A sustainable approach to existing structures

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Assessment of existing structures

One of the most important tasks in today engineering practice

Structural engineer demand

Assessment of actual safety level of existing structures

Extend residual life of existing structures

Social and economical constraints

Sustainability principles

Assessment should be done by use of "codes"

Use of codes conceived for the design of new structures leads to a relevant degree of conservativism

Excess of conservativism implies negative environmental, social and economical consequences

Structures substantially fulfilling the relevant limit states can be judged as unsafe / unsatisfactory

Requirement of large amount of investments for their

Upgrading Demolition Reconstruction

There is a need to establish for the existing structures

New principles

New design / verification methods

Beyond the scope of the design codes for new structures

It becomes fundamental the approach for the uncertainties treatment

New structures

Essentially based on information gained by experience

Existing structures

Acquisition of more or less detailed information on a specific structure is fundamental

- effect of construction process and use
- alteration
- deterioration
- misuse

Treatment of uncertainties in existing structures

Type 1 uncertainties

Uncertainties due to inherent natural variability

Aleatory or random

The scatter that we observe can be measured and described in objective terms

Type 2 uncertainties

Model uncertainties and statistical uncertainties

Epistemic

Related to lack of knowledge (intuition / expert opinion / engineering judgement)

In engineering problems

↓

Bayesian approach to treat at the same way the two types of uncertainties

Random uncertainties

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Frequentistic interpretation

Epistemic uncertainties

↓

Degree of belief interpretation

Random uncertainties

- Materials mechanical properties
- Actions
- Model uncertainties in description of material properties and actions

May be modeled by means of continuous random variables

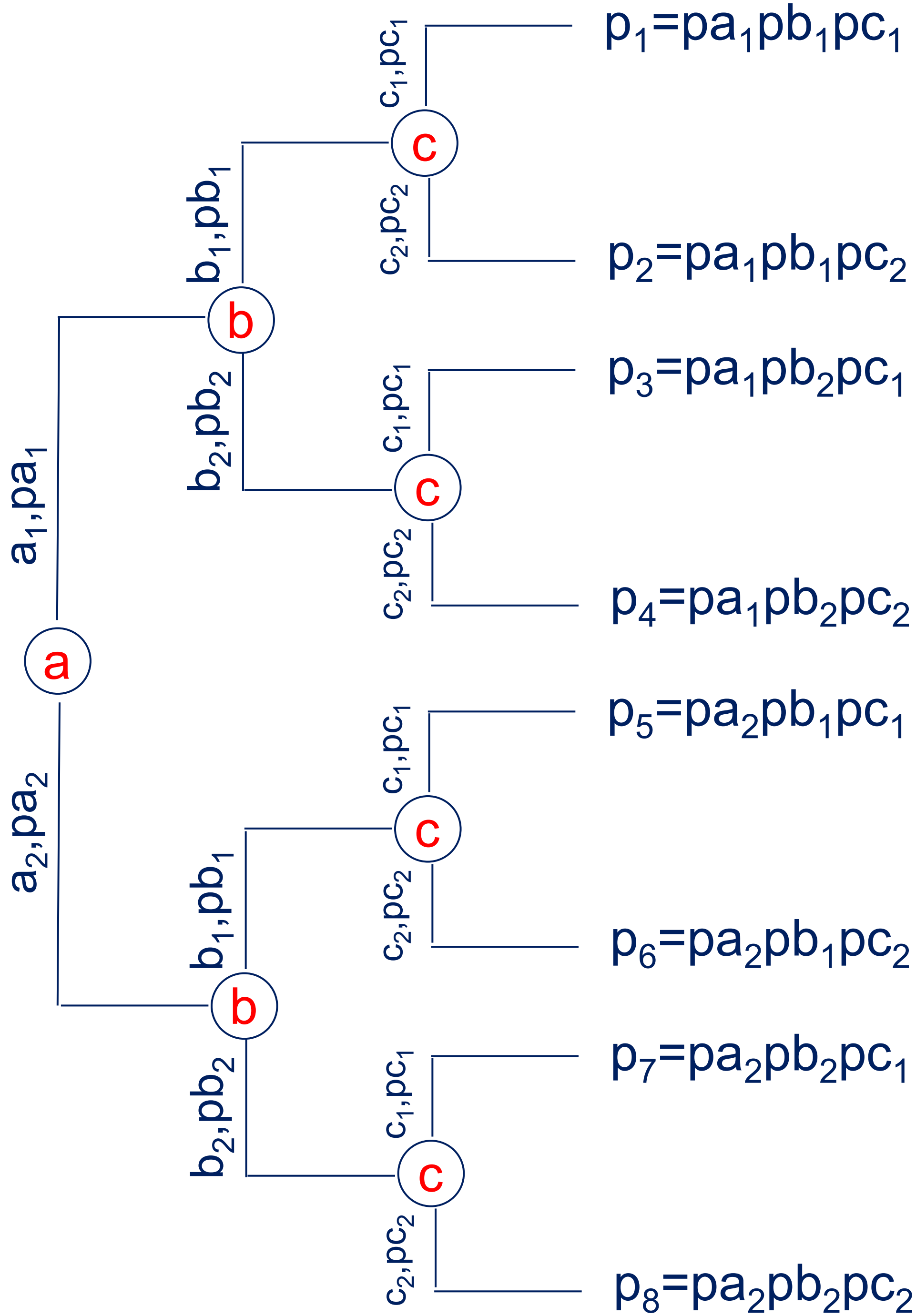
- Probability density function (PDF)
- Cumulative distribution function (CDF)

Epistemic uncertainties

Lack of structural knowledge

Choice between alternative resisting models

May be modeled as discrete random variables (described by a PDF) or, better, by means of event tree



- Variables a/b/c are supposed to be independent

- Weighted mean of probability of different legs with probabilities p_i

Target reliability levels of existing structures may be modified respect to the corresponding ones assumed for new structures, for

Economical considerations

Social considerations

Sustainability considerations

Economical considerations

↓

Larger value of incremental cost between acceptance and upgrading in existing structures respect to the corresponding ones in new structures design

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Design rules for new structures are conservatives

Social considerations

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In case of intervention on existing structures, the necessity of displacement of occupants and activities and limitations in case of heritage values

↓

Such considerations do not affect the new structures

Sustainability considerations

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Reduction of waste and recycling
reduction of energy consumption

Modification of target reliability values

Current design code values and practice

Economic optimization criteria

Minimum expected overall cost

Type and importance of structure

Possible failure conseque nces

Social and economical criteria

BUT !!

Human safety shall always be considered, at the same level of risk accepted for new constructions (10^{-5} per year of maximum probability to become victim of structural failure)

Target reliability levels proposed in ISO 13822:2010 and by Vrouwenvelder and Scholten

Limit states	Target reliability index, β	Reference period
Serviceability		
reversible	0,0	Intended remaining working life
irreversible	1,5	Intended remaining working life
Fatigue		
can be inspected	2,3	Intended remaining working life
cannot be inspected	3,1	Intended remaining working life
Ultimate		
very low consequences of failure	2,3	L_S years[a]
low consequences of failure	3,1	L_S years[a]
medium consequences of failure	3,8	L_S years[a]
high consequences of failure	4,3	L_S years[a]

[a] L_S is a minimum standard period for safety (e.g. 50 years).

Target reliability levels proposed by Vrouwenvelder and Scholten for buildings

Consequences class	Minimum reference period for existing building (years)	β-NEW		β-EXISTING	
		wn	wd	wn	wd
0	1	3,3	2,3	1,8	0,8
1	15	3,3	2,3	1,8*	1,1*
2	15	3,8	2,8	2,5*	2,5*
3	15	4,3	3,3	3,3*	3,3*

Class 0: as class 1, but no human safety involved.
wn = wind not dominant; wd = wind dominant.
*in this case is the minimum limit for human safety normative.

Reliability based derivation of partial factors on material side

$$\gamma_m = \frac{X_k}{X^*} = \frac{\mu_X(1-1.645\delta_X)}{\mu_X(1-\alpha_R\beta\delta_X)}$$ Gaussian distribution

$$\gamma_m = \frac{X_k}{X^*} = \frac{\mu_X \exp(-1.645\delta_X)}{\mu_X \exp(-\alpha_R\beta\delta_X)}$$ Lognormal distribution

With

δ_X = coefficient of variation of material property

α_R = 0.32 (FORM sensitivity factor)

Reliability based derivation of partial factors for permanent actions

$$\gamma_{\mathrm{g}} = \frac{G^*}{G_{\mathrm{k}}} = \frac{\mu_G\left(1 - \alpha_E \beta \delta_G\right)}{\mu_G\left(1 + k\delta_G\right)}$$

For unfavourable effect of action

With

- δ_G = coefficient of variation of action
- α_E = -0.28 (FORM sensitivity factor)

$$\gamma_{\mathrm{g}} = \frac{G^*}{G_{\mathrm{k}}} = \frac{\mu_G\left(1 - \alpha_{E,fav} \beta \delta_G\right)}{\mu_G\left(1 + k\delta_G\right)}$$

For favourable effect of action

With $\alpha_{E,fav}$=0.32

Reliability based derivation of partial factors for variable actions

$$\gamma_{\mathrm{q}} = \frac{Q^*}{Q_{\mathrm{k}}} = \frac{F_{Q,tref}^{-1}\left[\Phi\left(-\alpha_E \beta, t_{ref}\right)\right]}{Q_{\mathrm{k}}}$$

With $F^{-1}_{Q,tref}$ = inverse of distribution of maxima over t_{ref} period

For a climatic action

Maxima over basic reference period t_0 modeled by Gumbel distribution

Characteristic value defined as 98^{th} fractile of maxima over t_0

Mutually independent maxima over t_0

$$\gamma_q = \frac{\mu_{Q,tref}\left[1-\delta_{Q,tref}\left(0.45+0.78\ln\left(-\ln\left(\Phi^{-1}(-\alpha_E\beta)\right)\right)\right)\right]}{\mu_{Q,t0}\left[1-\delta_{Q,t0}\left(0.45+0.78\ln(-\ln(0.98))\right)\right]}$$

Partial factors for material resistance evaluated with $\delta_X = 0.05$

Consequence class	β		γ_m	
	wn	wd	wn	wd
0	1.8	0.8	0.99	0.95
1	1.8	1.1	0.99	0.96
2	2.5	2.5	1.02	1.02
3	3.3	3.3	1.05	1.05

