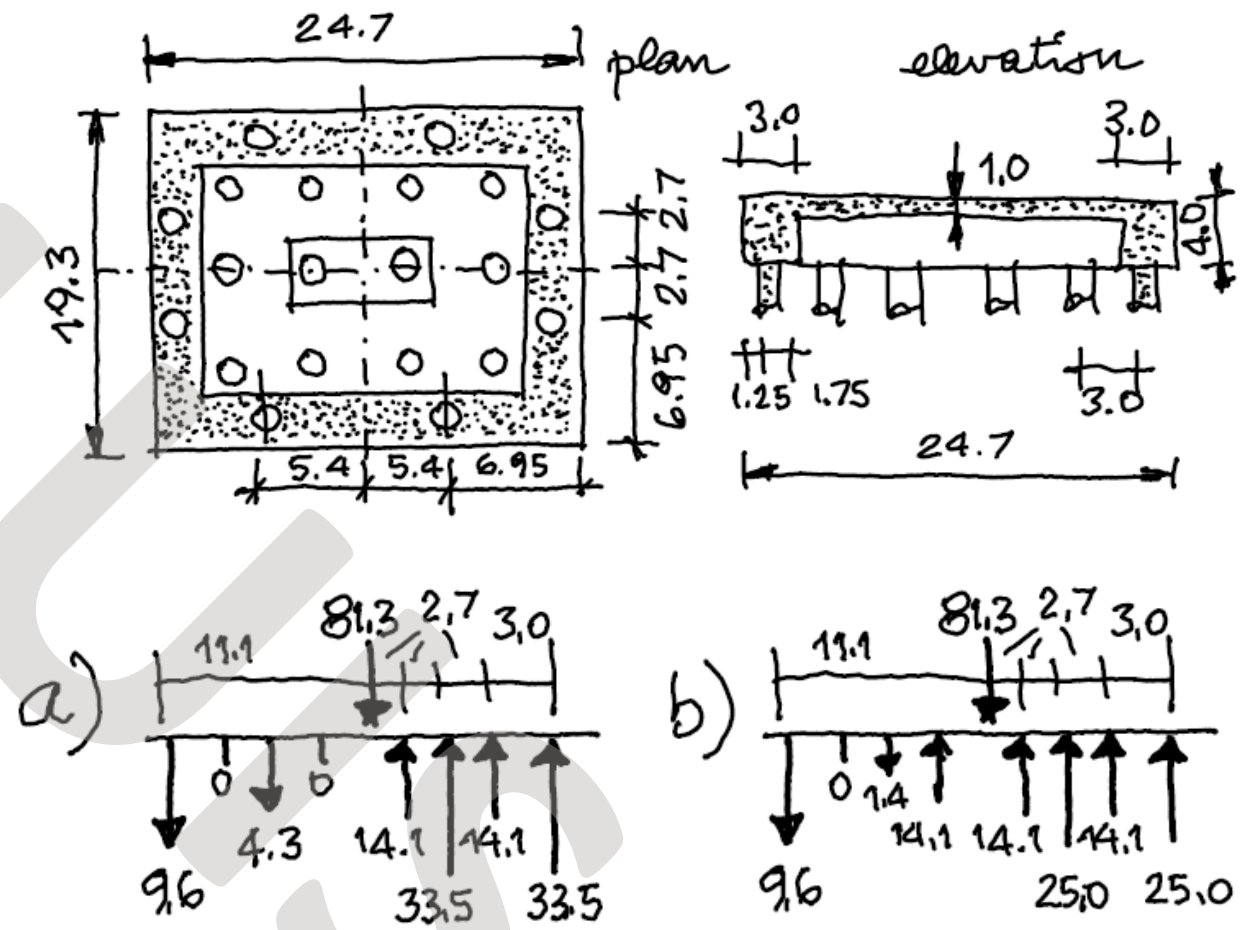
