EC8 Webinars
Second Generation of Eurocode 8

Basis of design and data for assessment

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CEN/TC250/SC8 N1236
Enquiry version

CEN/TC250/SC8 N1283
Formal vote version

OUTLINE

Basis of design (Clause 4 and related parts of Part 1-1 + EN1990-2)

CEN/TC250 N2555
Enquiry version

- Reference limit state for assessment of existing structures
- Reference method of analysis for assessment
- Safety format for displacement-based assessment
  - ~~Confidence factors~~ New partial factors $\gamma_{\mathrm{Rd,KL}}$
  - Relation to EN1990-2 Basis of assessment of existing structures

Data for assessment (Clause 5 and Annex A)

CEN/TC250/SC8 N1236
Enquiry version

- General information and history
- Required input data
  - Geometry, Construction details, Materials
- Knowledge levels
  - KLG, KLD, KLM
- Preliminary analysis

Basis of design

Reference limit state & method of analysis
Partial factors method

BASIS OF DESIGN – Limit states & return periods

EN1998-3:2005 vs prEN1998-3:2023

- Limit states
  - Part3'05 gave its own definitions
  - Assessment ≠ Design, global ductility cannot be assumed, SD cannot be checked in lieu of NC as a measure of safety
  - Nonetheless, the choice of how many and which LSs to check is left to Member States

- Return periods (NDP)
  - Part3'05 gave its own definitions
  - NC: **2475** years (2% in 50 years) and
  - DL: **225** years (Part 1 had 95 years!)

EN1998-3:2005

2.1 Fundamental requirements

(1)P The fundamental requirements refer to the state of damage in the structure, herein defined through three Limit States (LS), namely Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL). These Limit States shall be characterised as follows:

LS of Near Collapse (NC). The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity.

LS of Significant Damage (SD). The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infills have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair.

LS of Damage Limitation (DL). The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infills, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures.

NOTE The definition of the Limit State of Collapse given in this Part 3 of Eurocode 8 is closer to the actual collapse of the building than the one given in EN1998-1: 2004 and corresponds to the fullest exploitation of the deformation capacity of the structural elements. The Limit State associated with the 'no collapse' requirement in EN1998-1: 2004 is roughly equivalent to the one that is here defined as Limit State of Significant Damage.

(2)P The National Authorities decide whether all three Limit States shall be checked, or two of them, or just one of them.

NOTE The choice of the Limit States will be checked in a country, among the three Limit States defined in **2.1(1)**P, may be found in the National Annex.

(3)P The appropriate levels of protection are achieved by selecting, for each of the Limit States, a return period for the seismic action.

NOTE The return periods ascribed to the various Limit States to be checked in a country may be found in its National Annex. The protection normally considered appropriate for ordinary new buildings is considered to be achieved by selecting the following values for the return periods:
- LS of Near Collapse (NC): 2.475 years, corresponding to a probability of exceedance of 2% in 50 years
- LS of Significant Damage (SD): 475 years, corresponding to a probability of exceedance of 10% in 50 years
- LS of Damage Limitation (DL): 225 years, corresponding to a probability of exceedance of 20% in 50 years.

BASIS OF DESIGN – Limit states & return periods

EN1998-3:2005 vs prEN1998-3:2023

- Limit states:
  - Part3'23 refers to Part 1-1
  - Assessment ≠ Design: if a single LS is to be checked, it is NC
- Return periods (NDP):
  - Part3'23 refers to Part 1-1
  - NC: **1600 years** for CC2
  - Related to target reliability $\beta_{t,LS,CC}$ → safety format, later
  - Lower values of $\beta_{t,LS,CC}$, e.g., reflecting shorter residual life, explicitly mentioned as a possibility
    - Decision by relevant authorities

4.1 Performance requirements

(1) The performance requirements shall refer to the state of damage in the structure, herein described through the Limit States (LS) defined in prEN 1998-1-1:2022, 4.3(1).

(2) A seismic action should be associated with each Limit State to be verified. This seismic action should be characterised by its return period $T_{LS,CC}$ according to prEN 1998-1-1:2022, 4.3(3), or, alternatively, by a performance factor $\gamma_{LS,CC}$ according to prEN 1998-1-1:2022, 4.3(5).

NOTE The minimum values to be ascribed to $T_{LS,CC}$ or, alternatively, to $\gamma_{LS,CC}$ for each type of selected existing structure, for use in a country, can be provided by the relevant authorities or can be found in the National Annex. They can be lower than those used for new structures, if lower values of $\beta_{t,LS,CC}$ are accepted for existing structures with respect to those specified for new structures by the relevant authorities or in the National Annex (see the note in prEN 1998-1-1:2022, 4.3(3)). Lower values of $\beta_{t,LS,CC}$ in the service life t_L can reflect a shorter residual service life of an existing structure. In the absence of such requirements, the choice of the corresponding value can be agreed for a specific project by the relevant parties.

(3) The seismic performance of the structure should be verified for the full set or a subset of the four Limit States; as a minimum, the Near Collapse LS should be verified.

NOTE 1 Since existing structures in general do not possess the adequate ductility ensured in new ones by means of capacity design and detailing for local ductility, verification of the LS of Significant Damage for a certain intensity does not necessarily imply verification of the LS of Near Collapse for a higher one.

NOTE 2 The types of structures to which this standard applies in a Country and the choice of the Limit States to be verified in a country for each type of existing structure can be found in the National Annex or can be elsewhere provided by the relevant Authorities., and they can be different from those used for new structures. In the absence of such requirements, the choice of additional Limit States to be verified can be agreed for a specific project by the parties involved.

BASIS OF DESIGN – Reference analysis method

Force-based approach (FBA)

- This is a limited exception, since
  - q values for NC not available
  - In general, ductility supporting q for new structures (@SD) not available
- Permitted with q lower than for new structures…
- …and NC intensity (i.e., $T_{\mathrm{NC,CC2}} = 1600 > T_{\mathrm{SD,CC2}} = 475$ years)
  - In low, maybe moderate seismic action class, could lead to positive outcome → ok
  - If negative → reanalysis with DBA

4.1 Performance requirements

(4) In application of prEN 1998-1-1:2022, 6.1(4), in the cases of low and moderate seismic action class (prEN 1998-1-1:2022, 4.1(4)), the seismic performance of structures may be verified by means of the force-based approach, using the q values in 6.3.1(1) and the seismic action for the Near Collapse LS.

NOTE The force-based approach relies on global ductile behaviour and redistribution capacity of the structure. Its application to existing structures is thus approximate and only reduced values of q, corresponding to member overstrength, can be used. As a result, the method is conservative. If the outcome of the verification is negative, this can be due to this conservatism, rather than to an actual deficit of the structure. The state of the structure can be re-assessed via a more accurate displacement-based approach to calibrate the retrofit design.

Displacement-based approach (DBA)

- This is the general method
  - Tentatively advanced in Part3'05
  - Now possible for all structures (old & new)
    - DBA is in Parts 1-1 to 5
  - Based on nonlinear (static) analysis and deformation checks (+ force checks for brittle)
  - Requires specific safety format to ensure same safety as FBA, format should:
    - Be not in contrast with assessment for non-seismic design situations
    - Reflect specific uncertainties in assessment

BASIS OF DESIGN – Compliance criteria

Different uncertainties → different partial factors

Higher level reliability method

- Part1-1'23 introduces a method for probabilistic assessment (CC3b or CC4?)!

Partial factor method

- Design value of action effects E_d
- Additional uncertainty in assessing a damaged structure

$$E_d = \gamma_{Sd} E\{X_d; a_d; \sum F_{Ed}; A_{Ed}\}$$

- Design value of resistance R_d
- A single partial factor γ_{Rd} replaces the confidence factor CF and material factors γ_c, γ_s, etc and describes all uncertainties

$$R_d = \frac{1}{\gamma_{Rd}} R\{X_d; a_d; \underbrace{\sum F_{Ed}; A_{Ed}}_{\text{e.g., } N + \Delta N}\}$$

4.2 Compliance criteria for existing structures

4.2.1 Specificity of existing structures

(3) When designing a structural intervention for resistance against seismic actions, structural verifications should also be made with respect to non-seismic situations.

(4) Different sets of material and structural partial factors should be used, as well as different analysis procedures, depending on the completeness and reliability of the information available.

4.2.2 Verification rules

(4) Unless higher level (reliability) methods are employed (prEN 1998-1-1:2022, Annex F), safety verifications should be carried out using the partial factor method, according to prEN 1990:2021, 8.

(5) Design values of action effects (E_d) should be expressed according to prEN 1990:2021, 8.3.2.1(1), including all relevant actions for the seismic combination (permanent, variable, seismic). For the application of this general expression in the present Eurocode part, the following definitions of symbols should be applied.

γ_{Sd} is a partial factor considering uncertainty in modelling the action effects, depending on the state of the structure, and being equal to 1,0 for undamaged structures, and 1,15 otherwise;

A_{Ed} is the design value of the seismic action, appropriate for the Limit State to be verified (see 4.1(3)).

(6) Design values of the resistance (R_d) should be expressed according to the general expression given in prEN 1990:2021, 8.3.5.1(1). For the application of this general expression in the present standard, the following definitions of symbols should be applied.

γ_{Rd} is a partial factor accounting for uncertainty in the resistance (strength or deformation) model, calculated according to note 2 of prEN 1998-1-1:2022, 6.7.2(1);

$X_{d,i}$ is the design value of the i-th material or product property. For existing materials X_d is obtained from tests of *in situ* properties and from additional sources of information, according to 5.5(1) to (3). For added materials, design values are obtained according to 5.5(4) and (5), respectively.

NOTE The partial factor γ_{Rd} accounts for: a) target reliability; b) uncertainty in the relevant variables describing geometry, details and material properties entering the resistance model, including their statistical uncertainty of estimation based on limited sample size, plus the model error of the resistance model itself. The values of logarithmic standard deviation as a function of KL are given in this standard.