Partial factors for material resistance evaluated with $\delta_X = 0.15$

Consequence class	β		γ_m	
	wn	wd	wn	wd
0	1.8	0.8	1.00	0.86
1	1.8	1.1	1.00	0.89
2	2.5	2.5	1.05	1.05
3	3.3	3.8	1.16	1.23

Partial factors for permanent actions evaluated with $\delta_G = 0.05$

Consequence class	β		$\gamma_{g,fav}$		$\gamma_{g,unfav}$	
	wn	wd	wn	wd	wn	wd
0	1.8	0.8	0.97	0.99	1.06	1.03
1	1.8	1.1	0.97	0.98	1.06	1.04
2	2.5	2.5	0.96	0.96	1.09	1.09
3	3.3	3.3	0.95	0.95	1.12	1.12

Partial factors for snow load evaluated with t_0 = 1 year, t_{ref} = 50 years, $\mu_{q,t0} / q_k$=0.4 and $\delta_{q,t0}$ = 0.5

Consequence class	β		γ_q	
	wn	wd	wn	wd
0	1.8	0.8	1.25	1.01
1	1.8	1.1	1.25	1.03
2	2.5	2.5	1.40	1.11
3	3.3	3.3	1.61	1.16

Structural analysis

Structural model should reflect the actual condition of the existing structure

Proper deterioration models to be considered for prediction of actual and future evolution in time of structural behaviour

Knowledge of deterioration mechanism is necessary

Structural performance to be analyzed by means of

Linear elastic analysis

Linear elastic analysis with limited redistribution

Plastic analysis

Non-linear analysis

Selection of analysis type

Structural type

Failure consequence class

Validity of models used for new structures

Availability of new design models

If the minima requirements for the validity of resisting models used for new structures are not fulfilled and new models, able to describe the actual structural behaviour and / or the deterioration and its evolution are not available

Non-linear analysis

Design by testing

Combination of both techniques

Safety format

Partial factors format

Full probabilistic

Global resistance factors

Linear elastic, linear elastic with limited redistribution, plastic

Non-linear analysis

Partial safety factors format

$$g\,(F_d, X_d, a_d, \theta_d, C) \geq 0$$

Limit state function

Design value of actions

Design value of material properties

Design value of geometrical quantities

Design value of model uncertainties

Serviceability constraints

Design values to be determined on the basis of

Target reliability index (β)

Remaining service life

Outcomes of tests

Full probabilistic safety format

↓

Estimation of probability of failure or reliability index β evaluation

↑

Recommendations of JCSS "Probabilistic Model Code" and "Probabilistic assessment of existing structures"

Verification procedure

- Definition of structural model
- Description of aleatory input variables
  - Random variables
  - Random fields (spatial randomness)
- Description of epistemic uncertainties
  - Discrete distribution function
  - Event tree
- Probabilistic analysis (with bayesian approach) of structural performance
  - Monte Carlo method

Updating of random variables considering the actual structural condition

Actions

Material properties

Dimensions of structural elements

Deterioration models

Model uncertainties

Global resistance factor

Accounts for the uncertainties of structural behaviour at the level of structural resistance

Effects of various uncertainties integrated in a global design resistance and expressed by a global safety factor

Representative values of global resistance variables and global safety factors chosen to fulfil the reliability requirements in terms of β index

Global safety format

Reflects the variability of structural response due to random properties of basic variables and model uncertainties

Limit state function is described by N.L. analysis

Variability of R not constant for a set of materials, but depending on structural model

For failure governed by concrete, resistance variability higher than for reinforcement dominated failure

Global safety format

May be defined in the domain of

Actions

Action effects (internal actions)

P.G.A.

In the domain of actions

$$\gamma_G G_k + \gamma_Q Q_k + \gamma_P P_k \le \frac{q_u}{\gamma_R'}$$

$$\gamma_G G_k + \gamma_Q Q_k + \gamma_P P_k \le \frac{q_u}{\gamma_R \gamma_{Rd}}$$

With: q_u → failure load estimated with an incremental non linear analysis with the mean values of material resistances;

γ'_R → global safety factor accounting also for uncertainties in structural resistance and in resisting model;

γ_R → global safety factor accounting for the only uncertainties in structural resistance;

γ_{Rd} → partial factor accounting for the uncertainties in resisting model.

γ_R and γ'_R are derived by means of probabilistic approach

$$\mathrm{Prob}\left(R \le R_d\right) = \Phi\left(-\alpha_R \beta\right)$$

CDF of standard normal distribution of resistance

FORM sensitivity factor

Reliability index

Structural resistance is described by a two-parameter lognormal distribution

Assuming $V_R \leq 0.25$

$$R_d = \mu_R \exp(-\alpha_R \beta V_R)$$

Mean value of resistance

Coefficient of variation of resistance

→ V_R to be estimated by Monte Carlo method

Global resistance factor

$$\gamma_R = \frac{\mu_R}{R_d} = \frac{\mu_R}{\mu_R \exp(-\alpha_R \beta V_R)} \approx \exp(\alpha_R \beta V_R)$$

$$\gamma_R' = \exp(\alpha_R \beta V_R')$$

Accounting for uncertainties in resistance and resisting model

As a simplification: $\gamma_R' = \gamma_R \gamma_{Rd}$

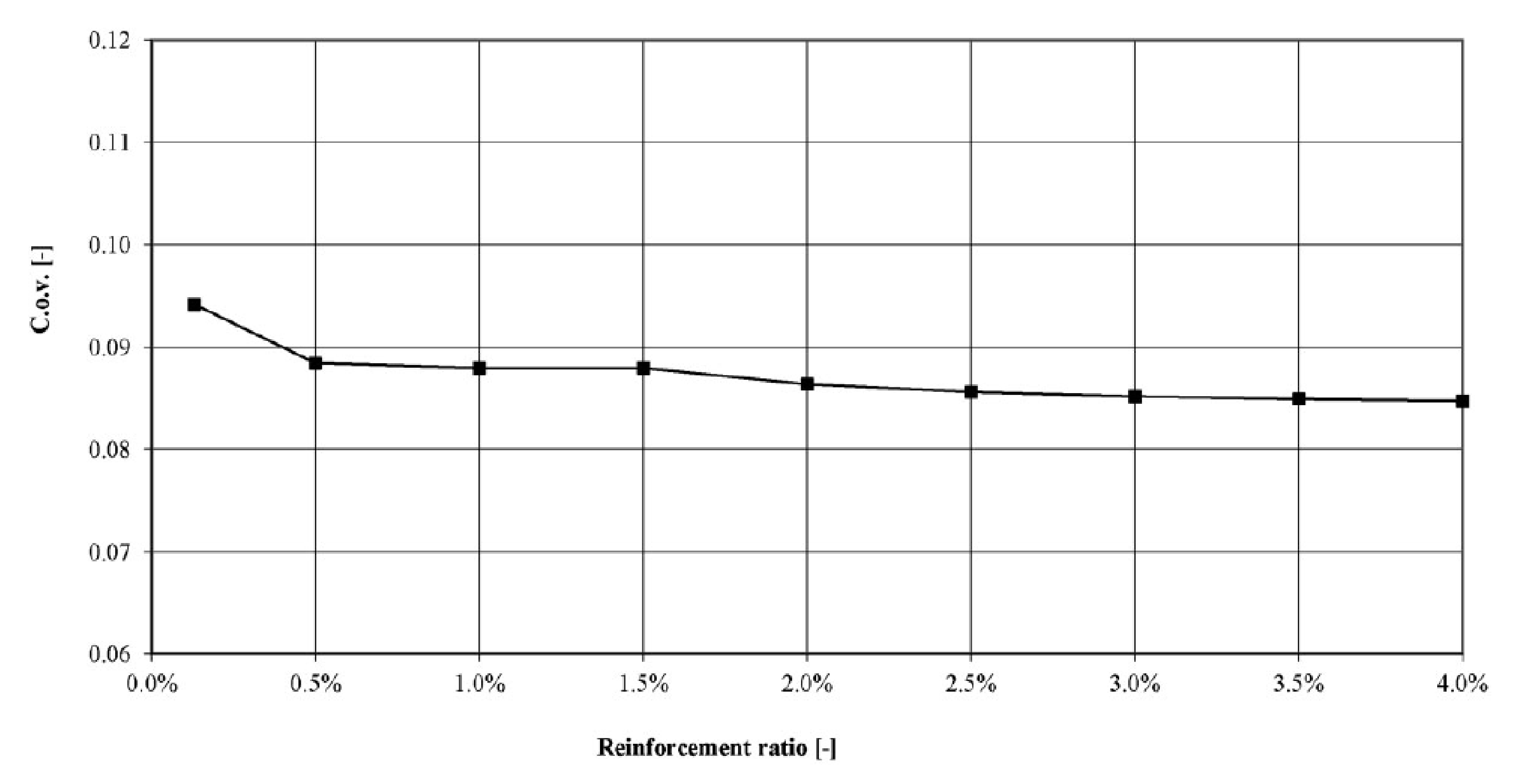
Two span continuos beam in bending

↓

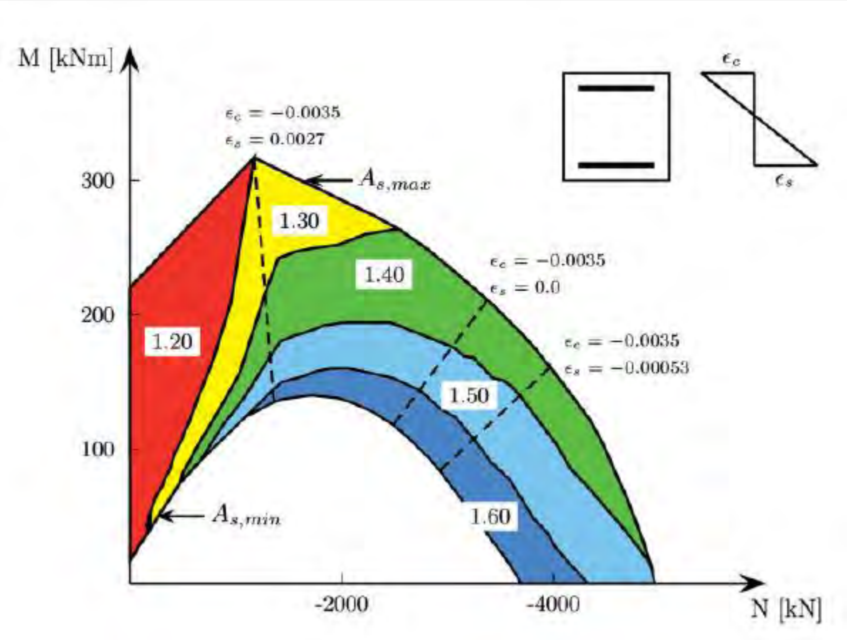
Probabilistic model

Variable	Description	Distribution	Mean value	Std. dev.	C.o.v.
f_{c35} [MPa]	Concrete compressive strength	log-normal	40.6	5.4	0.13
f_y [MPa]	Yield stress	log-normal	560.0	30.0	0.05
f_t [MPa]	Tensile strength	log-normal	644.0	40.0	0.06
ϑ_R [–]	Resistance model uncertainty	log-normal	1.1	0.077	0.07