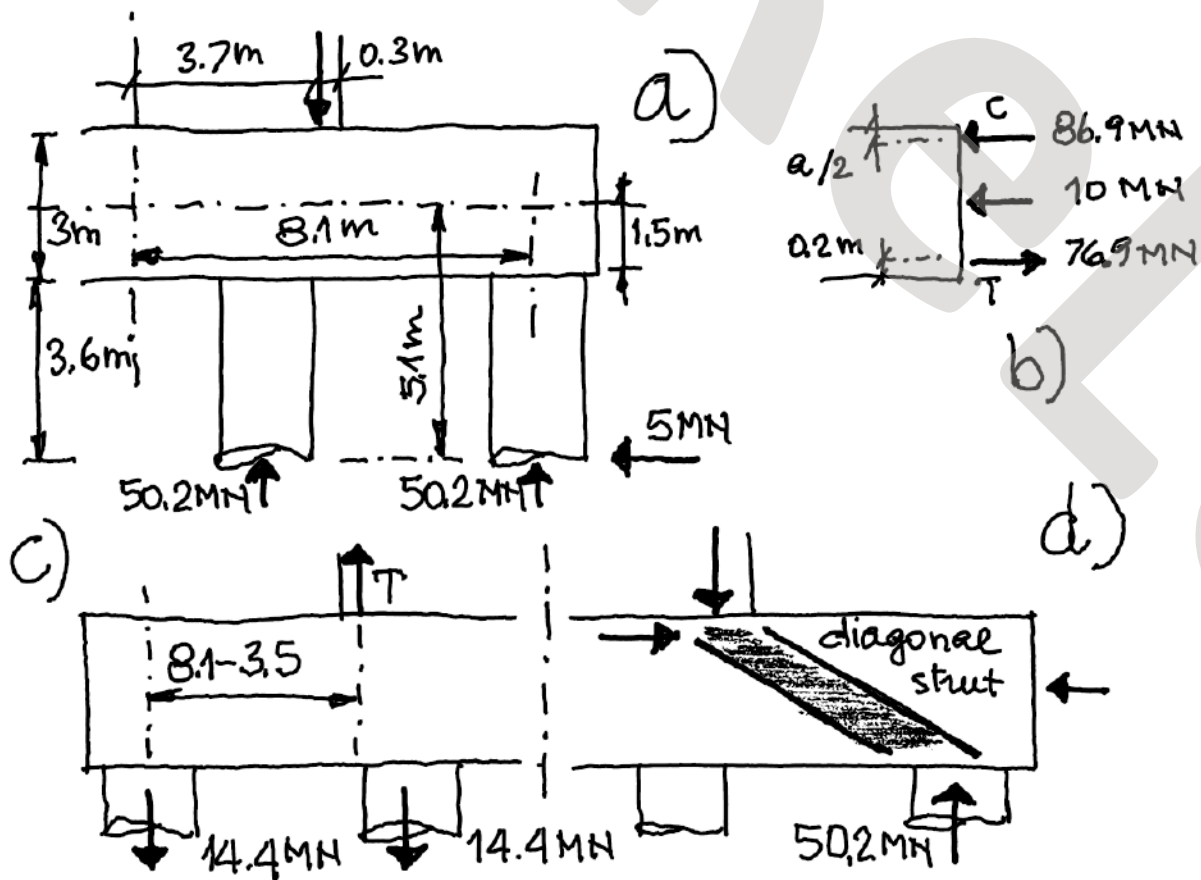