BASIS OF DESIGN – Partial factors method

Design value of i-th material or product property

$$R_{\mathrm{d}} = \frac{1}{\gamma_{\mathrm{Rd}}} R\{X_{\mathrm{d}}; a_{\mathrm{d}}; \sum F_{\mathrm{Ed}}\}$$

- No CF (un-calibrated) or material factors γ_c, γ_s, etc (calibrated for new construction), as in Part3'05:

$$f_{\mathrm{cd}} = \frac{f_{\mathrm{cm}}}{CF \cdot \gamma_{\mathrm{c}}}$$

- Design values are the mean from **tests + other sources** of information:

$$f_{\mathrm{cd}} = f_{\mathrm{cm}}$$

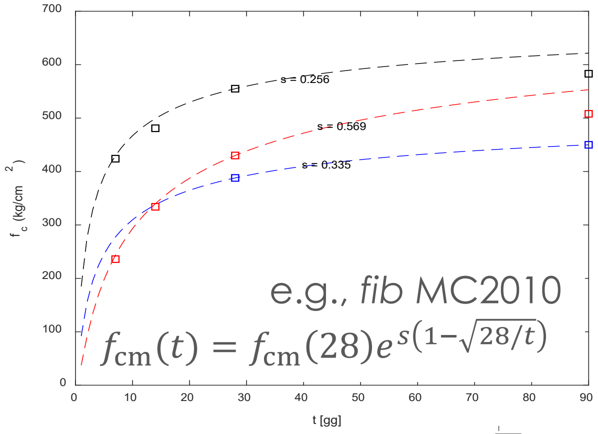
  - Care when combining data from different sources!
  - Same values into model & verifications (ease of use)

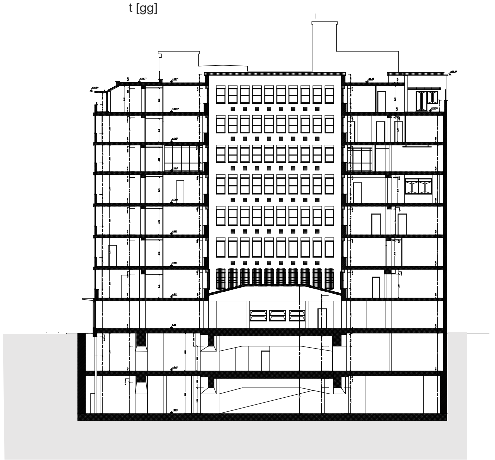
- Note: different means can be used in different parts of the structure for the same property

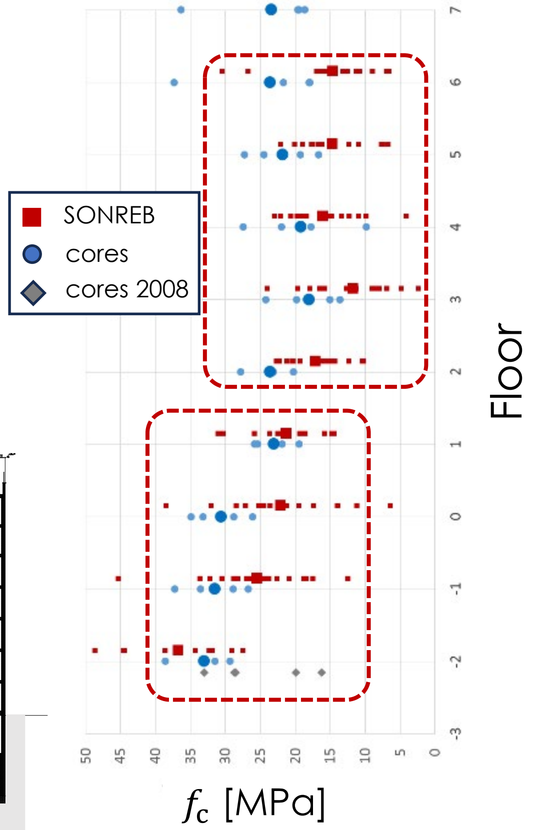
5.5 Representative values of material properties

(1) For existing materials, design values of material properties X_d for calculating resistances to be used in local verifications, should be taken as the mean. Mean values should be obtained from testing and additional sources of information, and different mean values may be considered in different areas of the structure, as appropriate, based on test results.

NOTE The variability of the material properties and the associated uncertainty of estimation based on a limited number of test values are accounted for through KL-dependent partial factors γ_{Rd}, specified for each resistance model, as a function of KLG, KLD and KLM in the relevant subclauses of 8 to 11.







BASIS OF DESIGN – Partial factors method

Design value of i-th material or product property

- $s_{\ln X} \cong$ CV when $\leq 0{,}3$
- Very important parameter, always report it

- Reference values (NOTE 1):
  - come from years of application of Part3'05-like national codes
  - refer to **intra-building values** (larger values reported in the literature often come from meta-analysis not accounting for inter-building variability)

- NOTE2: If larger than reference values, it rings a bell, indicating either
  - Poor construction quality
  - Possible unreliable testing

5.5 Representative values of material properties

(2) The standard deviation of the natural logarithm $s_{\ln X}$ of relevant test values (i.e. tests on the existing material and in the area of the structure under consideration) should always be reported, unless the properties are not derived from testing of *in situ* materials, as is the case for KLM1 for reinforcing steel, timber and masonry.

NOTE 1 Standard deviation of the natural logarithm of material strengths within the same structure are, indicatively, in the ranges:

- Infill walls: 0,20 to 0,40
- Concrete: 0,10 to 0,20
- Reinforcing steel: 0,05 to 0,10
- Structural steel: 0,05 to 0,10
- Timber: 0,15 to 0,25
- Masonry: 0,20 to 0,30 (specific values are provided in Annex D, Table D.1)

NOTE 2 Standard deviation of the natural logarithm of material properties determined from tests that lie above the upper bounds of the indicated ranges indicate poor construction quality and a lower confidence in the estimated mean values. In these cases, it is recommended to increase the level of testing of material properties.

BASIS OF DESIGN – Partial factors method

Design value of i-th material or product property

- Materials are classified in:
  - **Existing**
  - **Added**, i.e., new material used in retrofit, e.g., *concrete in a jacket*
  - **New**, new material in a new member, e.g., *steel in an exoskeleton*

- Design value = mean always, unless the retrofit choice is to build a new structure to withstand the full seismic action
  → new structure designed as such
- Note:
  - Mean values are used for both model and verifications in the DBA also for new structures
  - Using f_m or f_k has no effect on safety, as long as partial factors account for this (explicitly recognized in EN1990-2)

5.5 Representative values of material properties

(3) For existing materials, when the properties are not derived from testing of *in situ* materials, as is the case of KLM1 for reinforcing steel, timber and masonry, mean values should be obtained from standards in force at the time of construction (steel and timber), considering also a) and b):

a) For concrete, the mean value may be obtained from the characteristic value as given by Formula (5.2).

$$f_{\mathrm{cm}} = f_{\mathrm{ck}} + 8\ \mathrm{MPa} \tag{5.2}$$

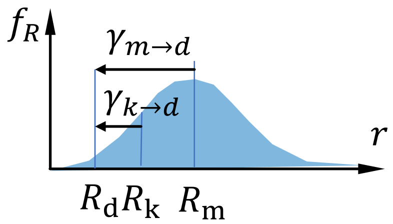
b) For steel and timber, characteristic values usually specified in codes should be converted to mean values accounting for the appropriate standard deviation for the identified steel or timber, assuming normal distribution.

NOTE 1 Appropriate standard deviation values are those given in NOTE 1 to (2).

NOTE 2 Additional information is given for masonry in Annex D, D.2.

(4) For added materials, design values of material properties X_{d} for calculating resistances to be used in local verifications, should be defined as the mean. The mean may be derived from the characteristic value as indicated in (3).

(5) For new materials, design values of material properties should be:



How have the new partial factors γ_{Rd} been calibrated?

BASIS OF DESIGN – Partial factors method

Reliability bases for the new partial factors format

- Reliability-based calibration of partial factors is an optimization problem:

$$\min_{\boldsymbol{\gamma}} \sum_{\mathrm{i}} w_{\mathrm{i}} [\beta_{\mathrm{t}} - \beta_{\mathrm{i}}(\boldsymbol{\gamma})]^2$$

  - Reliability index $\beta_{\mathrm{i}}(\boldsymbol{\gamma})$ is determined by reliability analysis of $g_{\mathrm{i}} = R_{\mathrm{i}} - E_{\mathrm{i}}$, where:
    - The distribution of the effect f_{E} and the uncertainty on resistance σ_{R} are known
    - The fractile of resistance R_{ki} is obtained through $E_{\mathrm{di}} = \gamma_{\mathrm{E}} E_{\mathrm{ki}} = R_{\mathrm{ki}}/\gamma_{\mathrm{R}} = R_{\mathrm{di}}$ and thus is function of $\gamma_{\mathrm{E}}\gamma_{\mathrm{R}}$

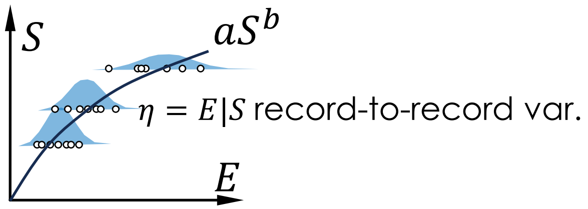
- Simpler alternative, **Design Value Method** (ISO, 1998)
  - Same used to calibrate $\gamma_{\mathrm{c}}, \gamma_{\mathrm{s}}$, etc for non-seismic design situations
  - Direct determination of γ_{E} and γ_{R} given β_{t}
  - EN1992 now gives it for assessment of existing structures in non-seismic design situations
  - If both variables are lognormal:

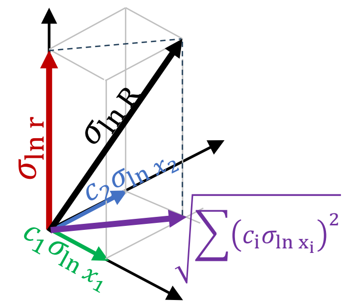
$$\gamma_{\mathrm{E}} = \frac{E_{\mathrm{di}}}{E_{\mathrm{ki}}} = e^{\boxed{\alpha_{\mathrm{E}}^2}\beta_{\mathrm{t}}\sigma_{\mathrm{t}}} e^{-\kappa_{\mathrm{E}}\sigma_{\ln \mathrm{E}}} \qquad \alpha_{\mathrm{E}} = \frac{\sigma_{\ln \mathrm{E}}}{\sqrt{\sigma_{\ln \mathrm{R}}^2 + \sigma_{\ln \mathrm{E}}^2}}$$

$$\gamma_{\mathrm{R}} = \frac{R_{\mathrm{ki}}}{R_{\mathrm{di}}} = e^{\boxed{\alpha_{\mathrm{R}}^2}\beta_{\mathrm{t}}\sigma_{\mathrm{t}}} e^{\kappa_{\mathrm{R}}\sigma_{\ln \mathrm{R}}} \qquad \alpha_{\mathrm{R}} = \frac{\sigma_{\ln \mathrm{R}}}{\sqrt{\sigma_{\ln \mathrm{R}}^2 + \sigma_{\ln \mathrm{E}}^2}}$$

  - Used because of the constant values for $\alpha_{\mathrm{E}} = -0{,}7$ and $\alpha_{\mathrm{R}} = 0{,}8$ provided by König&Hosser (1982)
  - New constant values were calculated for the seismic case $\alpha_E = -0{,}91$ and $\alpha_R = 0{,}42$

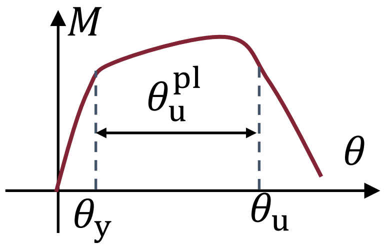
BASIS OF DESIGN – Partial factors method

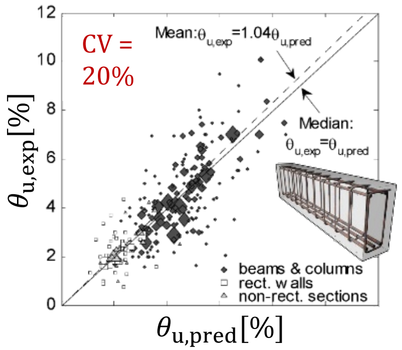


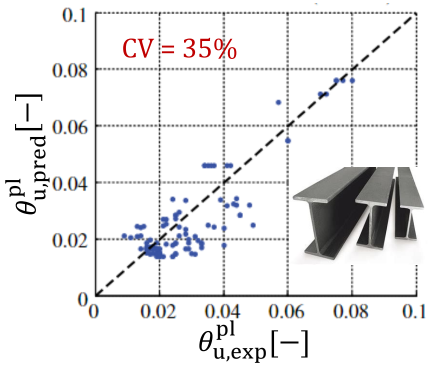


Seismic-specific developments

- $\sigma^2_{\ln E} = b^2\sigma^2_{\ln S} + \sigma^2_{\ln E|S}$ introduced (related to seismic hazard)
- $\sigma^2_{\ln R} = \sigma^2_{\ln r} + \sum (c_i\sigma_{\ln x_i})^2$ evaluated for all resistance models







Total uncertainty in resistance (New RC members)

Model	Section	$\sigma_{\ln r}$	$\sigma_{\ln R}$
θ_u	Rectangular	0.20	0.22
	Circular	0.15	0.17
	Other (e.g., hollow)	0.20	0.21

Table 3: Target reliability: $\beta_{50,t}$ by LS and CC.

LS	CC1	CC2	CC3a	CC3b
NC	1,75	2,33	2,56	2,91
SD	1,20	1,60	1,76	2,00
DL	0,38	0,50	0,55	0,63

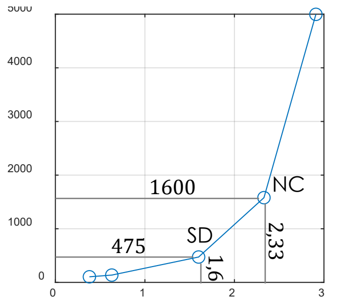
$\gamma_{Rd} = e^{0,85\cdot 2,33\cdot 0,22} = 1,55$

$\gamma_{Rd,KL} > 1,55$

- EN1998: no γ_E is used on seismic action→A <u>single</u> partial factor $\gamma_{Rd} = e^{\bar{\alpha}^*_R\beta_{t,LS,CC}\sigma_{\ln R}}$ is introduced
  - It accounts for uncertainty on both E and R: $\bar{\alpha}^*_R = 0,85 > 0,42$ (average over all seismic action classes)
  - Part 1-1 (Annex E) provides target reliability (NDP)
  - For existing structures $\gamma_{Rd,KL} = e^{\bar{\alpha}^*_R\beta_{t,LS,CC}\sigma_{\ln R}}$ because $\sigma_{\ln R}$ larger and (mildly) KL-dependent
  - For this to work: $T_{LS,CC} = -t_L/\ln\Phi(0,8\beta_{t,LS,CC})$ (risk-targeted seismic action…)
  - Part 1-1 (Clause 4) provides return periods

Table 4: T_R of the seismic action by LS and CC.

LS	CC1	CC2	CC3a	CC3b
NC	600	1600	2500	5000
SD	275	475	600	900
DL	100	115	125	140



Data for assessment

History, geometry, construction details and material properties

DATA FOR ASSESSMENT – Sources

Two types:

- Relevant generic data sources (b)
  - To fill in gaps in structure-specific information, or in the initial stage of assessment, to orient field investigations
- Structure-specific information (a,c,d):
  - Available documentation (original design, design of subsequent interventions, material tests during construction, etc)
  - Field investigations, measurements (including dynamic), material testing, made at the time of assessment

5 Information for structural assessment

5.1 General information and history

(1) In assessing the earthquake resistance of existing structures, the input data should be collected from a variety of sources, including as given in a) to d):

a) available documentation specific to the structure in question;

b) relevant generic data sources (e.g. contemporary codes, standards and documented practice);

c) field investigations and measurements;

d) destructive, *in situ* and/or laboratory, and non-destructive tests (that may include *in situ* measurements of dynamic properties), as described in more detail in 5.2 and 5.4.

(2) Field investigations should also aim at identifying all possible threats to life safety posed by ancillary components, such as, e.g., chimneys, cornices, poorly braced equipment, or inadequate configurations (circulation or vulnerable access). These vulnerabilities should be considered in designing retrofitting interventions.

(3) The following features should be assured in order to ensure an appropriate inspection: accessibility, visibility, lighting, cleaning of surfaces.

(4) Cross-checks should be made between the data collected from different sources to minimise uncertainties. In case of conflicting information, *in situ* structure-specific information should be relied upon.

DATA FOR ASSESSMENT – Data

History of the structure

- A structure-specific piece of info that may provide input on a), e), **f)** → directs towards the correct generic information
  - Assessment has general aspects and others that are very local in space & time:
    - National codes
    - Typical materials
    - Design and construction practice

5 Information for structural assessment

5.2 Required input data

(1) The information for structural assessment should cover the items defined in a) to i):

a) Identification of the structural system. The information should be collected either from on-site investigation or from original design and/or construction drawings, if available. In the latter case, information on possible structural changes since construction should also be collected.

b) Identification of the type of foundations.

c) Identification of the site conditions as defined in prEN 1998-1-1:2022, 5.1.

d) Information about the overall dimensions and cross-sectional properties of the structural members and the mechanical properties and condition of constituent materials.

e) Information about identifiable material defects and inadequate detailing.

f) Information on the seismic design criteria and the level of seismic action used for the initial design. In the case of non-engineered structures, information on the compliance with the rules of practice normally used in the area.

g) Description of the present and/or the planned use of the structure (with identification of its consequence class, as described in the relevant part of EN 1998).

h) Re-assessment of imposed actions considering the future use of the structure.

i) Information about the type and extent of previous and present structural damage, if any, including earlier repair and retrofitting measures.

DATA FOR ASSESSMENT – Data

History of the structure

- A structure-specific piece of info that may provide input on a), e), **f)** → directs towards the correct generic information
    - Assessment has general aspects and others that are very local in space & time:
        - National codes
        - Typical materials
        - Design and construction practice

National Annex!

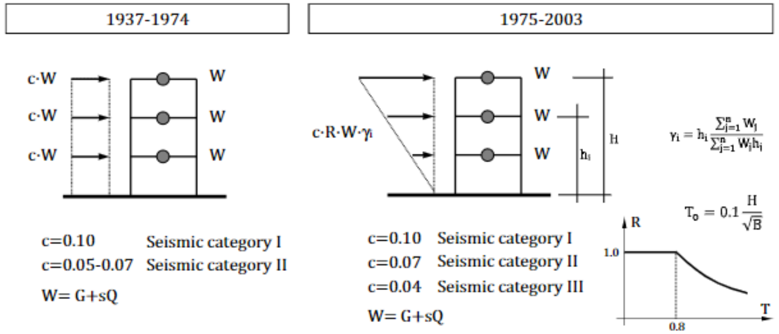


Figure 2. Seismic load definition pre- (left) and post-1975 (right).

Table 2. Evolution of concrete classes and corresponding mechanical properties.