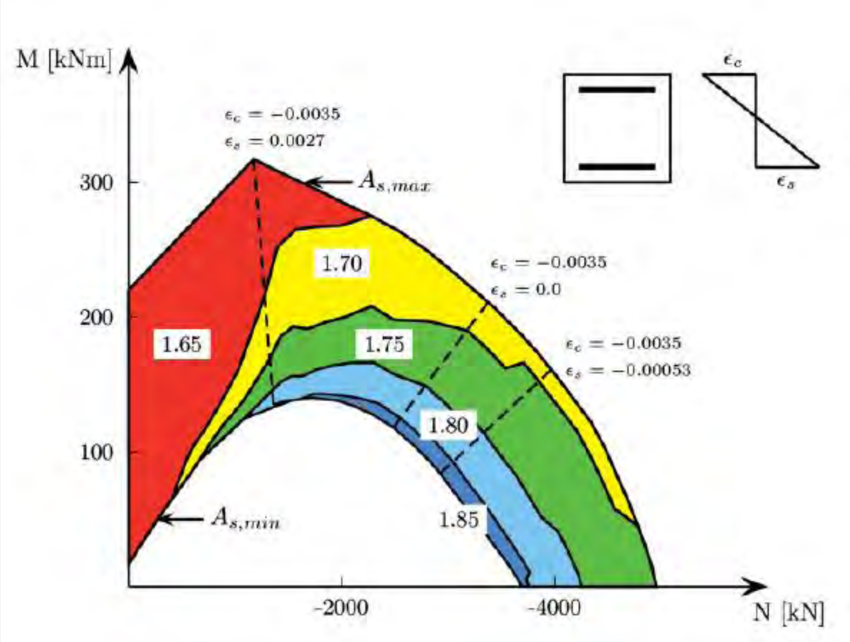
Coefficient of variation of resisting bending moment versus reinforcement ratio



Global resistance factors γ_R and γ'_R in short columns for different M / N combinations

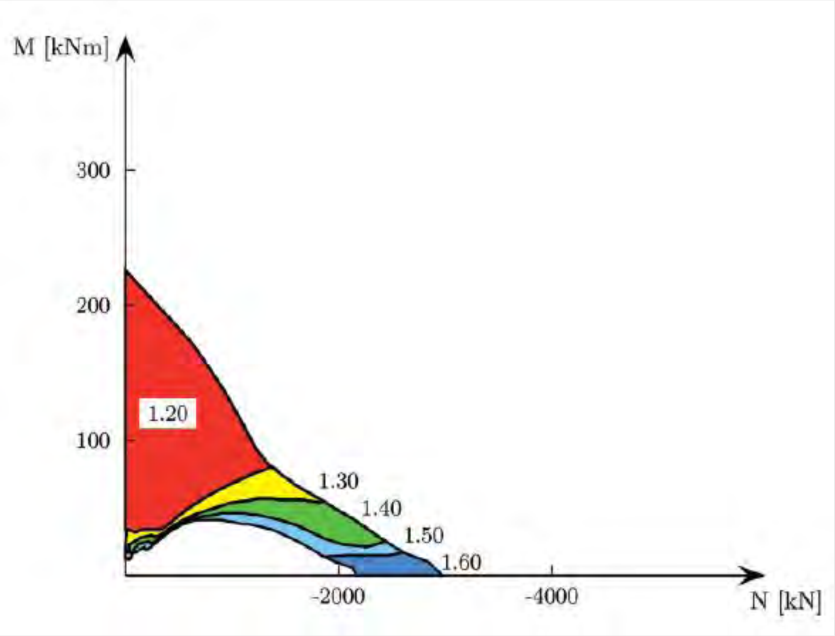


Global resistance factor γ_R

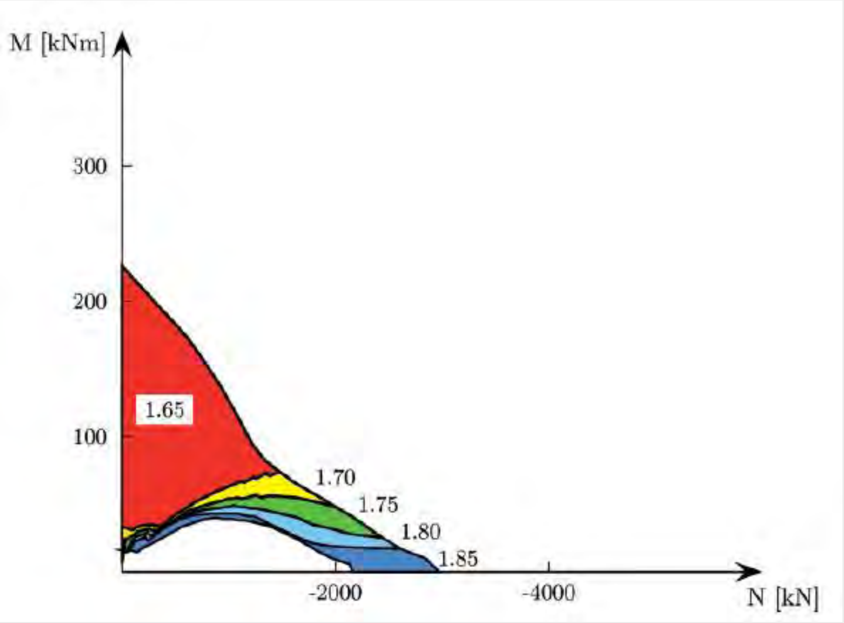


Global resistance factor γ'_R

Global resistance factors γ_R and γ'_R in slender columns (λ =100) for different M / N combinations



Global resistance factor γ_R



Global resistance factor γ'_R

Most important deterioration phenomena in concrete structures

- Reinforcement corrosion (carbonation and chlorides)
- Freeze and thaw
- ASR
- Chemical agent attack
- Fatigue

Corrosion effect on reinforcement

Distributed corrosion

Carbonation and
chloride high content

Pitting corrosion

Chlorides low content

Corrosion effect on reinforcement

- Reduction of cross section
- Modification of constitutive relationship (Cairns et al.)

$$f_y = \left(1.0 - a_y \times Q_{corr}\right) \times f_{y0}$$

$$f_u = \left(1.0 - a_u \times Q_{corr}\right) \times f_{u0}$$

$$e_u = \left(1.0 - a_1 \times Q_{corr}\right) \times e_{10}$$

Q_{corr} = corroded percentage area

Pay attention to redistribution
and plastic analysis !!

Empirical coefficients for strength and ductility reduction of reinforcement

Authors		Exposure	Q_{corr}%	α_y	α_u	α_1
Palsson e Mirza [5]	Concrete	Sevice, chloride	0 to 80	0.0	0.0	NS
Castel et al. [6]	Concrete	Chloride, 0.0 mA/cm^2	0 to 20	0.0	NS	0.035
Du [4]	Bare	Accelerated, 0.5 a 2.0 mA/cm^2	0 to 25	0.014	0.014	0.029
	Concrete	Accelerated, 1.0 mA/cm^2	0 to 18	0.015	0.015	0.039
Maslehuddin et al. [7]	Bare	Service, marine	0 to 1	0	0	0
Allam et al. [8]	Bare	Service, marine	0 to 1	0	0	0
Morinaga [9]	Concrete	Sevice, chloride	0 to 25	0.017	0.018	0.06
Zhang et al. [10]	Concrete	Sevice, carbonation	0 to 67	0.01	0.01	0
Andrade et al. [11]	Bare	Accelerated, 1.0 mA/cm^2	0 to 11	0.015	0.013	0.017
Clark e Saifullah [12]	Concrete	Accelerated, 0.5 mA/cm^2	0 to 28	0.013 0.012	0.017 0.014	NS
Lee et al. [13]	Concrete	Accelerated, 13.0 mA/cm^2	0 to 25	0.012	NS	NS
Cairns et al. [3]	Concrete	Accelerated, 0.01 a 0.05 mA/cm^2	0 to 3	0.012	0.011	0.03