Moc

Strengthening of existing V-beams

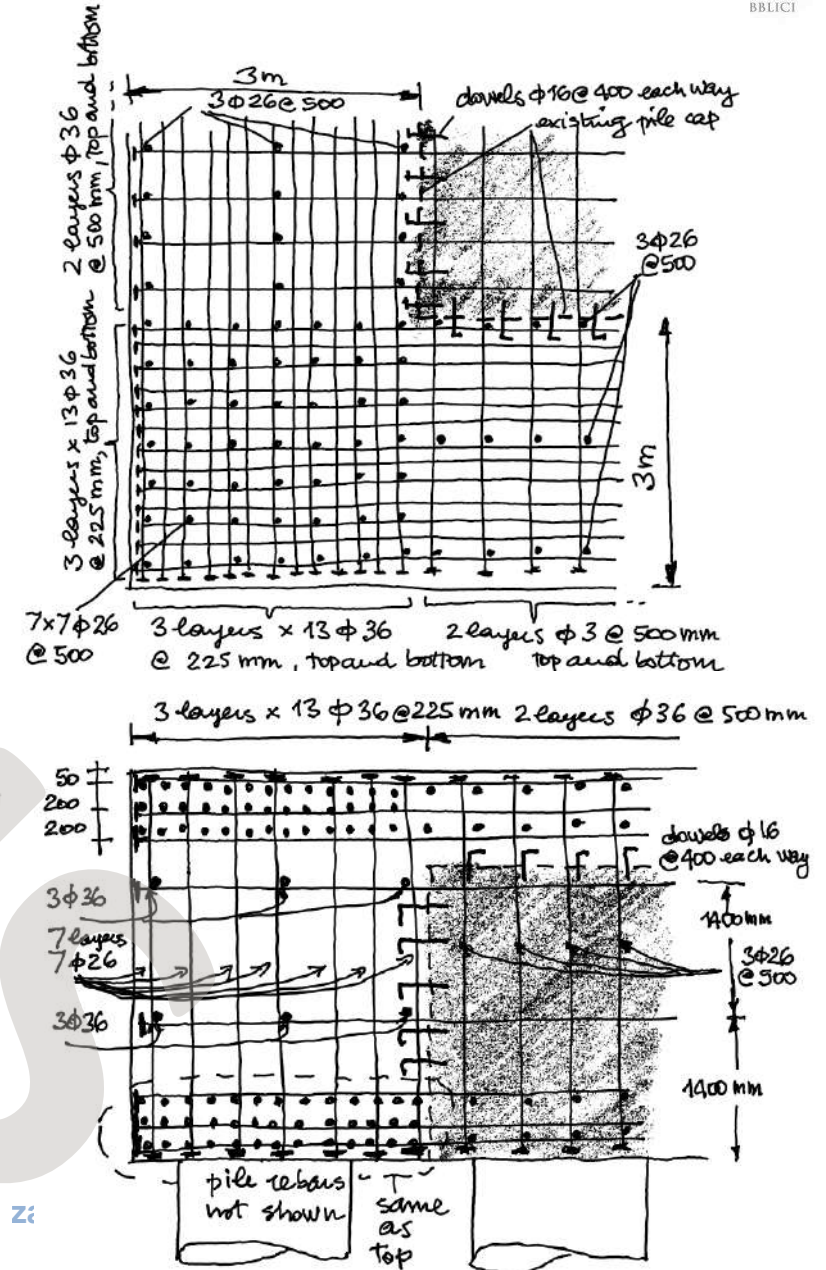