			RD 16/11/1939		DM 30/05/1972 – DM 30/05/1974 – DM 16/06/1976 DM 27/07/1985 – DM 14/02/1992					
Concrete class			Portland	High Strength	Rck 15	Rck 20	Rck 25	Rck 30	Rck 40	Rck 50
Compressive strength	R_{cm}/R_{ck}*	[MPa]	12	16	15	20	25	30	40	50
Allowable	$\sigma_{c,\ N+M}$	[MPa]	4.0	5.00	6.00	7.25	8.50	9.75	12.25	14.80
normal stress	$\sigma_{c,\ N}$	[MPa]	3.5	4.50	4.20	5.08	5.95	6.83	8.58	10.30
Allowable	τ_{co}	[MPa]	0.40	0.60	0.40	0.47	0.53	0.60	0.73	0.87
shear stress	τ_{c1}	[MPa]	1.40	1.60	1.40	1.54	1.69	1.83	2.11	2.40

(*) R_{cm} is the mean value of the concrete compressive strength (valid for RD 16/11/1939); R_{ck} is the characteristic compressive strength and it was introduced by DM 30/05/1972

Table 3. Evolution of reinforcing steel classes and corresponding mechanical properties.

		RD 16/11/1939			Circolare 23/05/1957				DM 30/05/1972					DM 30/05/1974 DM 16/06/1976 DM 27/07/1985 DM 14/02/1992			
		plain			plain			def.	plain		def.			plain		def.	
Steel class		L-C**	M-C**	H-C**	AQ 42	AQ 50	AQ 60	Sp (*)	FeB 22k	FeB 32k	A38	A41	FeB 44k	FeB 22k	FeB 32k	FeB 38k	FeB 44k
Yield stress	[MPa]	230	270	310	230	270	310	-	220	320	380	410	440	220	320	380	440
Ultimate	[MPa]	420–500	500–600	600–700	420–500	500–600	600–700	-	340	500	460	500	550	340	500	460	550
Allowable	[MPa]	140	200	200	140	160	180–220	220–240	120	160	190–200	200–240	220–240	120	160	190–220	220–260

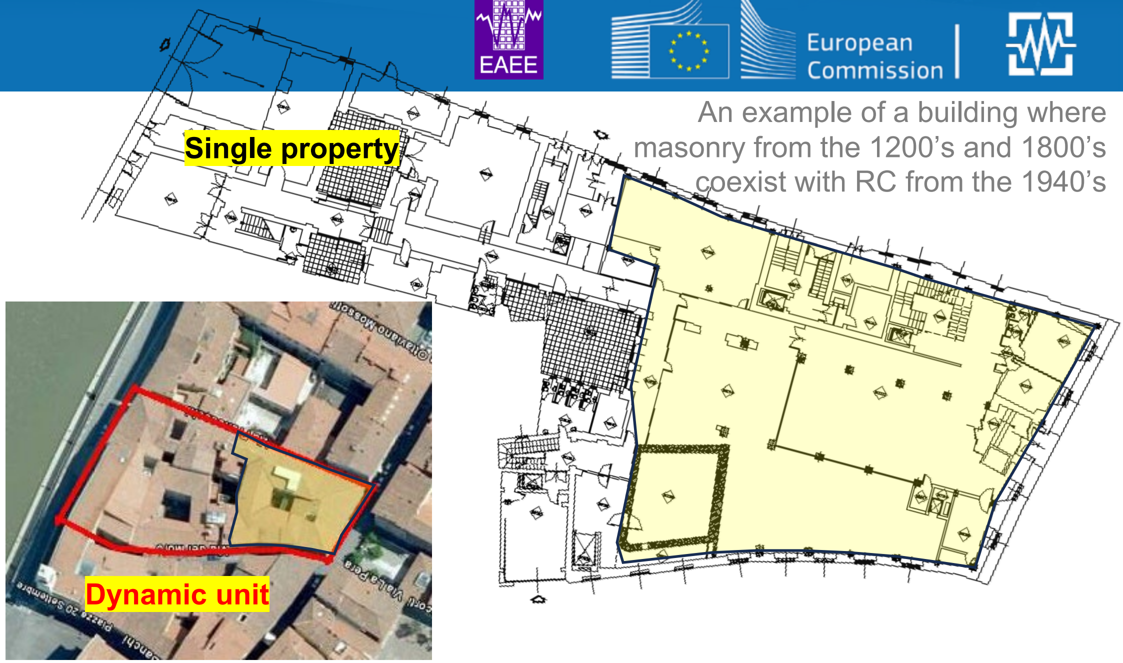
(*) deformed (def.) bars (Sp = special); steel properties modified in the following Circolare 17/05/1965 (n.1547): allowable stress = 220–260 MPa
(**) L-C = low-carbon steel; M-C = medium-carbon steel; H-C = high-carbon steel

Sample info on design practice and typical materials for RC construction in Italy, subdivided by period [De Risi et al 2022. "Modelling and Seismic Response Analysis of Italian pre-code and low-code Reinforced Concrete Buildings. Part I: Bare Frames." J. Earthq. Eng., 1–32.]

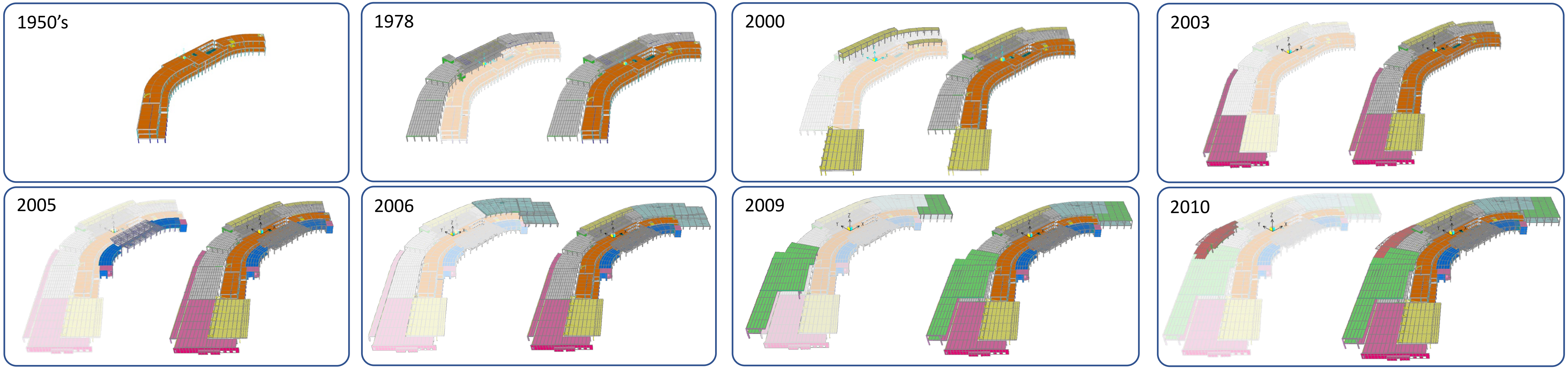
DATA FOR ASSESSMENT – Data

History of the structure

- Helps in identifying distinct homogeneous areas, to be treated separately for inspections & testing
- Typically considered for masonry structures, it is by no means limited to them
  - Besides, mixed materials are a most common feature



An example of a building where masonry from the 1200's and 1800's coexist with RC from the 1940's



Airport terminal built starting in RC (NATO military airport, ACI code from the time) and ending with structural steel. Almost each addition was designed and built according to a different code…

DATA FOR ASSESSMENT – Data

Categories of data and Knowledge Levels

- Categories are the same as in Part3'05
  - Geometry
  - Details
  - Materials
- Knowledge Levels are different
  - One for each category (KLG, KLD, KLM)
  - Can have distinct values
    - Minimum
    - Average
    - High (formerly 'complete')
  - Values can be non-uniform over the structure

5 Information for structural assessment

5.3 Knowledge levels: Definitions

(1) The information collected should be classified into three categories defined in a) to c):

a) *Geometry*: the geometric properties of the structural system and of such ancillary elements (e.g. masonry infill panels) that may affect the structural response;

b) *Construction details*: these include, as appropriate, the amount and detailing of reinforcement in reinforced concrete, connections between steel and/or timber members, connections between masonry walls and the nature of any reinforcing elements in masonry, the type of lintels and masonry spandrels, connections of floor diaphragms to the lateral-load resisting structure, etc.;

c) *Materials*: the mechanical properties of the constituent materials.

NOTE The investigation of geometric properties extends to all members that affect structural response to the seismic action. This means that some structural members can be excluded, if they can be classified at this stage as secondary, while some ancillary members can be included, like masonry infills, that in general affect stiffness and strength and, especially, when irregularly distributed infills can significantly alter the distribution of action effects.

(2) Amount and quality of the collected information in each category should be expressed through three distinct knowledge levels (KL), as defined in a) to c):

a) KLG: knowledge level on Geometry, as detailed in 5.4.1;

b) KLD: knowledge level on Construction Details, as detailed in 5.4.3;

c) KLM: knowledge level on Material properties (one for each constituent material), as detailed in 5.4.4.

NOTE These KL are used differently.

(3) Each KL defined in (2) should take one of three distinct values, as defined in a) to c):

a) Minimum.

b) Average.

c) High.

(4) Values of KLs may be different in different areas of the structure. Individual values for each category of information may be determined in different areas of the structure.