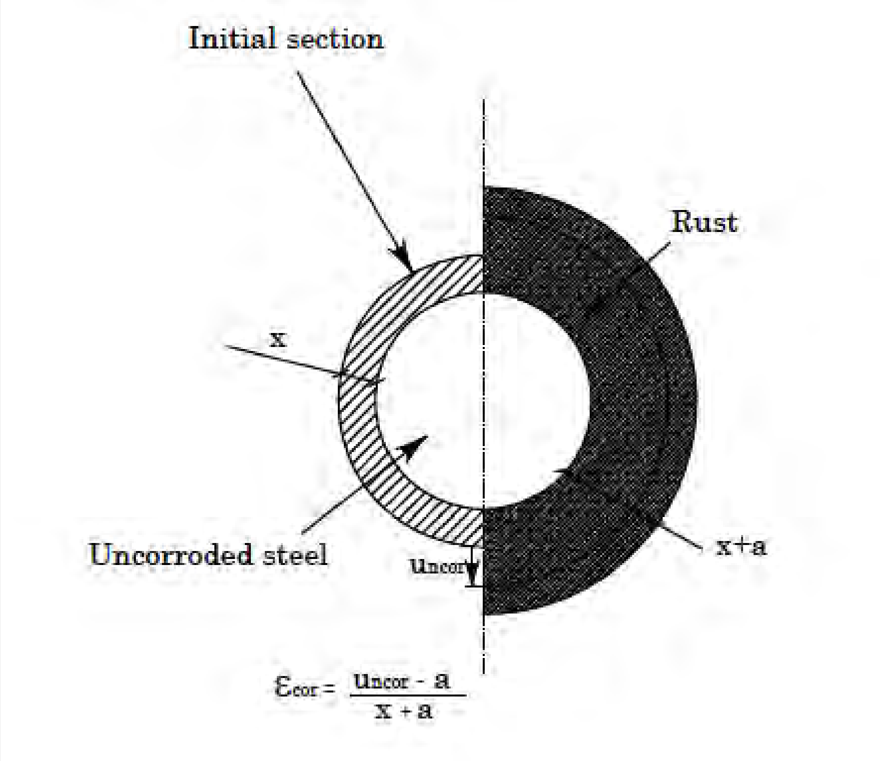
Corrosion effect on concrete

↓

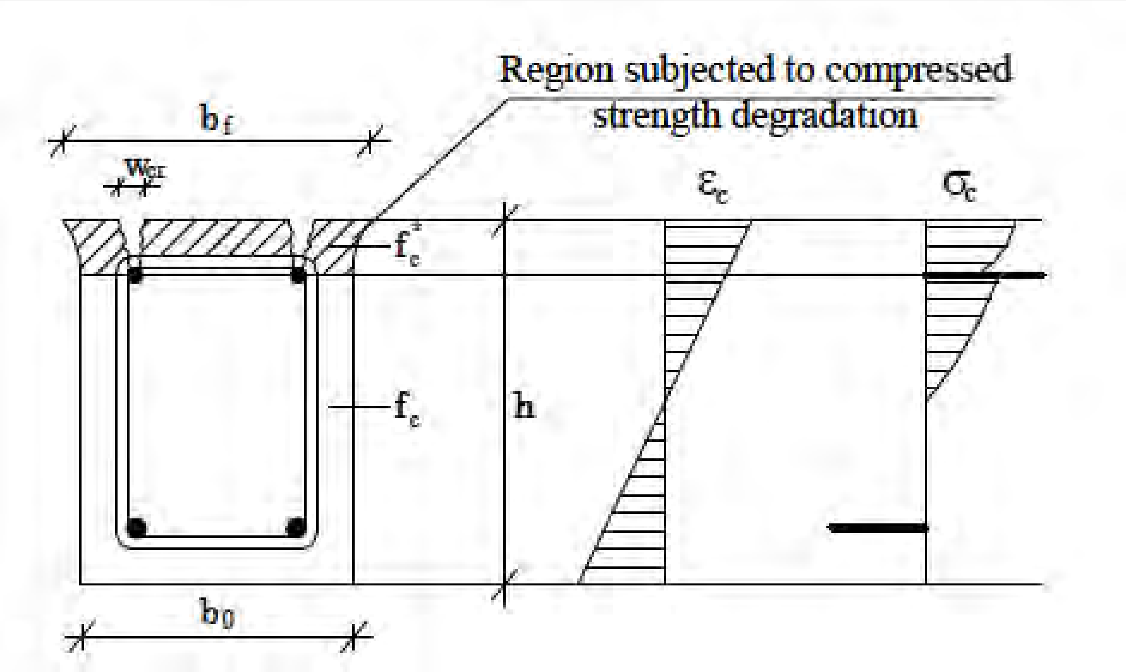
Corrosion product characterized by a volumetric expansion ratio

Corrosion product	Volume increase
Fe_3O_4	2.2
$Fe(OH)_2$	3.8
$Fe(OH)_3$	4.2
$Fe(OH)_3$, $3H_2O$	6.4

A radial pressure is generated with tensile strain in concrete around the bars



Exceeding the maximum tensile strain, concrete cracks, so reducing the compressive resistance along the crack, in particular outside the confining effect of reinforcement



A reduced concrete strength may be defined (Coronelli et al.)

$$f_c^* = \frac{f_c}{1 + k \times \dfrac{e_1}{e_{c0}}}$$

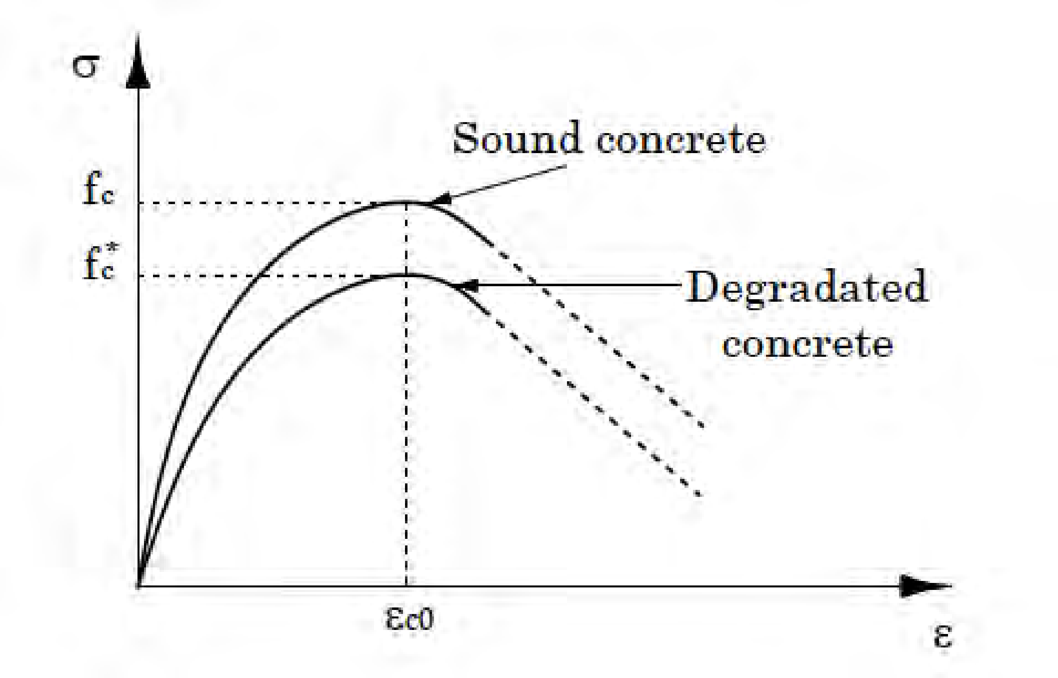
where $k = 0.1$, e_{c0} = deformation at peak load and

$$e_1 = \frac{b_f - b_0}{b_0} = \frac{n_{bars} \times w_{crack}}{b_0}$$

n_{bars} = number of corroded bars

w_{crack} =crack openings

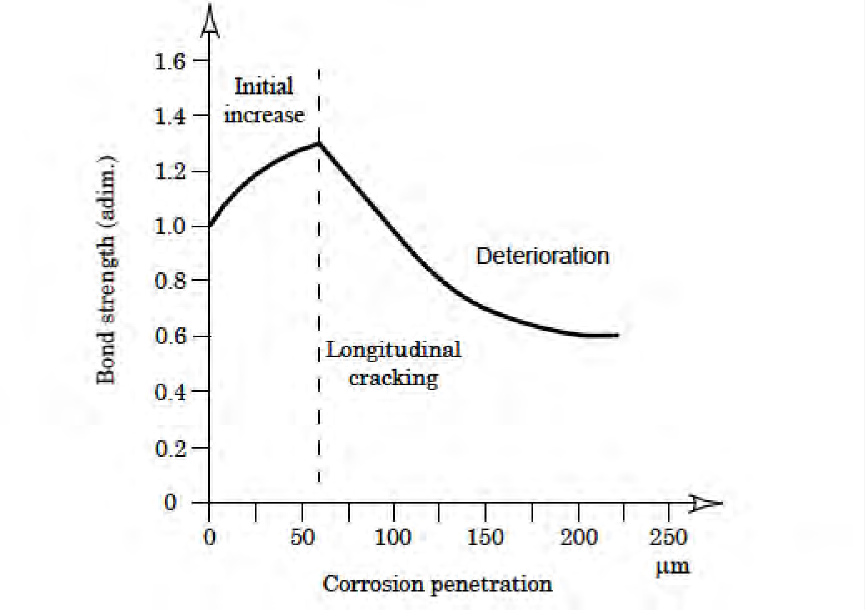
Homotetic reduction of concrete strength due to reinforcement corrosion



Corrosion effect on bond

Bond increase due to confinement exerted by oxide expansive formation up to the appearance of first crack in concrete

Bond reduction after the appearance of longitudinal cracks along the bars (important role of confinement exerted by reinforcement, Berra)



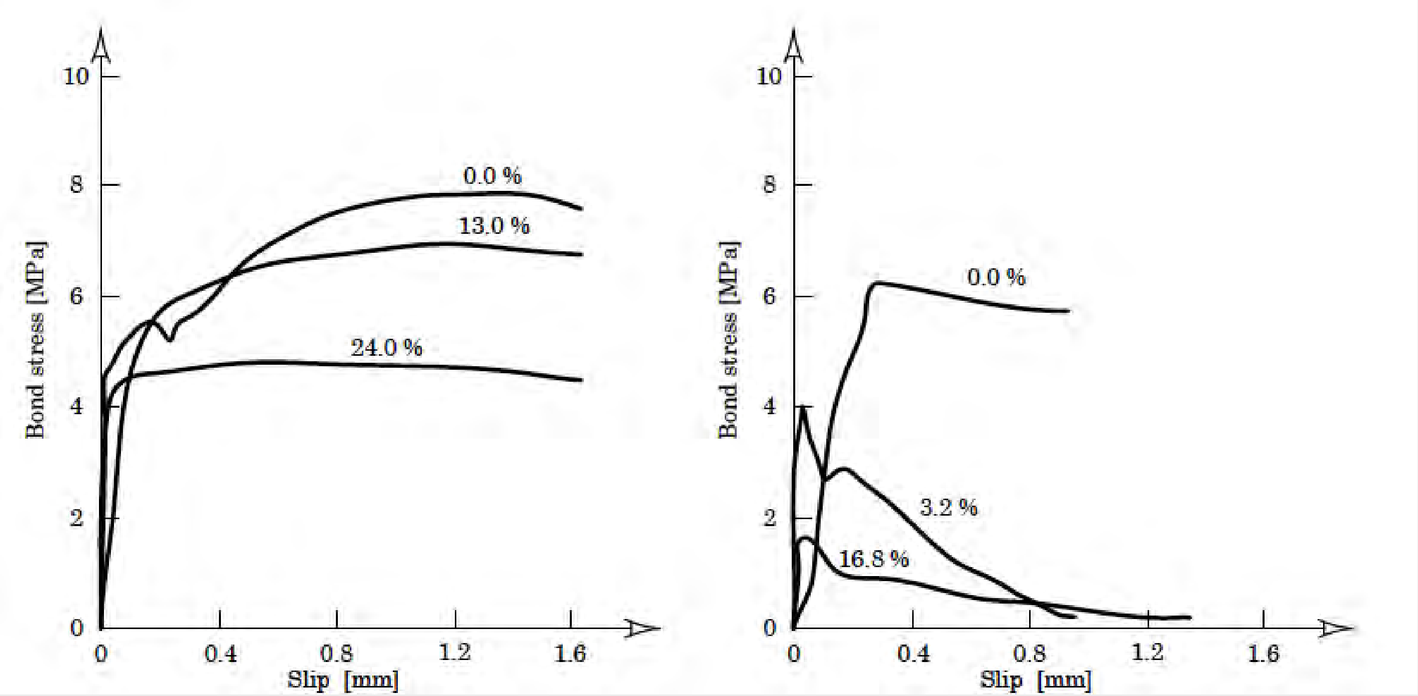
Main parameter influencing bond reduction