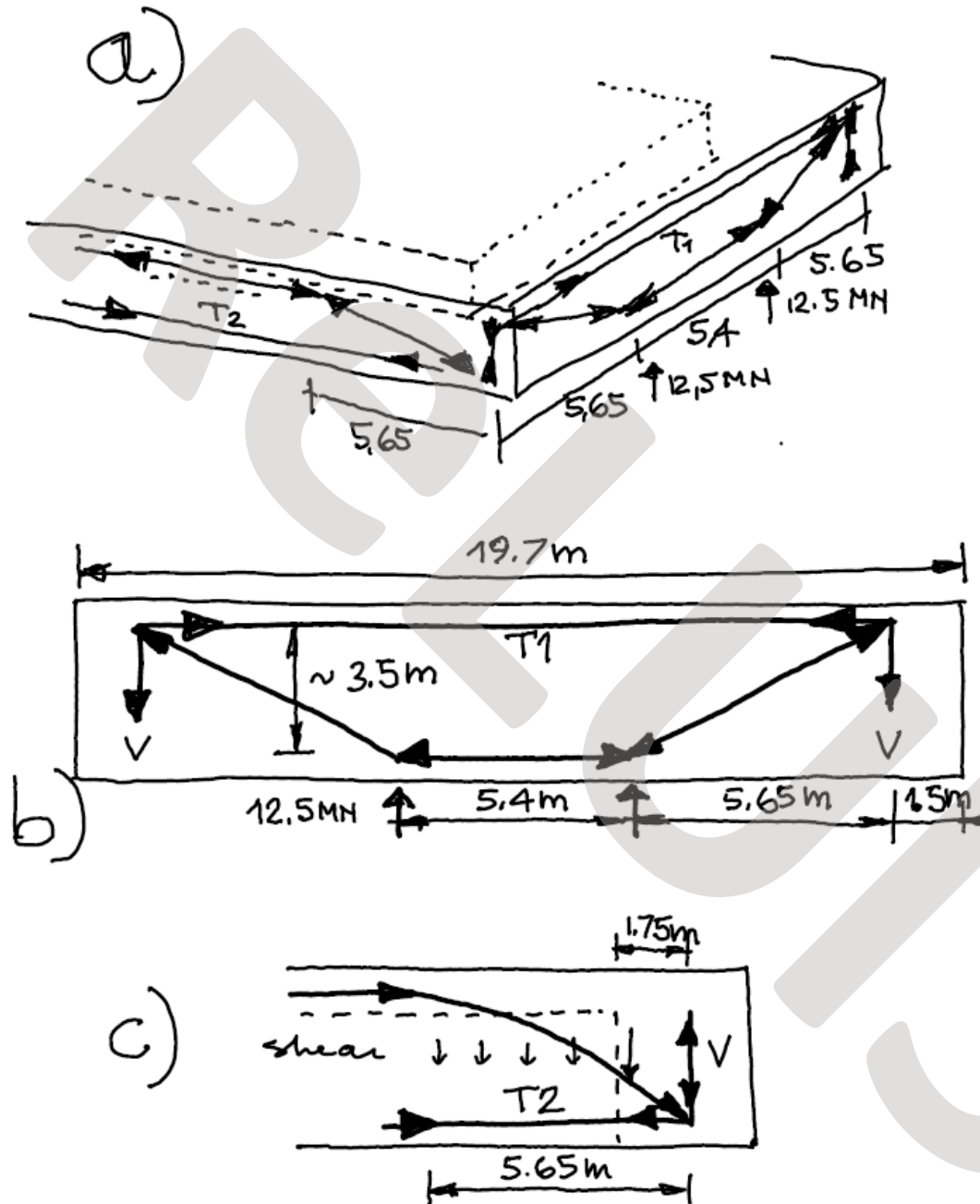