DATA FOR ASSESSMENT – KL identification

Geometry (KLG)

- Table 5.1: different combinations of original + new information
- Geometry can be derived from:
  - Structural outline drawings
  - Detailed construction drawings

5 Information for structural assessment

5.4 Knowledge levels: identification

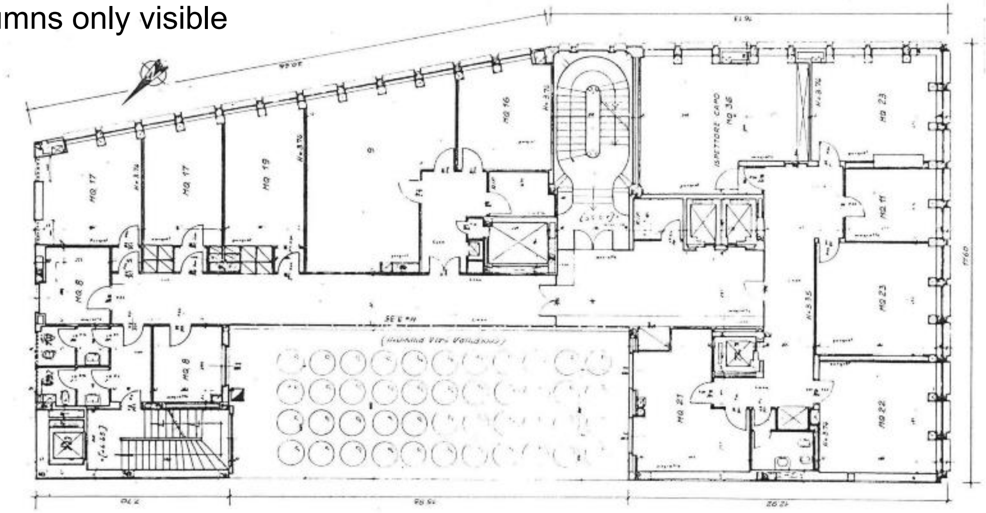
5.4.1 Geometry

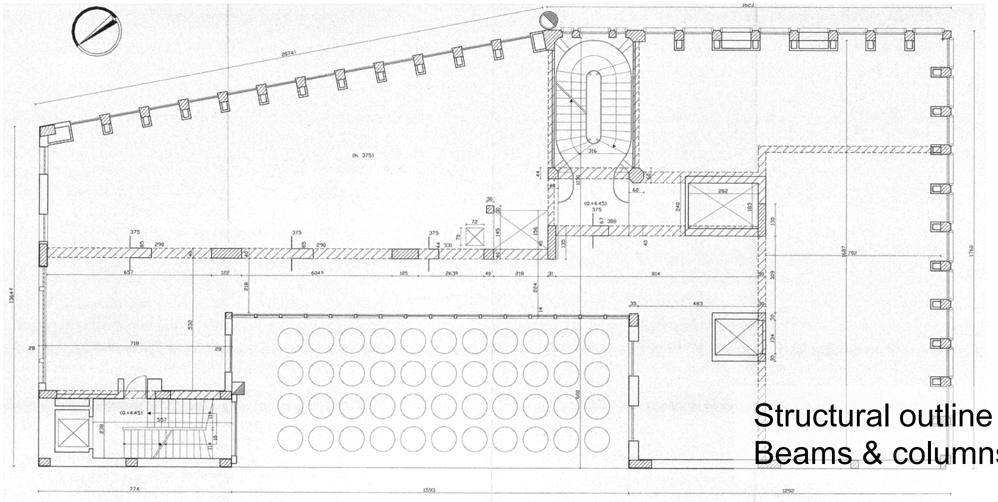
(1) For each type of structural member (beam, column, wall, diaphragm, etc.) and area of the structure, the achieved KL on geometry based on the collected information should be defined, based on Table 5.1.

Table 5.1 — KL on Geometry as a function of collected information

Original design documents (outline or detailed construction drawings)	Extent of survey*		
	L	E	C
Not available	KLG1	KLG2	KLG3
Incomplete set	KLG2	KLG3	
Complete set	KLG3		
* L: limited; E: extended; C: comprehensive (see 3.1.3)			

Architectural drawing
Columns only visible





Structural outline drawing
Beams & columns visible

DATA FOR ASSESSMENT – KL identification

Geometry (KLG)

- Table 5.1: different combinations of original + new information
- Geometry can be derived from:
  - Structural outline drawings
  - Detailed construction drawings

5 Information for structural assessment

5.4 Knowledge levels: identification

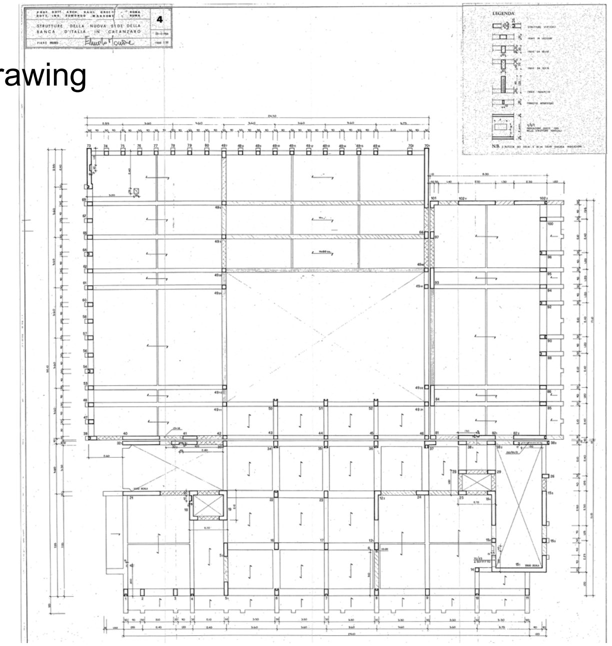
5.4.1 Geometry

(1) For each type of structural member (beam, column, wall, diaphragm, etc.) and area of the structure, the achieved KL on geometry based on the collected information should be defined, based on Table 5.1.

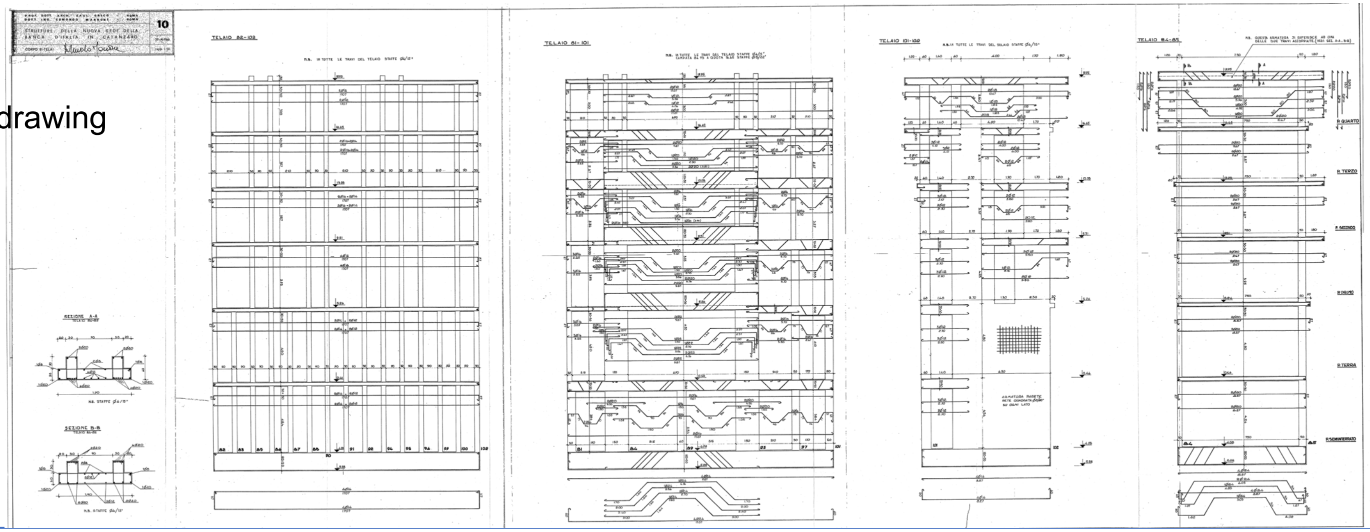
Table 5.1 — KL on Geometry as a function of collected information

Original design documents (outline or detailed construction drawings)	**Extent of survey***		
	L	**E**	**C**
Not available	KLG1	KLG2	KLG3
Incomplete set	KLG2	KLG3	
Complete set	KLG3		
* L: limited; E: extended; C: comprehensive (see 3.1.3)			

Outline drawing



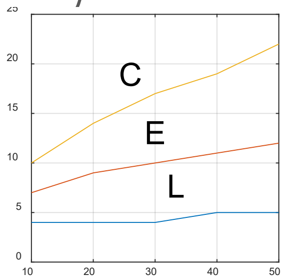
Detailed drawing



DATA FOR ASSESSMENT – KL identification

Extension of geometrical survey

- Percentage of elements to be inspected depends on the size of the structure (decreases with it):
$$m = p \cdot n$$
is not linear in n



- Table 5.2: values are related to statistical uncertainty on the geometrical dimensions, which is accounted for in γ_{Rd}
  - Direct link between survey and verifications
- Accessibility:
  - Avoid inspecting too many similar members, just because easily accessible
  - Horizontal members more difficult to inspect + often less relevant (Limited is enough)
- Survey should cover the entire structure
  - Model cannot be set up without geometry

5 Information for structural assessment

5.4 Knowledge levels: identification

5.4.1 Geometry

(3) For each type of structural member (column, wall, beam, diaphragm, etc.), the minimum percentage of members (reinforced concrete or steel) that should be surveyed for dimensions is given by Formula (5.1), depending on the required extend of survey.

$$p = p_1 n^{-c} \leq 100 \tag{5.1}$$

where

n is the total number n of members of this type in the structure, determined according to (5);

p_1 and c are coefficients which should be taken from Table 5.2 for each level of survey.

Table 5.2 — Minimum requirements for different levels of survey (vertical members)

Level of survey	Limited (L)	Extended (E)	Comprehensive (C)
p_1	200	250	300
c	0,8	0,6	0,5

(4) The values of p_1 and c in Table 5.2 should be used for vertical members; for horizontal members, irrespective of the target KL, Limited survey may be undertaken.

NOTE The level of survey for horizontal members is not taken into account in determining the KL.

(6) Conditions of symmetry and repetitiveness should be considered in planning surveys to avoid concentrating efforts on similar members.

NOTE Considering symmetry and repetitiveness means that the target percentage p of members to be inspected is not achieved by inspecting too many similar members.

(7) In buildings, members inspected should cover the entire height.