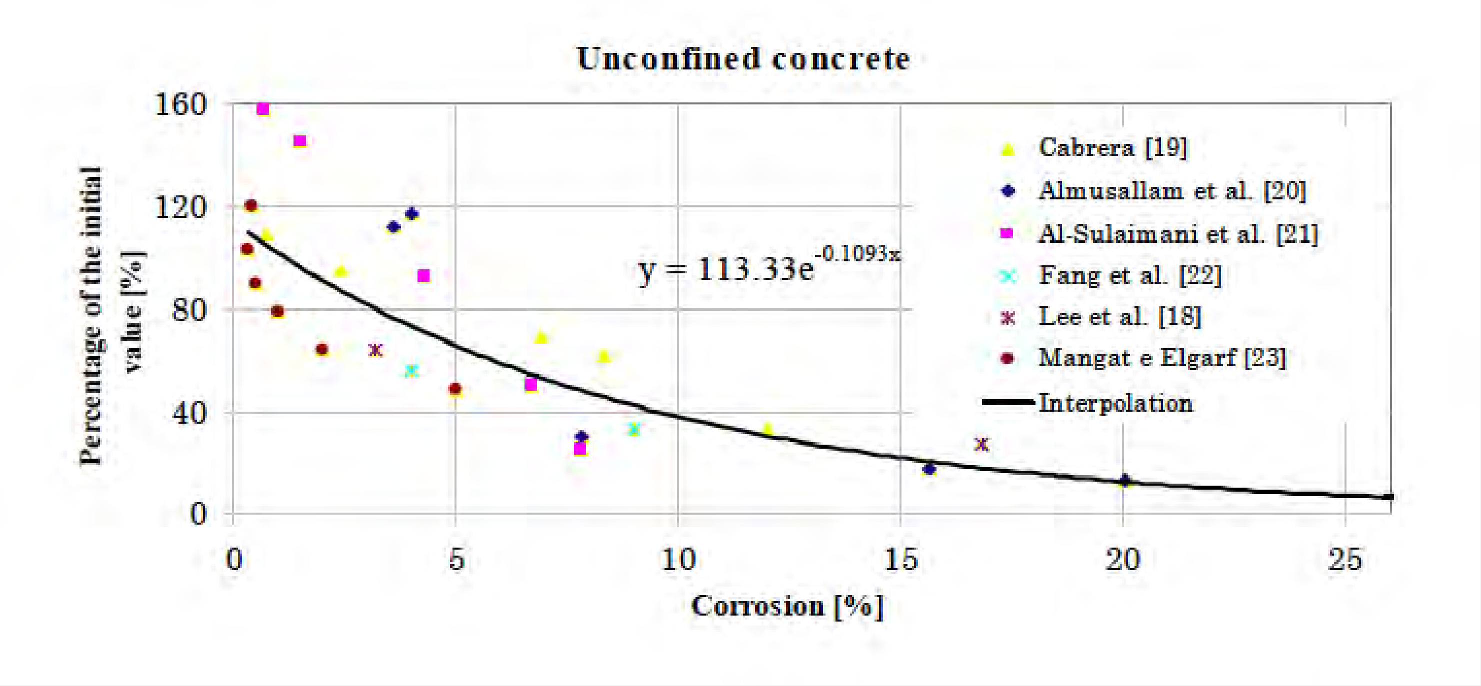
- Concrete tensile strength
- Bar type (bare or ribbed)
- Type and rate of corrosion (natural/accelerated)
- Ratio between cover and bar diameter (c/Φ)
- Level of confinement

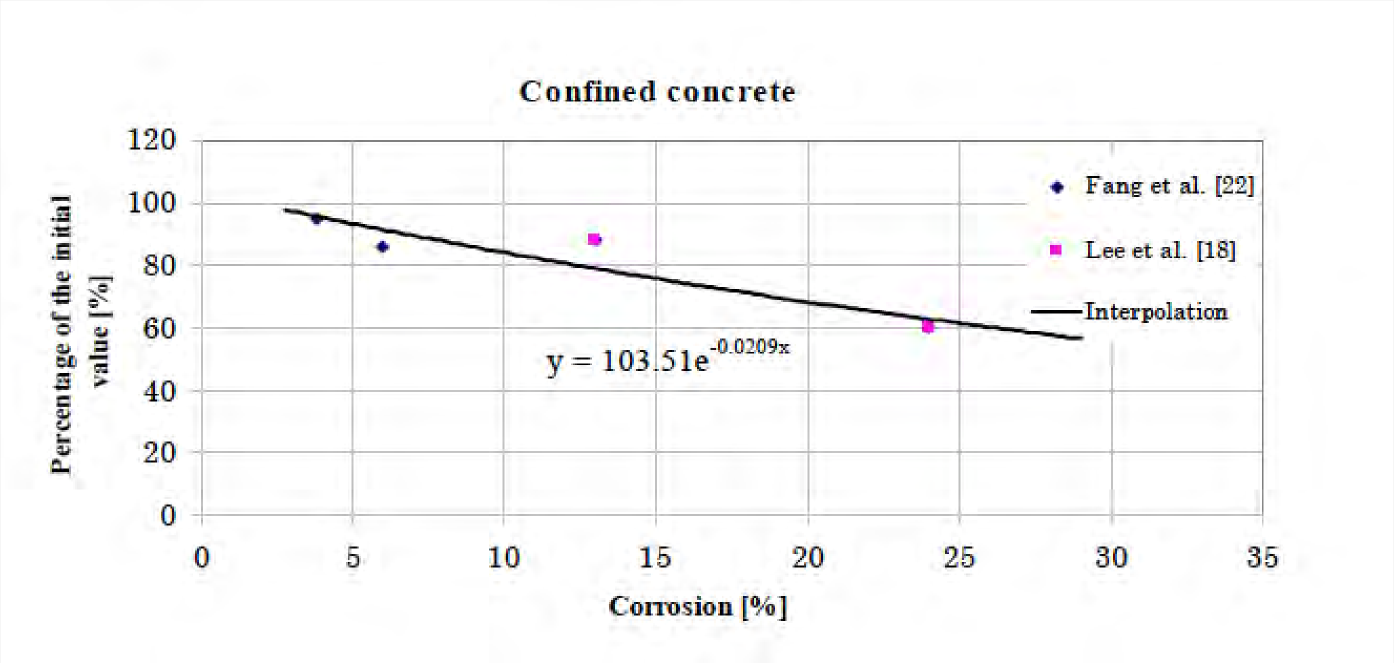
High influence

Pull out tests with and without confinement done by Lee et al.

(cube 14mm , Φ=13mm, bond length = 8Φ)




Unconfined concrete
Percentage of the initial value [%]
Corrosion [%]
$y = 113.33e^{-0.1093x}$
160
120
80
40
0
0
5
10
15
20
25
Cabrera [19]
Almusallam et al. [20]
Al-Sulaimani et al. [21]
Fang et al. [22]
Lee et al. [18]
Mangat e Elgarf [23]
Interpolation



Confined concrete
Percentage of the initial value [%]
120
100
80
60
40
20
0
0
5
10
15
20
25
30
35
Corrosion [%]
Fang et al. [22]
Lee et al. [18]
Interpolation
y = 103.51e-0.0209x

Test results on bond deterioration are not exhaustive. A large experimental campaign is on the way in Turin

	Φ^1 [mm]	Specimen side=10Φ [mm]	$\rho_w{}^2$ [%]	Level of corrosion [%]					n° specimens
				0	2	5	10	20	
Casting 1	12	120	0	2	2	2	2	2	10
			7.28	-	-	-	-	-	
	16	160	0	-	-	-	-	-	10
			2.73	2	2	2	2	2	
Casting 2	12	120	0	-	-	-	-	-	10
			7.28	2	2	2	2	2	
	16	160	0	2	2	2	2	2	10
			2.73	-	-	-	-	-	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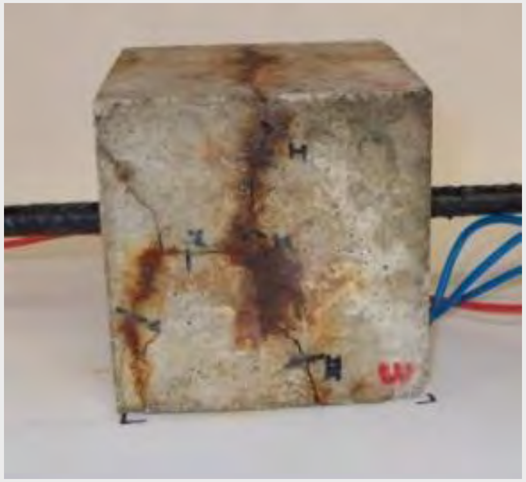

Bond reduction effect in SLS

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In linear elements, mainly a reduction of β coefficient accounting for tension stiffening in moment-curvature diagram