Foundations: equilibrium and strengthening

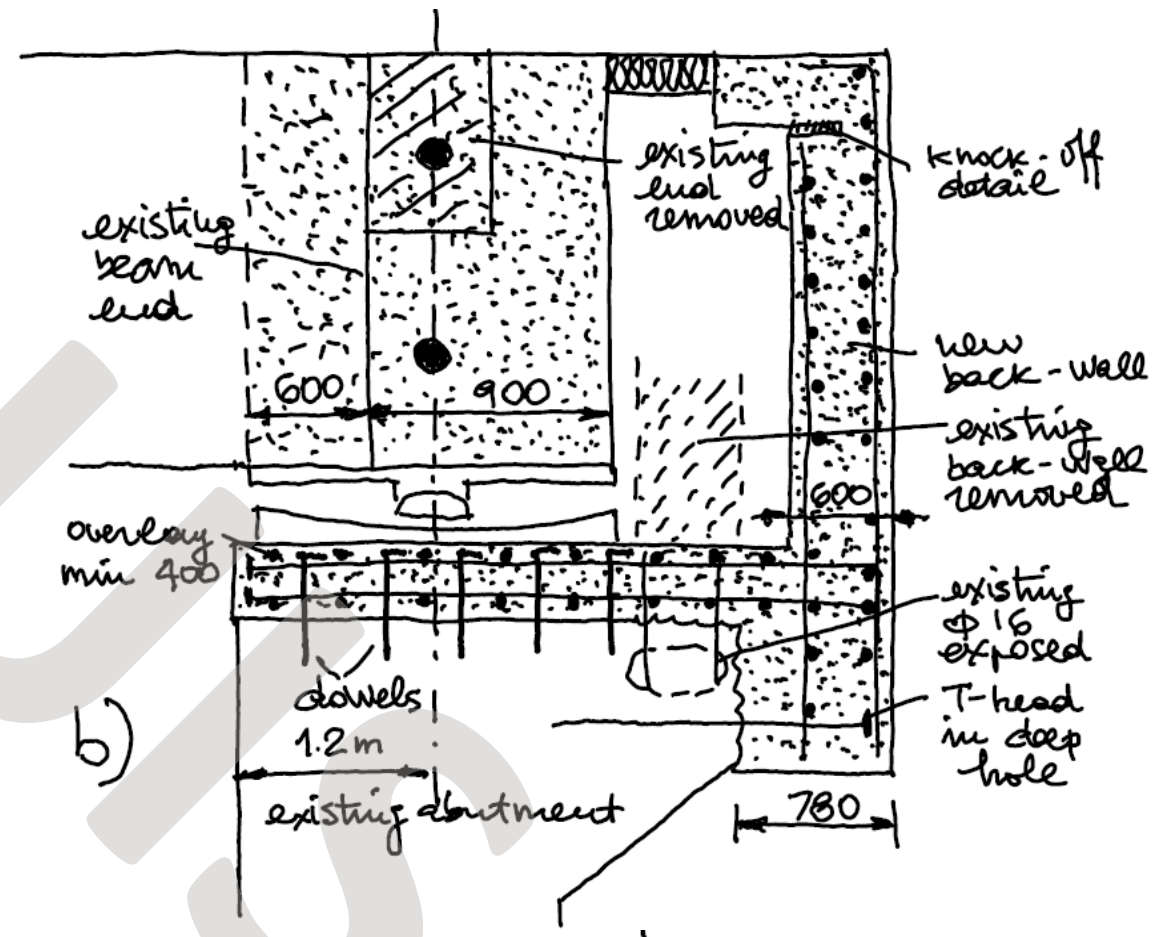
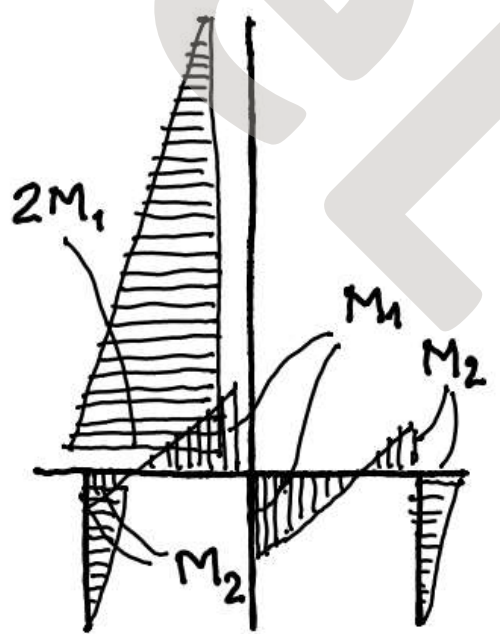
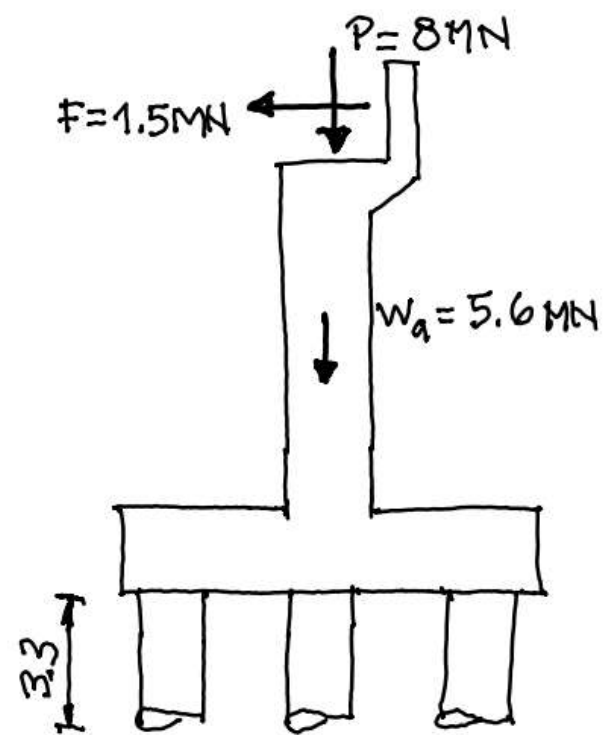