DATA FOR ASSESSMENT – KL identification

Preliminary analysis

- Tool allowing to focus inspections on details and tests on materials on limited areas
- Displacement-based analysis:
  - NC spectrum & Assumed mean properties
  - Masonry: nonlinear
  - RC: linear (full spectrum)
    - Mean properties (assumed)
    - 25% cracked stiffness
    - Including masonry infills if irregular
    - D/C ratio of i-th member and k-th floor

$$\lambda_{ki} = \frac{\theta_{ki}}{\theta_y} \rightarrow \lambda_k = \frac{\sum_i V_{ki}\lambda_{ki}}{\sum_i V_{ki}} \rightarrow \max \lambda_k \text{ critical floor}$$

where $\theta_y = 2\frac{\phi_y L_V}{3}$

and $\phi_y = c\frac{f_y}{E_s h}$

5 Information for structural assessment

5.4.2 Preliminary analysis

(1) Once the geometry of the structure is known, a preliminary analysis may be carried out to identify critical portions (e.g. storeys) and members in the structure and inform the planning of inspection of construction details and tests on material properties.

NOTE It is highly recommended that the engineer develops a qualitative view of the structural behaviour and, while achieving this by carrying out a preliminary analysis is not mandatory, it is strongly encouraged. The value of preliminary analysis in this context is not necessarily the seismic rating that it delivers but the opportunity it provides to gain a holistic view of potential structural weaknesses. The preliminary analysis, carried out with nominal properties, is in general linear elastic for reinforced concrete, steel frame structures and timber structures, and non-linear static for masonry structures. More detail can be found in Annex A.

(2) If a preliminary analysis is carried out according to (1), further investigations on construction details (5.4.3) and material properties (5.4.4) may be limited to, or focus mainly on, the identified critical portions. In the latter case the total number of members n should refer to the number of members in the identified critical portions.

A.3 Reinforced concrete structures

(1) For reinforced concrete structures, preliminary analysis may be carried out as a lateral force analysis, when applicable, or a multi-mode response spectrum analysis, with unreduced-elastic response spectrum, with a simplified resistance evaluation. The purpose of this analysis is to evaluate the distribution of inelastic demand throughout the structure, in order to identify areas where ductility demand is expected to be higher.

(2) Mean material properties to be used in the model and for resistance evaluation according to A.3 should be taken equal to typical (mean) values for materials used at the time of construction. Cracked stiffness in the model may be taken equal to 25% of gross.

NOTE 1 Typical values for material properties by age of construction to be used in preliminary analysis can be found in the National Annex.

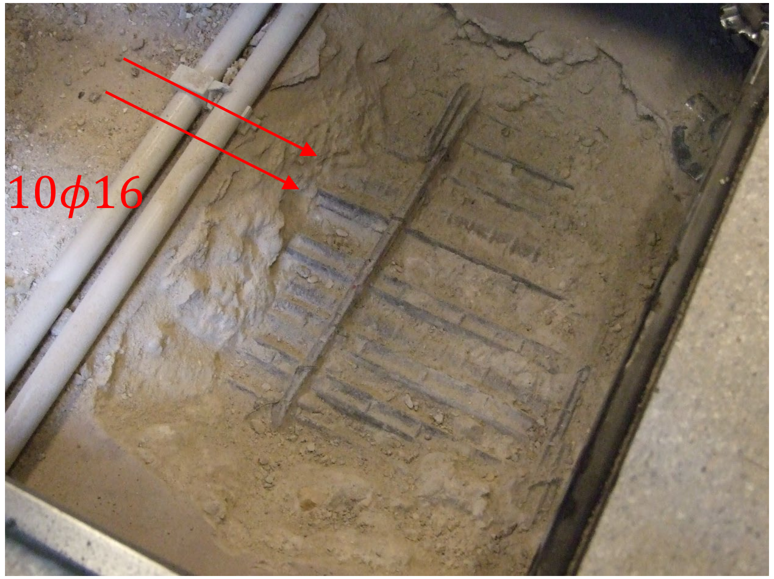
NOTE 2 The preliminary analysis described in this Annex is displacement-based. Action effects are in terms of chord rotations. For this reason, the cracked to initial stiffness ratio is lower than the common 50% used to get conservative estimates of forces. The value of 25% is an average value for effective (secant to yield) to initial stiffness ratio.

(3) If preliminary analysis is undertaken to identify critical areas where to focus further inspections on materials and details (as in A.2(1) a)), masonry infills may be included in the model and should always be included in the case of infills irregularly distributed along the height of the building.

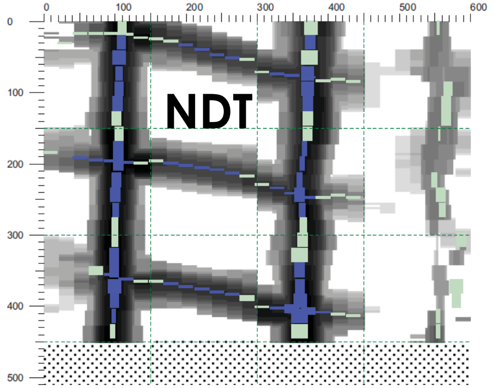
DATA FOR ASSESSMENT – KL identification

Details (KLD)

- Only detailed drawings can be used
- Destructive methods preferred
- Reliable non-destructive (calibrated) permitted



Bars in the top layer are not of the same type: TOR bars and regular deformed bars. This cannot be detected with non destructive techniques.



5 Information for structural assessment

5.4.3 Construction details

(1) Destructive inspection methods should in general be preferred. Reliable non-destructive methods may also be adopted in the inspections. Calibration against destructive methods should be carried out to an extent depending on the non-destructive method.

NOTE Cover removal (to inspect the reinforcement) is an example of destructive method.

(2) For each type of structural member (beam, column, wall, diaphragm, etc.) and area of the structure, the achieved KL on Construction Details based on the collected information should be defined, based on Table 5.3.

Table 5.3 — KL on Construction Details as a function of collected information

Original design documents (detailed structural drawings)	Inspections*		
	L	E	C
Not available	KLD1	KLD2	KLD3
Incomplete set	KLD2	KLD3	
Complete set	KLD3		

* L: limited; E: extended; C: comprehensive (see 3.1.5).
** For meaning of 1, 2, 3 see 5.4.4.

(3) 5.4.1(3) should be applied.

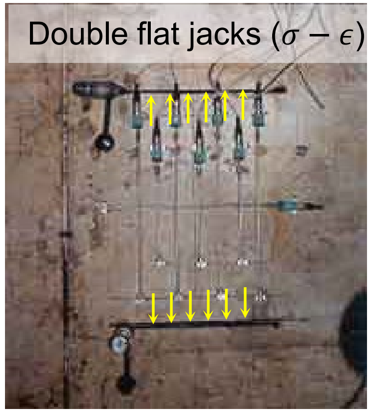
(4) In buildings, members inspected should cover the entire height, unless focusing on an identified critical portion is justified based on a preliminary analysis (5.4.2(2)).

DATA FOR ASSESSMENT – KL identification

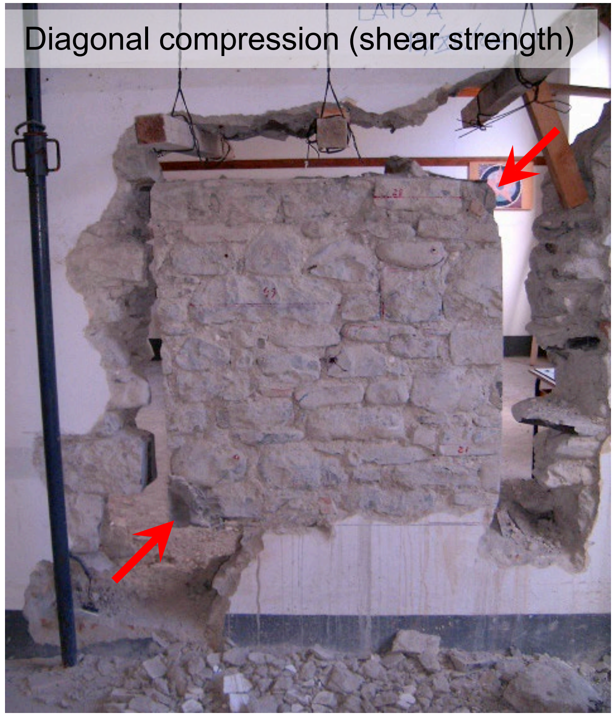
Materials (KLM)

- A matrix similar to those given in Tables 5.1 and 5.2 could be given for **RC** or **steel**, but it cannot be generalized to **masonry** & **timber**
- Destructive methods can mostly be avoided with the latter materials

Single flat jack (σ)



Double flat jacks ($\sigma - \epsilon$)



Diagonal compression (shear strength)

5.4.4 Materials

(1) The knowledge levels concerning the properties of materials in the structure should be classified according to definitions a) to c):

a) KLM1 (Minimum Knowledge) is attained when no direct information on the mechanical properties of the construction materials is available, either from original design specifications or from original test reports. Default values should be assumed in accordance with standards at the time of construction, accompanied by limited *in situ* testing in the most critical members. In the case of masonry structures, direct testing may be avoided, and reference values of predefined masonry types may be attributed after an extended visual survey of masonry features (according to Table 5.1). In the case of timber buildings and timber members, direct testing may be avoided provided that an accurate visual inspection is performed according to 10.2.4.1;

b) KLM2 (Average Knowledge) is attained when information on the mechanical properties of the construction materials is available either (i) from extended *in situ* testing; or (ii) from original design specifications complemented by limited *in situ* testing. In the case of masonry structures, when original design documents are not available, direct testing may still be avoided, but, in addition to what is required for KLM1, the knowledge should be enhanced by extended non-destructive testing, as specified in Table 5.3 for inspections, which allows a more accurate classification of masonry types in the structure. In the case of pre-1940 timber buildings, when original design documents are not available, direct testing may be avoided, but, in addition to what is required for KLM1, the knowledge should be enhanced by non-destructive testing, as specified in Table 10.1;

NOTE In case of masonry and pre-1940 timber structures, original design documents are rarely available.

c) KLM3 (High Knowledge) is attained when information on the mechanical properties of the construction materials is available either (i) from comprehensive *in situ* testing; or (ii) from original test reports, complemented by limited *in situ* testing; or (iii) from original design specifications, complemented by extended *in situ* testing. In the case of masonry structures, in addition to what is required for KLM2, direct testing of material properties in the critical areas should be performed, in order to update the reference values of predefined masonry types; material properties should then be defined by using results of tests for updating the reference values for the masonry types. In the case of timber structures, in addition to what is required for KLM2, (semi) non-destructive testing, e.g. by resistance drilling, and/or density measurements on small samples in order to define the material properties in the critical zones should be performed (see Table 10.1).