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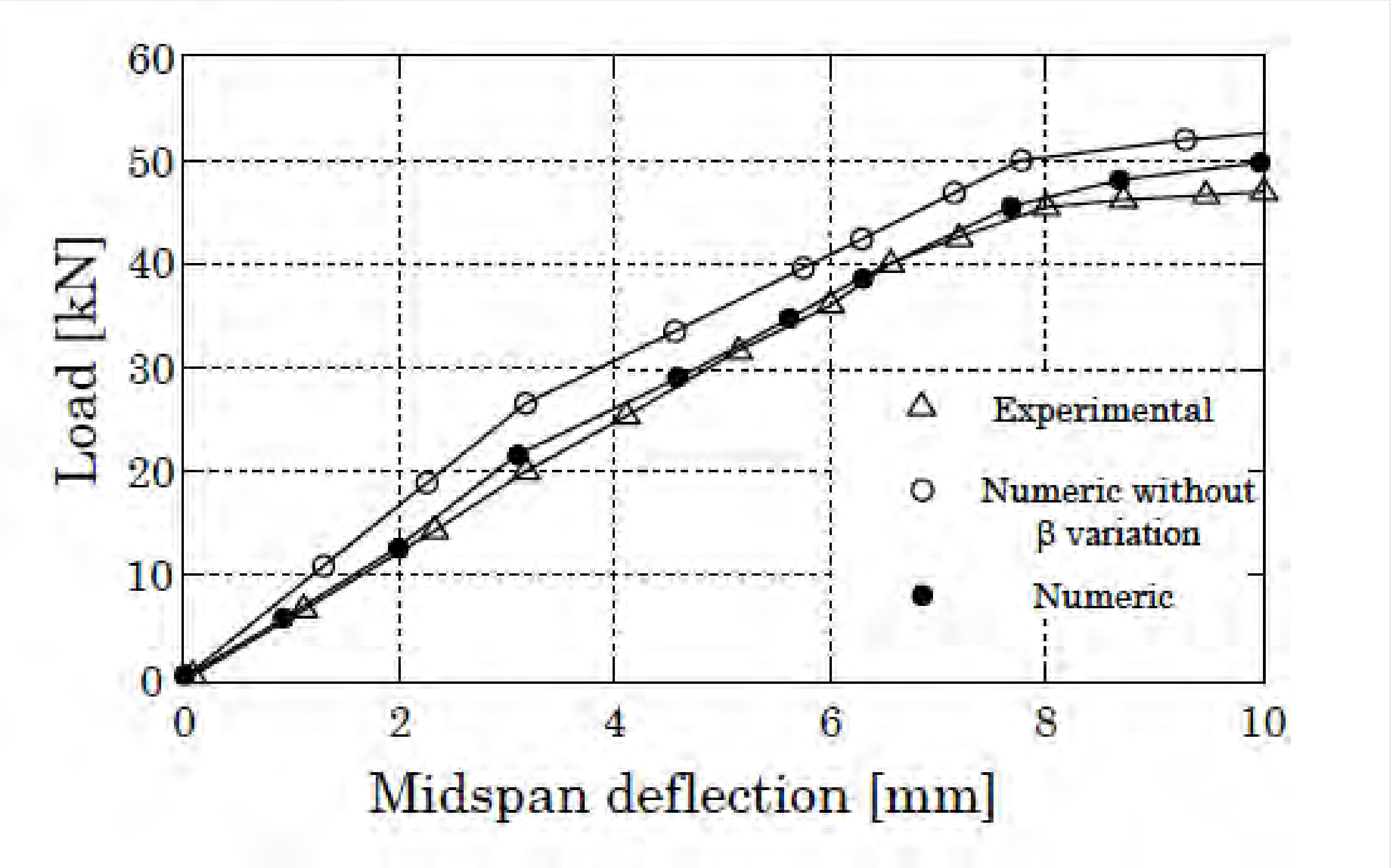
Experimental tests by Rodriguez et al.



Load [kN]
Midspan deflection [mm]
Experimental
Numeric without β variation
Numeric

Bond reduction effect at ULS

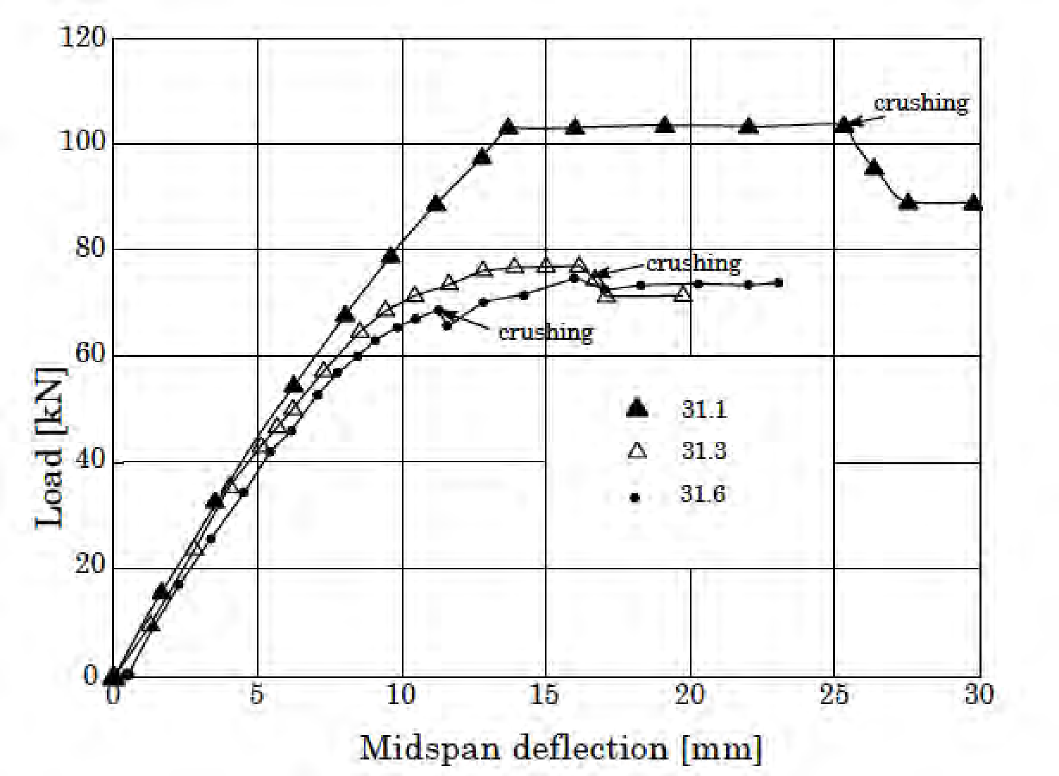
↓

Important reduction of bearing capacity and ductility

↓

- Slip between reinforcement and concrete
- Rising of neutral axis
- Premature failure of concrete
- Ductility reduction
- Need for a N.L. analysis because planarity of section is lost

Beams tested by Rodriguez et al. with different levels of corrosion



Important reduction of bearing capacity and ductility

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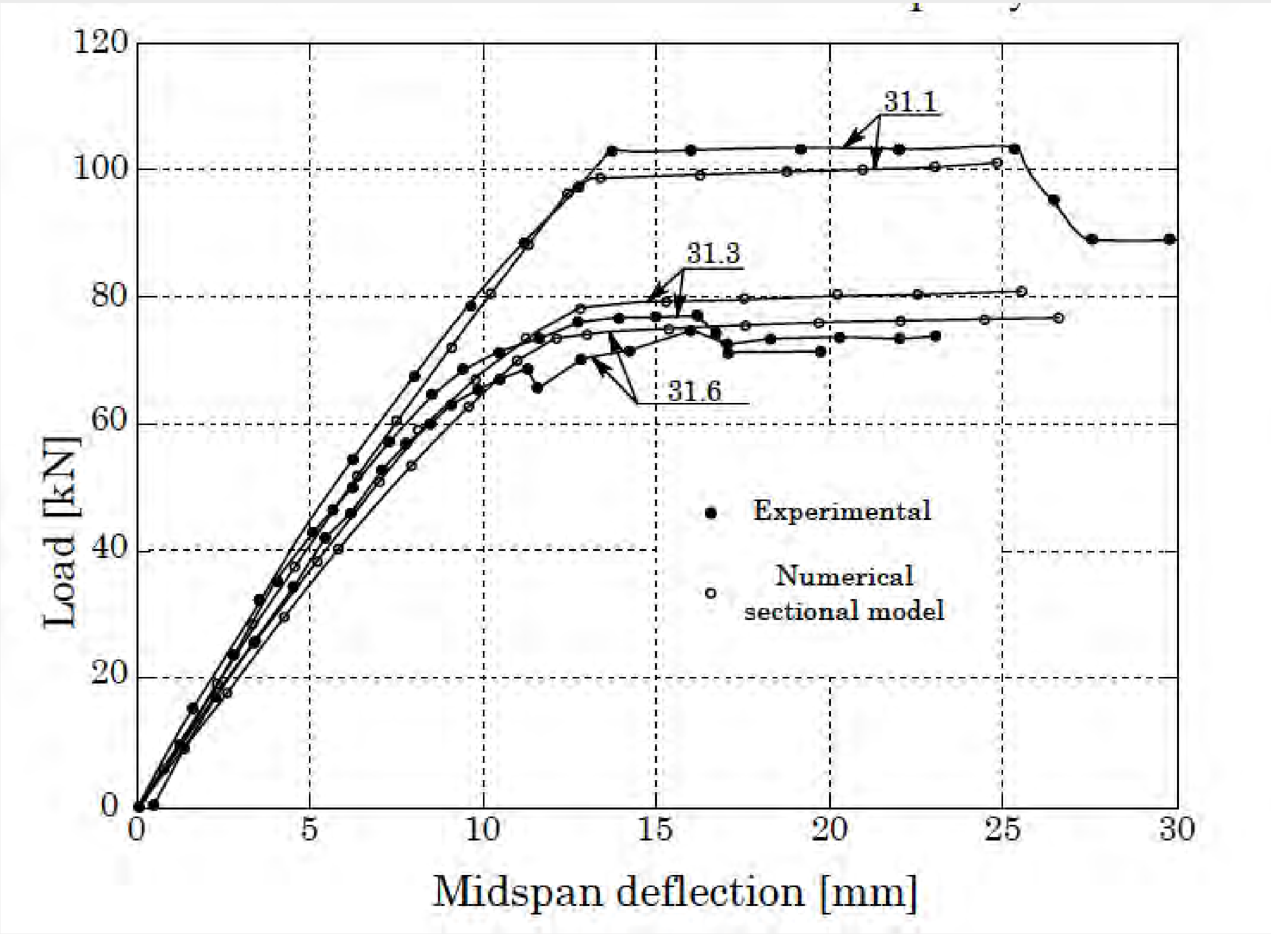
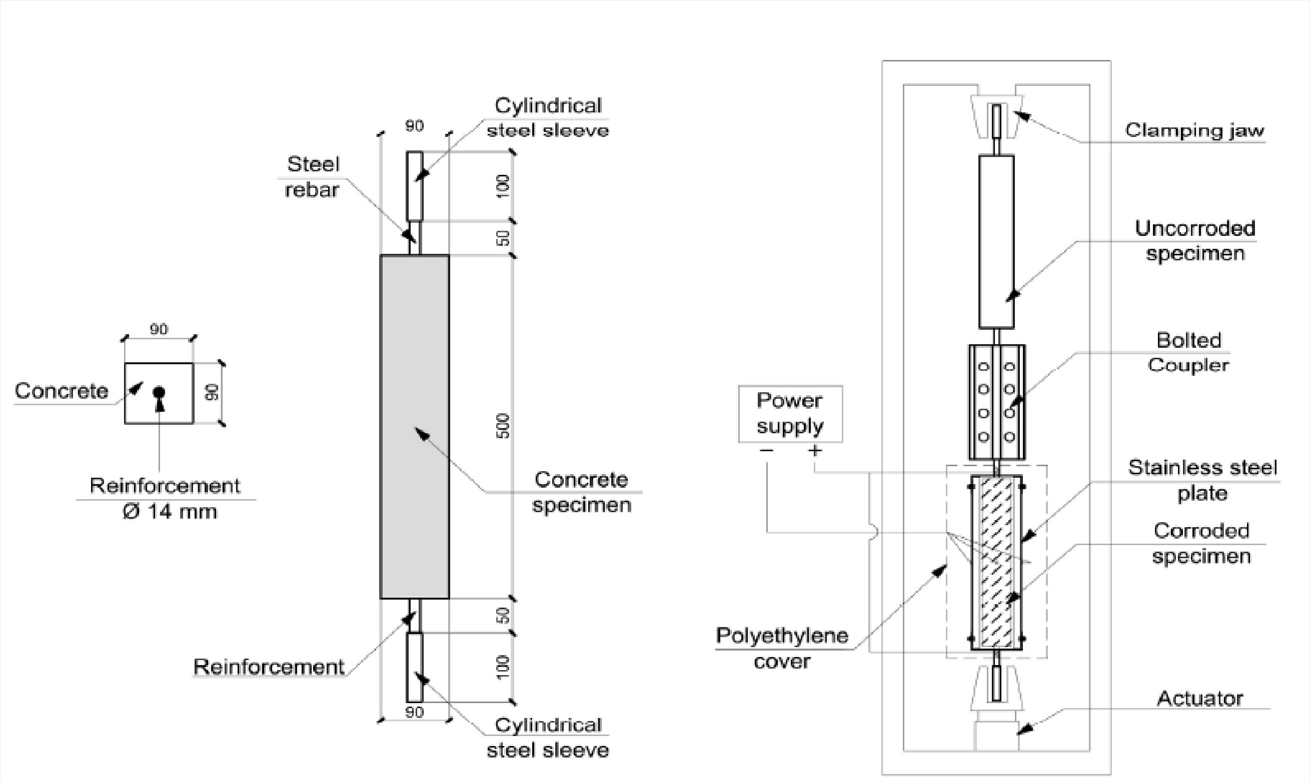
- Equilibrium of a block located between two cracks (S_{rm})
- Compatibility conditions applied along the entire block
- Evaluation of point of nil slip within each block
- Extension of compatibility to adjacent blocks and to all the beam