Foundations: strut and tie 3-D modeling

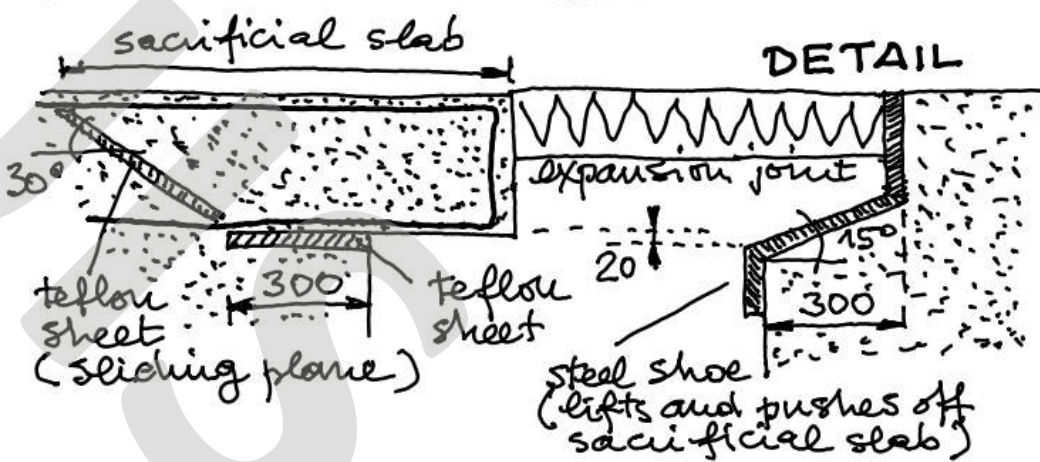
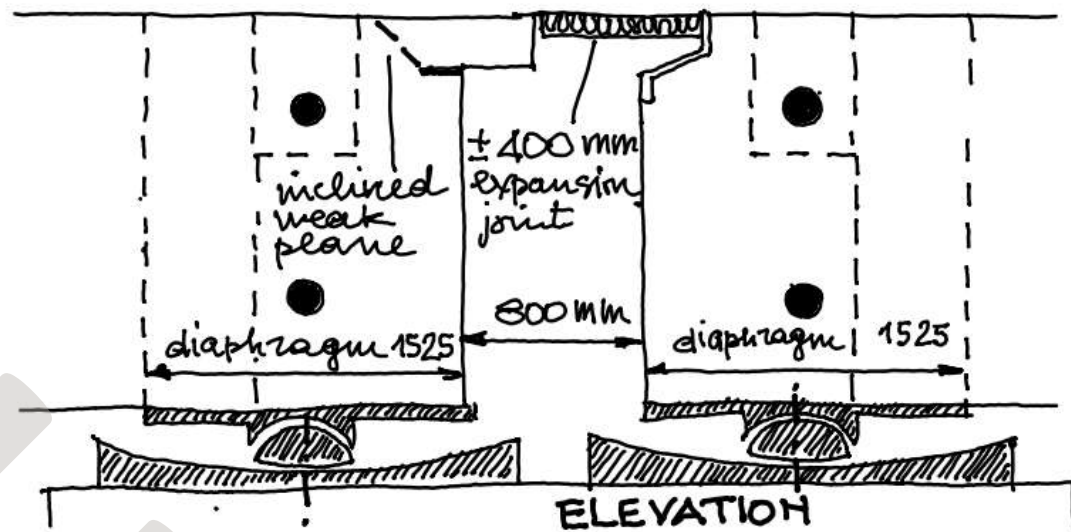
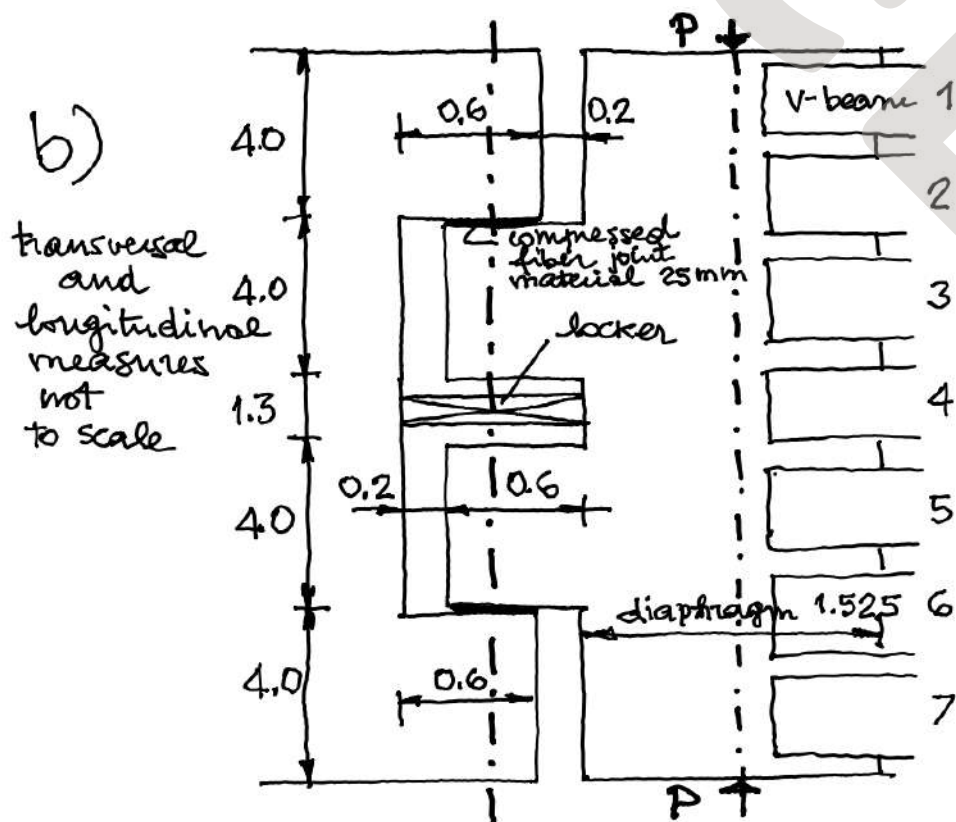
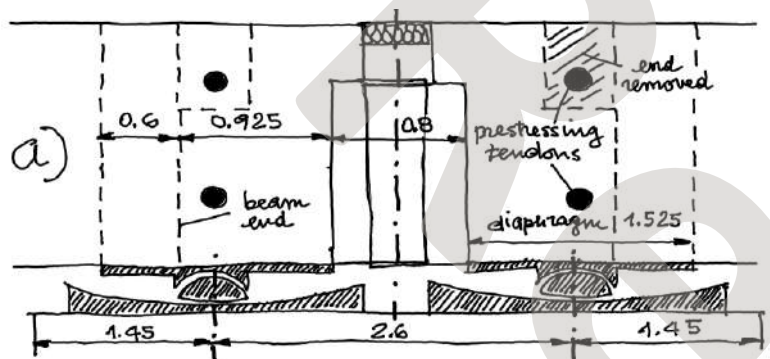
Reinforcement details