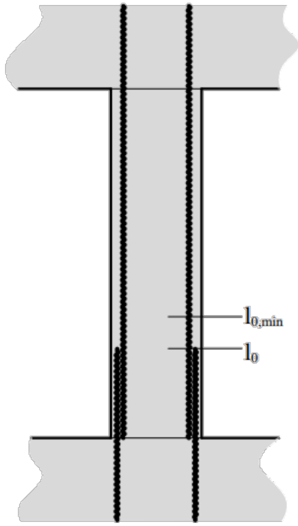
DATA – Material-specific provisions: RC

Geometry

- Lateral load-path in both directions
- Members' size
- Orientation of one-way floor slabs
- Eccentricities

Details

- Amount of longitudinal (including slabs for T- & L-sections) & transverse reinforcement
- Seating length for simply supported members
- Lap-splices
  - Lap-splices cannot be reliably established w NDT
  - Most often cover removal done at mid-height → l_o not inspected, must be assumed < $l_{o,min}$



8.2 Identification of geometry, details and materials

8.2.2 Geometry

(1) The collected data should include all items a) to e):

a) Identification of the lateral load-resisting systems in the two main horizontal directions;

b) Orientation of one-way floor slabs;

c) Depth and width of beams, columns and walls;

d) Width of flanges in T-beams or L-beams (see FprEN 1992-1-1:2022, 7.2.3(2), for the definition of the effective width of flanges);

e) Possible eccentricities between axes of beams and columns at joints.

8.2.3 Details

(1) The collected data should include all items a) to f):

a) Amount of longitudinal steel bars in beams, columns and walls;

b) Amount and detailing of confining steel in critical regions and in beam-column joints;

c) Amount of steel reinforcement in floor slabs contributing to the negative resisting bending moment of T- or L-beams;

d) Seating lengths and support conditions of horizontal members;

e) Depth of concrete cover;

f) Lap-splices of longitudinal reinforcement.

(2) If lap-splice length is not reliably established for each structural member typology of the structure or its critical portion, when identified based on a preliminary analysis, short lap-splice should be assumed in the evaluation of deformation capacity according to 8.4.2, taking l_0 equal to 2/3 of $l_{0y,min}$.

NOTE According to 5.4.3(1), destructive inspection methods should be preferred in general. In particular, lap-splice length is not a parameter that can be easily or reliably established through an indirect non-destructive method.

DATA – Material-specific provisions: RC

Materials

- Original design information can be from:
  - **Specifications in report** or drawings

Concrete: $R_c > 214 \frac{kg}{cm^2}$

5

c) Copertura del salone pubblico: sarà realizzata con cupolette trasparenti in plastica inserite in una maglia di nervature incrociate.

d) Calcestruzzi: nei getti verranno impiegati calcestruzzi confezionati con cemento tipo "730" con carico di rottura pari almeno a 214 kg/cmq, risultando la massima tensione di compressione uguale a 71,3 kg/cmq. La massima tensione tangenziale è di 11,10 kg/cmq.

e) Armature: verrà usato ferro tondino Aq 42 con tensione massima di esercizio di 1400 kg/cmq.

f) Fondazioni: saranno del tipo continuo a travi rovesce, impostate sul terreno roccioso (arenaria compatta, come da pozzo di saggio; (vedi sezione geognostica del 25/7/1965) mediante un sottofondo di calcestruzzo magro. Le fondazioni della sacristia saranno ad un livello alquanto inferiore a quello delle fondazioni generali dell'edificio. La massima pressione unitaria sul terreno è stata valutata in 6,60 kg/cmq.
Si ritiene questa pressione ammissibile, data la natura del terreno. Giova tuttavia osservare che essa non è una pressione di esercizio ma è da considerare eccezionale, perchè si verifica sotto una parete irrigidente nell'ipotesi di scossa ondulatoria. Sotto la sola azione dei carichi verticali, infatti, la pressione unitaria non supera il valore massimo di 2,70 kg/cmq.

Steel quality Aq42

5

c) Copertura del salone pubblico: sarà realizzata con cupolette trasparenti in plastica inserite in una maglia di nervature incrociate.

d) Calcestruzzi: nei getti verranno impiegati calcestruzzi confezionati con cemento tipo "730" con carico di rottura pari almeno a 214 kg/cmq, risultando la massima tensione di compressione uguale a 71,3 kg/cmq. La massima tensione tangenziale è di 11,10 kg/cmq.

e) Armature: verrà usato ferro tondino Aq 42 con tensione massima di esercizio di 1400 kg/cmq.

f) Fondazioni: saranno del tipo continuo a travi rovesce, impostate sul terreno roccioso (arenaria compatta, come da pozzo di saggio; (vedi sezione geognostica del 25/7/1965) mediante un sottofondo di calcestruzzo magro. Le fondazioni della sacristia saranno ad un livello alquanto inferiore a quello delle fondazioni generali dell'edificio. La massima pressione unitaria sul terreno è stata valutata in 6,60 kg/cmq.
Si ritiene questa pressione ammissibile, data la natura del terreno. Giova tuttavia osservare che essa non è una pressione di esercizio ma è da considerare eccezionale, perchè si verifica sotto una parete irrigidente nell'ipotesi di scossa ondulatoria. Sotto la sola azione dei carichi verticali, infatti, la pressione unitaria non supera il valore massimo di 2,70 kg/cmq.

8.2.4 Materials

8.2.4.1 General

(1) The collected data should include items a) and b):

a) Concrete strength;

b) Steel yield strength, ultimate strength and ultimate strain.

(2) For each type of member (beam, column, wall, etc.), the achieved KL on each material (KLM) should be based on the collected information, as given in Table 8.1 (concrete and steel reinforcement).

Table 8.1 — KL on Materials as a function of collected information on concrete or steel reinforcement

Original design documents	Testing		
	L	E	C
Not available	KLM1 (*)	KLM2	KLM3
Design specifications (**)	KLM2	KLM3	
Material test reports	KLM3		

* When original design documentation on material is not available and testing is not undertaken (as allowed for reinforcing steel), default values according to the ruling standards at the time of construction or the state of practice can be assumed.

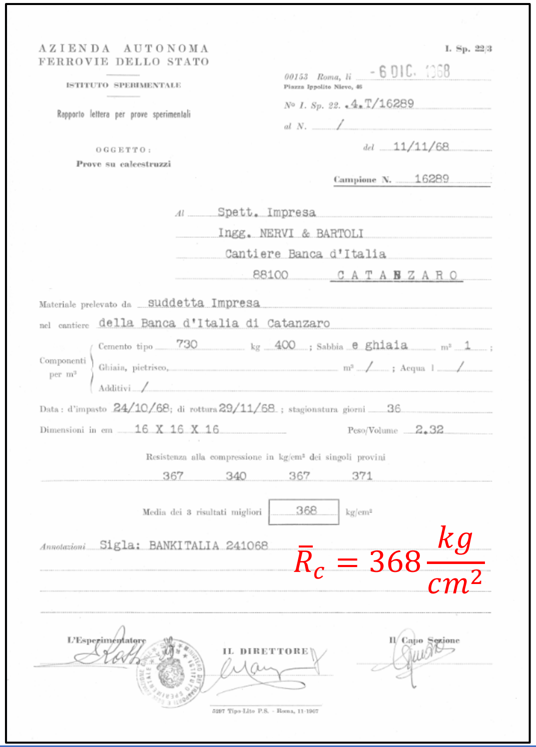
** For instance, from design report or notes on drawings.

NOTE Default values for the material properties based on state of practice and ruling standard as a function of time of construction can be found in the National Annex.

DATA – Material-specific provisions: RC

Materials

- Original design information can be from:
  - Specifications in report or drawings
  - **Tests** performed **at time of construction**



AZIENDA AUTONOMA
FERROVIE DELLO STATO

ISTITUTO SPERIMENTALE

Rapporto lettera per prove sperimentali

I. Sp. 22/3

00153 Roma, li -6 DIC. 1968
Piazza Ippolito Nievo, 46

Nº I. Sp. 22. .4.T/16289

al N. /

del 11/11/68

OGGETTO:
Prove su calcestruzzi

Campione N. 16289

Al Spett. Impresa
Ingg. NERVI & BARTOLI
Cantiere Banca d'Italia
88100 CATANZARO

Materiale prelevato da suddetta Impresa

nel cantiere della Banca d'Italia di Catanzaro

Componenti per m³: Cemento tipo 730 kg 400 ; Sabbia e ghiaia m³ 1 ;
Ghiaia, pietrisco, m³ / ; Acqua l /
Additivi /