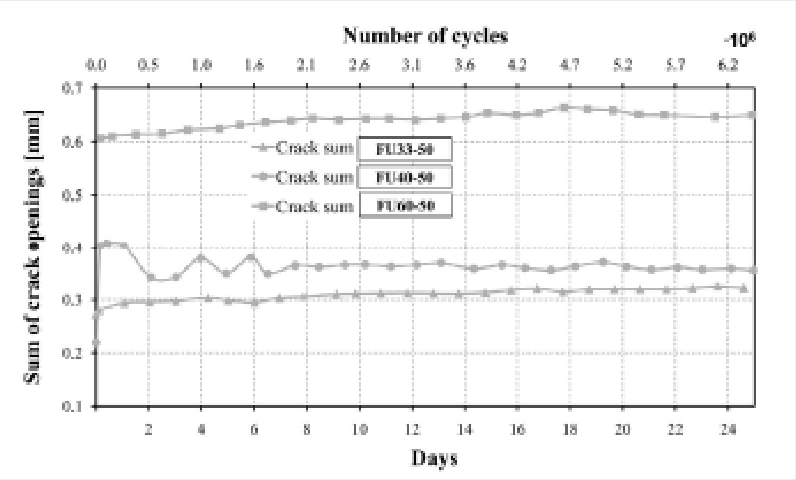
Numerical evaluation of tested beams

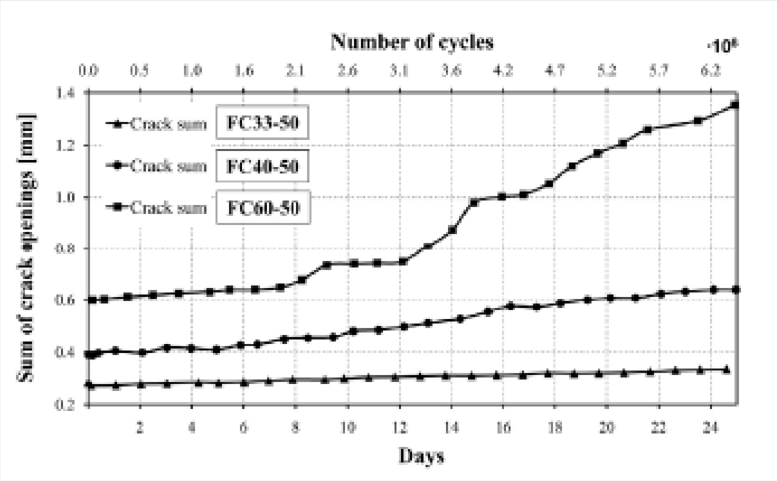


Combined fatigue and corrosion effect on bond in unconfined concrete (bridge slab)