Abutments



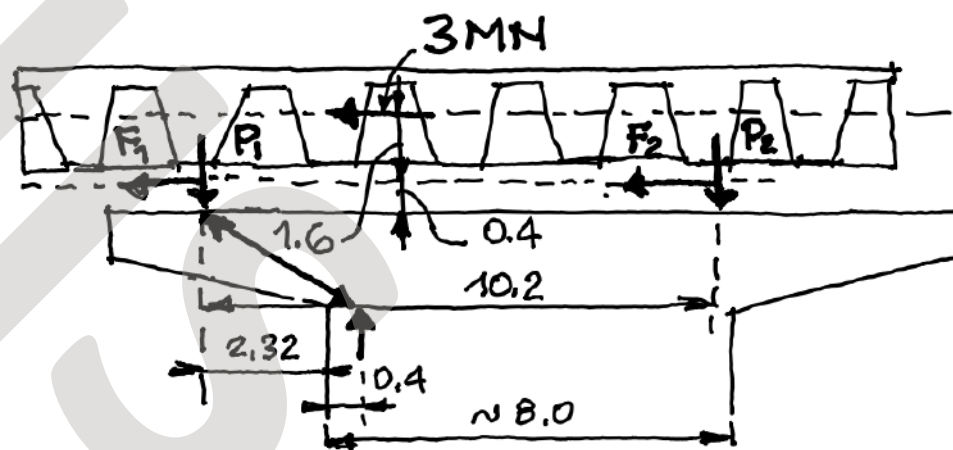
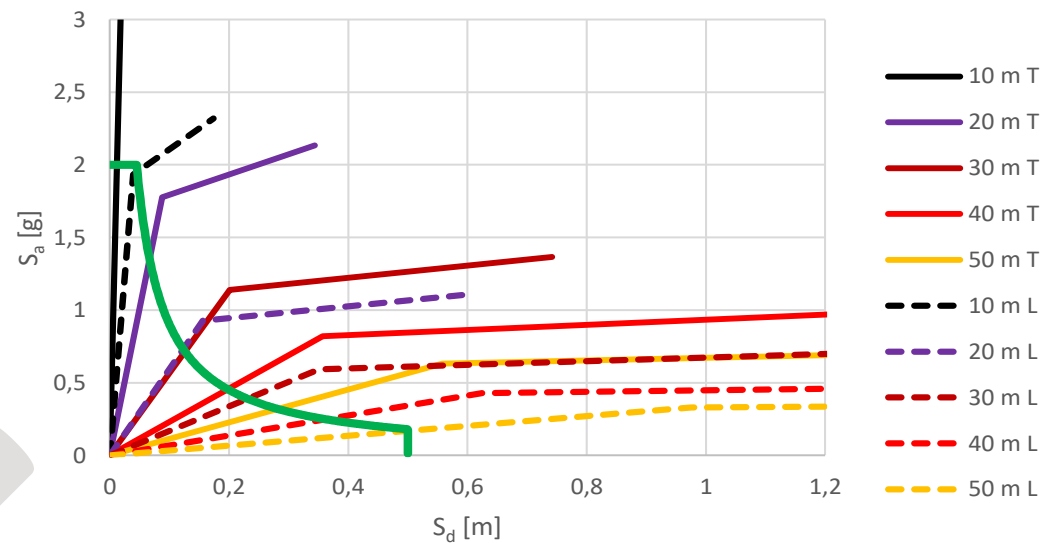
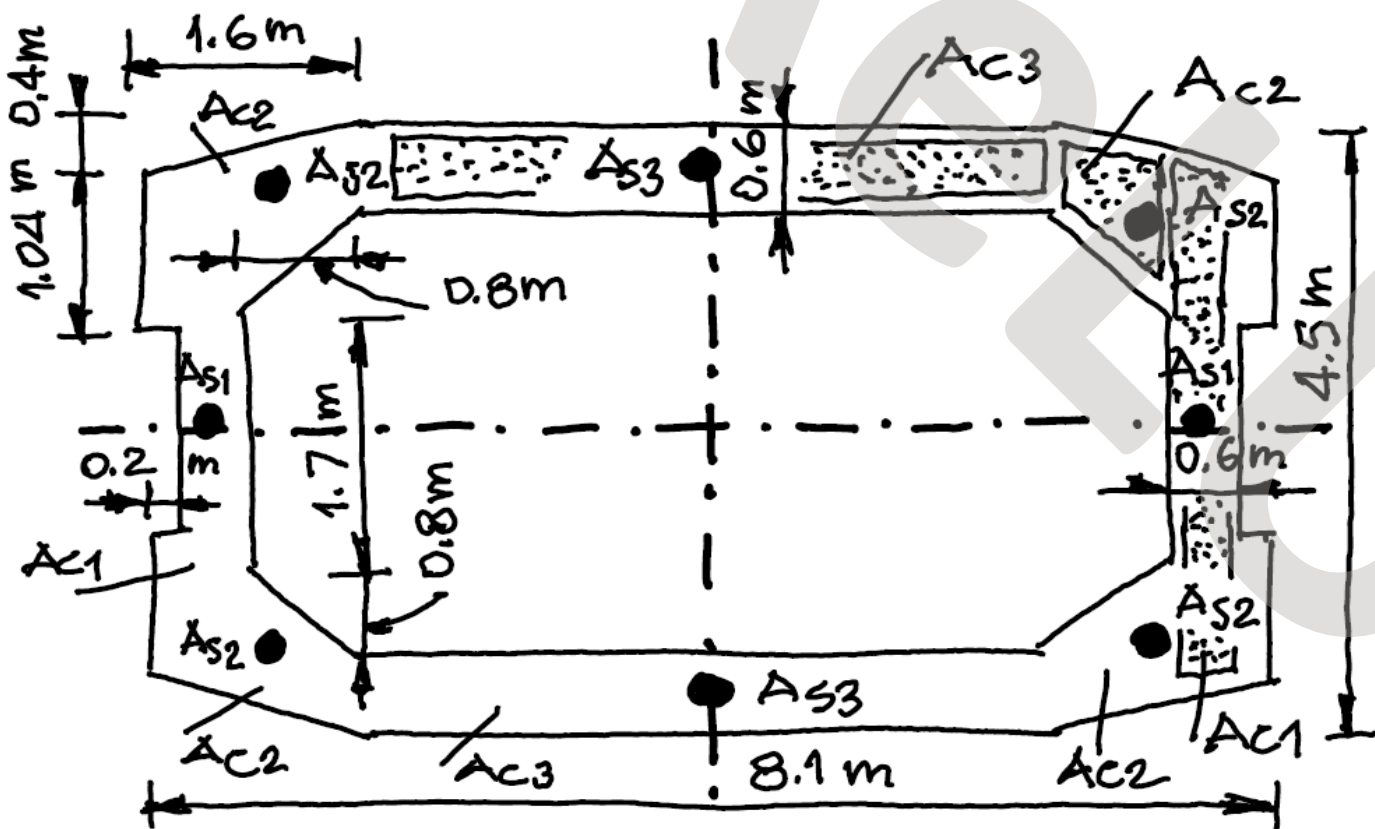
Movement joints



eventi di mitigazione per la sicurezza dei ponti

guida per i ponti esistenti

Piers



**to find the answer
you must know the answer**



"Fighting Against Time"

THE DUSTY DOLLS