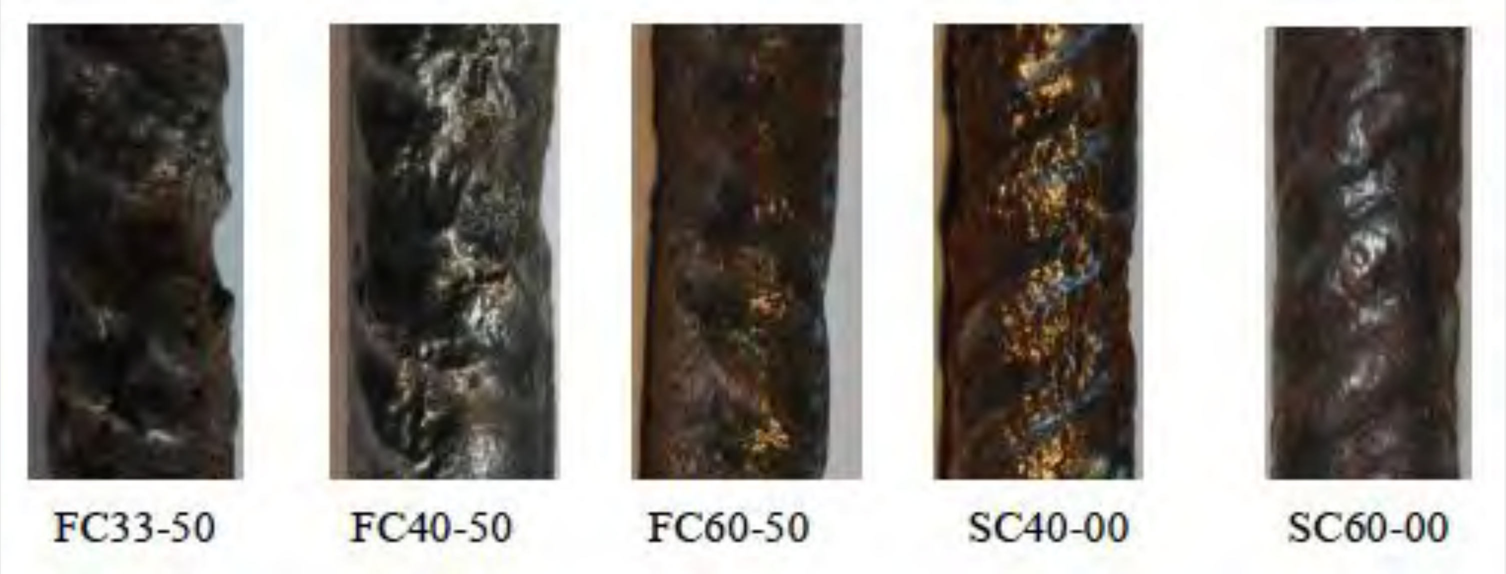


Crack evolution with cyclic number

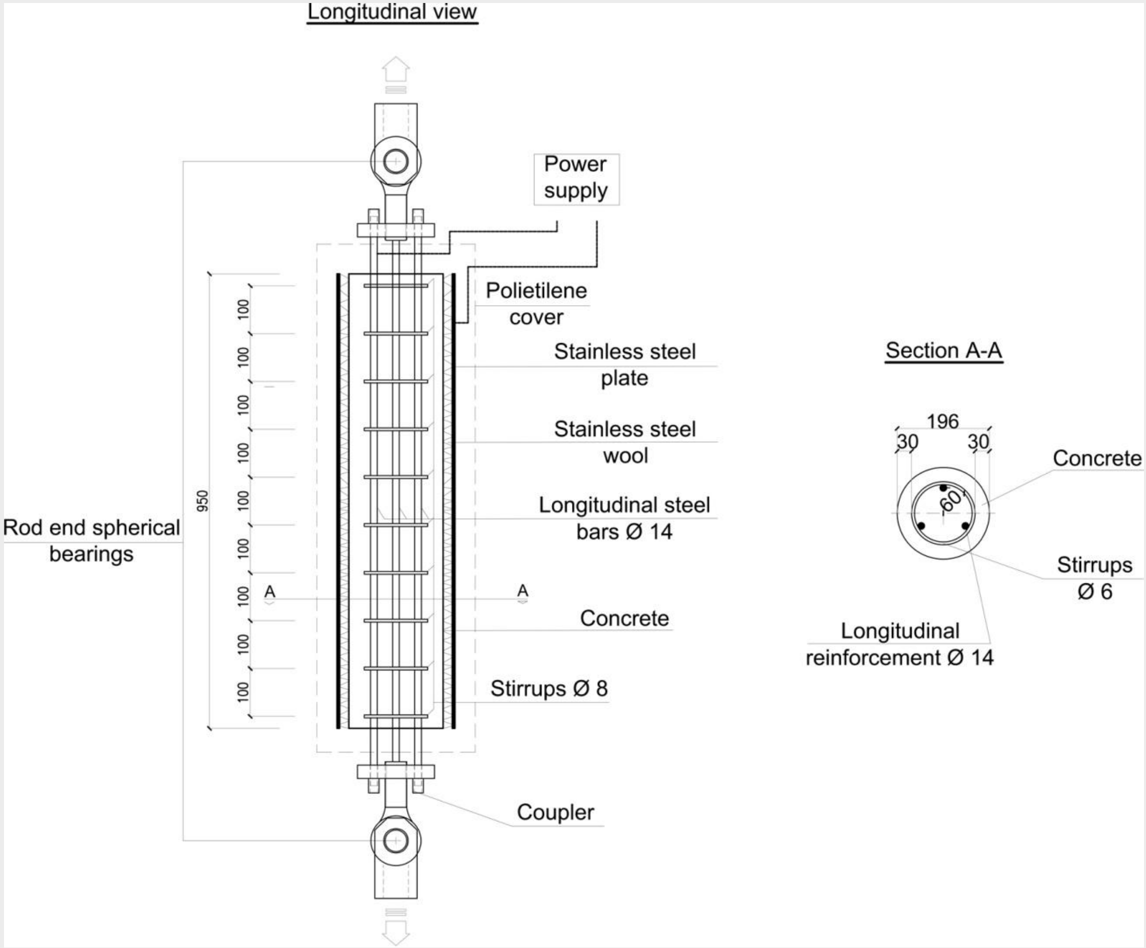




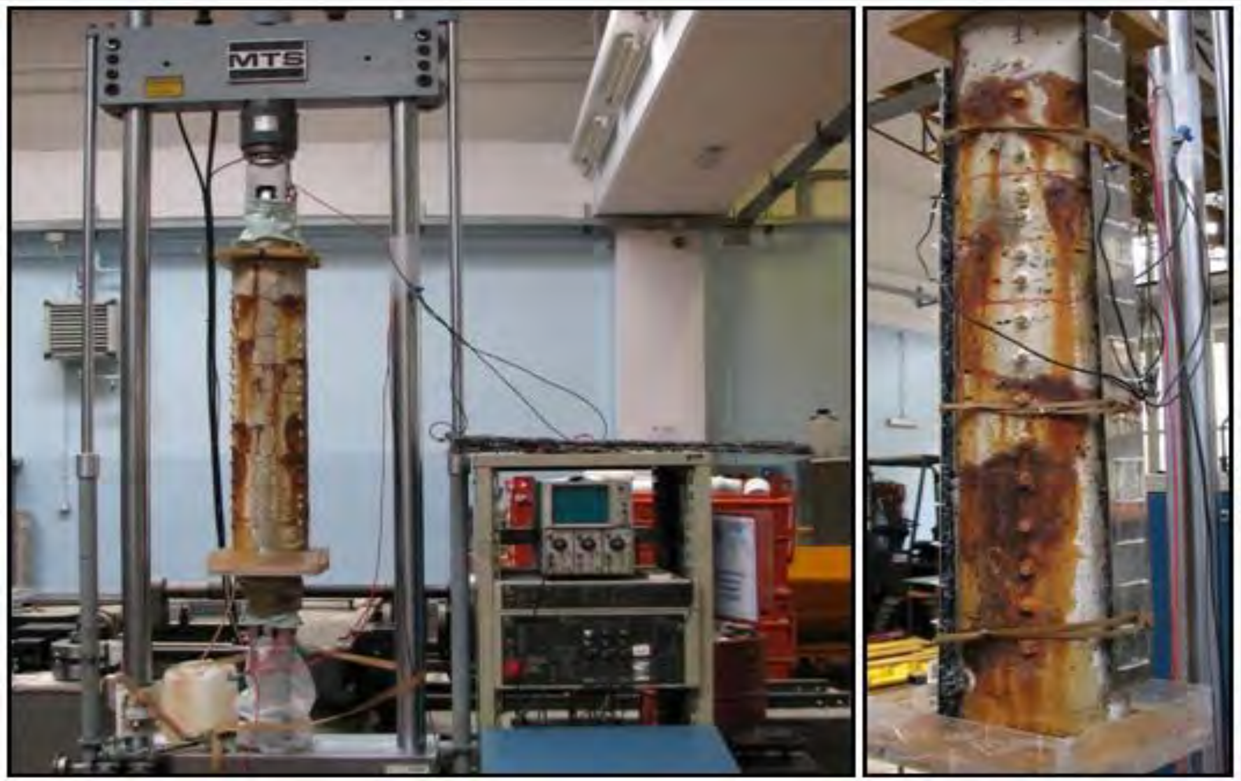
Cyclic action combined with corrosion implies localization of corrosion around cracks due to fretting fatigue (relative slip between concrete and reinforcement near the cracks)



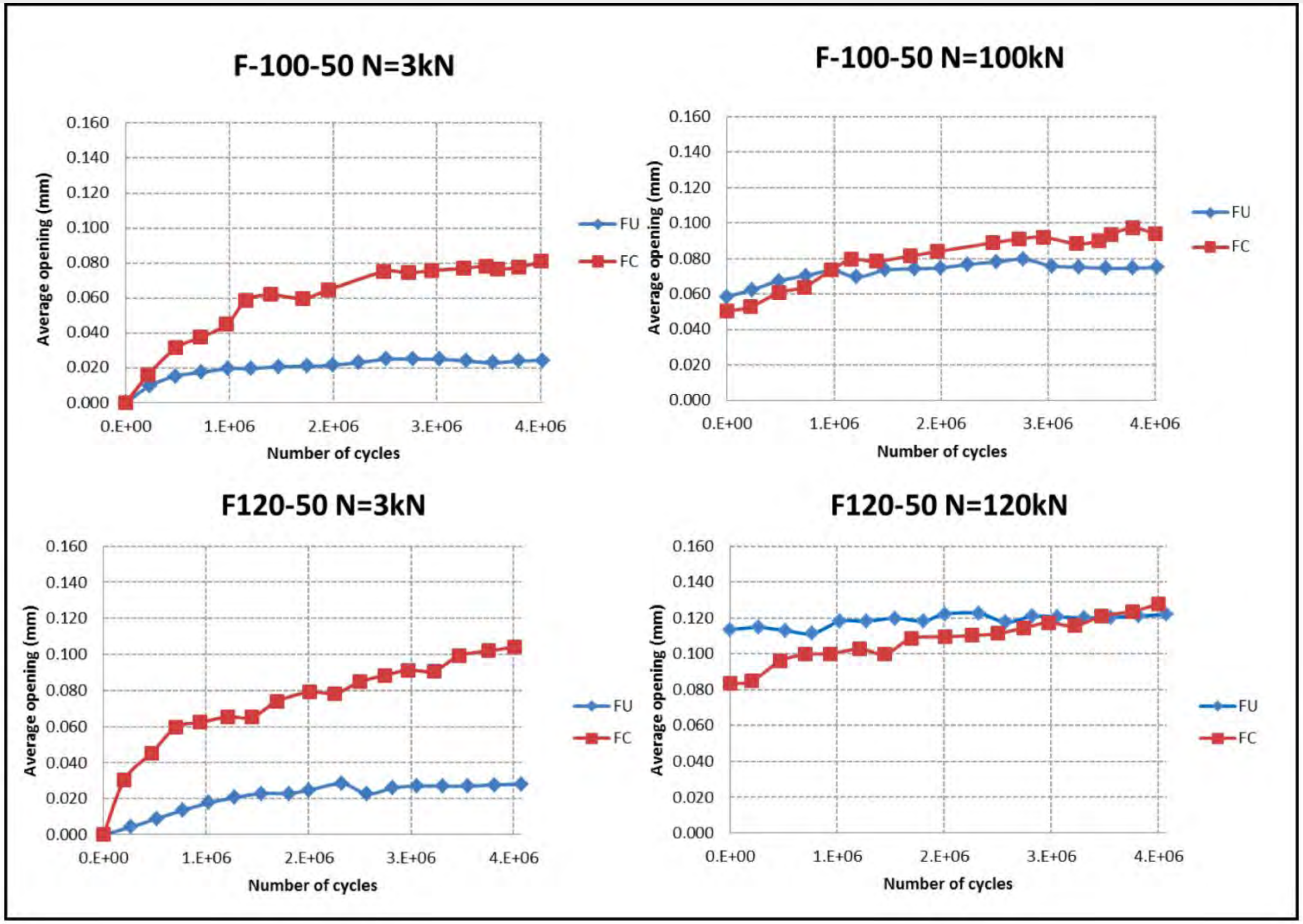
Combined fatigue and corrosion effect on bond in confined concrete



Test on corroded specimens



Crack evolution with cracks number at maximum and minimum load



Expected evolution in time of deterioration level asks for monitoring

Control of deterioration model uncertainties

Evaluation of anomalous evolution of deterioration

Feedback of priority criteria adopted on a set of deteriorated structures

Measured parameters

Overall behavior indicators

Vibration,

overall deformation,

……………..

Local behavior indicators

Local deformation,

local stresses,

………………..

A new approach to measure compressive stresses in concrete in static and dynamic conditions with wireless sensors

↓

Smart Concrete Program

MONITORING

SMART CONCRETE PROGRAM

POLITECNICO DI TORINO

Università degli Studi di Catania

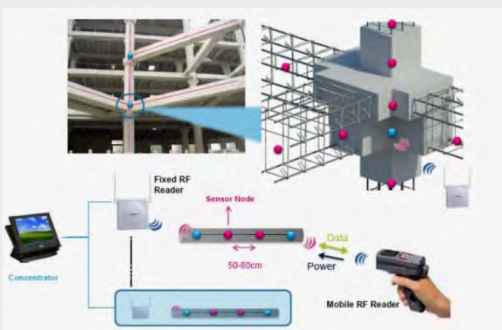
INGV Istituto Nazionale di Geofisica e Vulcanologia

life.augmented

SPEA YOUR BEST WAY TO TEST

PMI

EMBEDDED SENSORS



LOCAL PRESSURE MEASURE

ADVANTAGES (1)

LOW COST

WIRELESS SYSTEM

MEASURE STABILITY

LONG LIFE

ADVANTAGES (2)

- STATIC AND DYNAMIC MEASURE CAPABILITY
- COMPLETE EVALUATION OF STRESS TENSOR
- UNAFFECTED BY CREEP AND SHRINKAGE
- ABLE TO EVAUATE THE REDISTRIBUTION OF INTERNAL ACTIONS (IF ANY) DUE TO CREEP AND SHRINKAGE

APPLICATION TO EXISTING STRUCTURES

⇩

TO EVALUATE THE ACTUAL DAMAGE EVOLUTION RESPECT TO THE EXPECTED ONE

APPLICABILITY

⇩

AT THE MOMENT UNDER CALIBRATION ON CONCRETE SAMPLES IN POLITECNICO DI TORINO / DISEG LABORATORY, THEN APPLICATION IN NEW STRUCTURES AND IN EXISTING INFRASTRUCTURES (BRIDGES)

Thank you for your kind attention