Data: d'impasto 24/10/68; di rottura 29/11/68 ; stagionatura giorni 36

Dimensioni in cm 16 X 16 X 16 Peso/Volume 2.32

Resistenza alla compressione in kg/cm² dei singoli provini

367 340 367 371

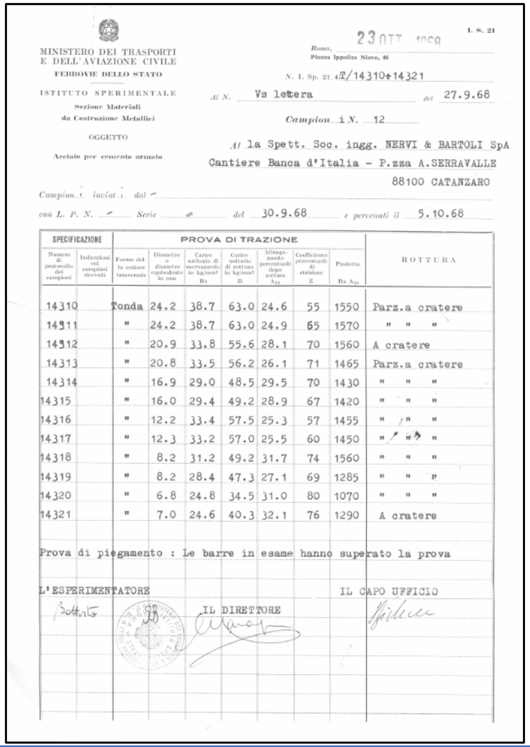
Media dei 3 risultati migliori 368 kg/cm²

Annotazioni Sigla: BANKITALIA 241068

$$\bar{R}_c = 368\,\frac{kg}{cm^2}$$

L'Esperimentatore IL DIRETTORE Il Capo Sezione

5297 Tipo-Lito P.S. - Roma, 11-1967



MINISTERO DEI TRASPORTI
E DELL'AVIAZIONE CIVILE
FERROVIE DELLO STATO

ISTITUTO SPERIMENTALE
Sezione Materiali
da Costruzione Metallici

OGGETTO
Acciaio per cemento armato

I. S. 21

Roma, 23 OTT. 1968
Piazza Ippolito Nievo, 46

N. I. Sp. 21.4T/14310÷14321

Al N. Vs lettera del 27.9.68

Campioni N. 12

Alla Spett. Soc. ingg. NERVI & BARTOLI SpA
Cantiere Banca d'Italia - P.zza A.SERRAVALLE
88100 CATANZARO

Campioni inviati dal –

con L. P. N. – Serie – del 30.9.68 e pervenuti il 5.10.68

SPECIFICAZIONE		PROVA DI TRAZIONE							ROTTURA
Numero di protocollo dei campioni	Indicazioni sui campioni ricevuti	Forma della sezione trasversale	Diametro o diametro equivalente in mm	Carico unitario di snervamento in kg/mm² Rs	Carico unitario di rottura in kg/mm² R	Allungamento percentuale dopo rottura A_{10}	Coefficiente percentuale di strizione Z	Prodotto R x A_{10}	
14310		Tonda	24.2	38.7	63.0	24.6	55	1550	Parz.a cratere
14311		"	24.2	38.7	63.0	24.9	65	1570	" " "
14312		"	20.9	33.8	55.6	28.1	70	1560	A cratere
14313		"	20.8	33.5	56.2	26.1	71	1465	Parz.a cratere
14314		"	16.9	29.0	48.5	29.5	70	1430	" " "
14315		"	16.0	29.4	49.2	28.9	67	1420	" " "
14316		"	12.2	33.4	57.5	25.3	57	1455	" " "
14317		"	12.3	33.2	57.0	25.5	60	1450	" " "
14318		"	8.2	31.2	49.2	31.7	74	1560	" " "
14319		"	8.2	28.4	47.3	27.1	69	1285	" " "
14320		"	6.8	24.8	34.5	31.0	80	1070	" " "
14321		"	7.0	24.6	40.3	32.1	76	1290	A cratere

Prova di piegamento : Le barre in esame hanno superato la prova

L'ESPERIMENTATORE IL DIRETTORE IL CAPO UFFICIO

8.2.4 Materials

8.2.4.1 General

(1) The collected data should include items a) and b):

a) Concrete strength;

b) Steel yield strength, ultimate strength and ultimate strain.

(2) For each type of member (beam, column, wall, etc.), the achieved KL on each material (KLM) should be based on the collected information, as given in Table 8.1 (concrete and steel reinforcement).

Table 8.1 — KL on Materials as a function of collected information on concrete or steel reinforcement

Original design documents	Testing		
	L	E	C
Not available	KLM1 (*)	KLM2	KLM3
Design specifications (**)	KLM2	KLM3	
Material test reports	KLM3		

* When original design documentation on material is not available and testing is not undertaken (as allowed for reinforcing steel), default values according to the ruling standards at the time of construction or the state of practice can be assumed.

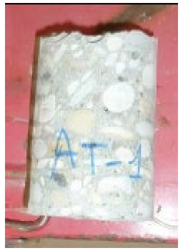
** For instance, from design report or notes on drawings.

NOTE Default values for the material properties based on state of practice and ruling standard as a function of time of construction can be found in the National Annex.

DATA – Material-specific provisions: RC

Concrete strength

- All other properties can be derived from f_c
- A mix of destructive & non-destructive tests
- Identification of homogenous areas as a minimum through structural joints
  - NDT can be used for this
- NDT first, then cores where NDT performed
  - Calibration of correlation e.g., RILEM: $R = 7{,}695 \times 10^{-11} V^{2{,}60} I^{1{,}40}$
- NDTs required: $m = pn$ ($p = p_1 n^{-c}$, Table 5.2)
- Core strength should be corrected for:
  - D (obviously smaller than standard to limit invasiveness)
  - Aspect ratio (often $1 < H/D < 2$, to limit invasiveness and for rectification)



8.2.4.2 Concrete

(1) The investigation of concrete should aim mainly at determining the compressive strength for each area of the structure. Other properties, such as modulus of elasticity, tensile strength etc. may be determined indirectly based on the compressive strength, if no specific investigation is conducted.

(2) A combination of non-destructive methods and destructive methods (such as core sampling) should be made to improve knowledge in more positions, when required for greater reliability.

(4) Only destructive tests should be performed if the number of cores m_c to be taken is larger or equal to the number m of non-destructive test measurements required, depending on the desired KLM, according to (8). In all other cases, non-destructive testing should be carried out prior to core sampling to identify homogeneous areas within the structure (i.e. areas where concrete property values may be assumed to be from the same population, which can be provisionally established based on low sample variability, e.g. a coefficient of variation lower than 15%, or, even with higher coefficients of variation, by statistical testing of the difference in the means or analysis of variance). As a minimum, it may be assumed that each distinct structural block in which the structure is delimited by joints represents a different homogeneous area. When identifying homogeneous areas, the expected systematic variation of concrete strength should be taken into account, depending on its position in the structure, and the conditions of concreting, compaction and maintenance. Statistical tests in EN 13791:2019, 7 may be used to identify homogeneous areas within the structure.

(5) When core sampling serves the purpose of calibrating the results of non-destructive tests, a structure-specific correlation curve should be established through least squares regression based on test results from destructive and non-destructive testing in the structure. Parallel core sampling at positions where non-destructive testing has already been carried out should be performed all within the same homogeneous area. The correlation established by least squares regression in this homogenous area may then be used in other homogeneous areas. The homogeneous area should preferably coincide with the critical area of the structure, as identified by a preliminary analysis, if performed. At least $m_c = 5$ cores should be taken at locations that include the extremes of the indirect test values, to better constrain the regression.

(6) The core strength should be converted into the real *in situ* strength (see EN 13791:2019).

(7) Core testing should be undertaken in conformity to EN 13791:2019, 6(1) to (6).

DATA – Material-specific provisions: RC

Steel properties

- Visual identification (after cover removal):
  - w/o original documents → KLM1
  - + Design report or detailed drawing → KLM2
  - + Original test reports → KLM3
- DT on steel: regular tensile test

- Bars segments extracted with a grinder
- After extraction, bar segment of larger diameter welded to ends of original bar, covered with a protective bi-component grout & reduced shrinkage concrete used to restore the cover
- Usually done at positions where cover removed for visual inspection or if critical elements have small sections, from larger elements like walls

- NDT on steel: hardness test
  - Need calibration vs DT as NDTs on concrete
  - Bar should be uncovered «just enough». If too much concrete removed, vibration problems can alter results
  - Small diameter bars should not be tested, bar curvature makes difficult probe positioning and alter results

8.2.4.3 Steel reinforcement

(1) KLM1 (Minimum knowledge) may be considered as attained if original design documents are not available and classification of steel is done by visual identification (surface smooth or ribbed, any readable markings on the surface of the bars), with consideration of the time of construction of the building. The mechanical properties of steel (yield strength, ultimate strength, ultimate strain) should be taken as specified in the appropriate Standards for the identified category of steel (see notes to 5.5(1) and 5.5(2)).

(2) KLM2 (Average knowledge) may be considered as attained, when either a) or b) applies:

a) original design documents are not available, no readable markings are found during visual identification and the *in situ* properties of steel are determined by testing at least three samples of approximately the same diameter from structural members of the critical portion of the structure, as identified by preliminary analysis if performed;

b) indications on the steel used are available from design specifications (rather than from test reports) and visual identification confirms the information.

(3) KLM3 (High knowledge) may be considered as attained when either a) or b) applies:

a) the *in situ* properties of steel are determined based on testing of at least three samples of approximately the same diameter for each structural member typology of the critical area of the structure, as identified by preliminary analysis if performed, and at least one sample per floor elsewhere;

b) original test reports for steel bars are available and visual identification confirms the information.

(4) For KLM2 and KLM3, if results of testing reveal the presence of steel of different grades, then the investigation should be expanded to identify in which structural members each different grade has been placed; conditions a) or b) in (2) or (3) should be met for each steel grade separately.

(6) If it can be proven that it is not possible to reliably replace the bars, non-destructive tests (hardness test) may be performed instead. Non-destructive (hardness) test may always be used to identify where each grade has been used according to (4).

Conclusions

Basis of design and data for assessment

CONCLUSIONS

Basis of design

- Assessment is carried out with respect to Near Collapse LS
- Return period is linked to target reliability $\beta_{\mathrm{t,LS,CC}}$ and default is 1600 years (<2475 years)
- Displacement-based approach is the reference method
  - Force-based approach permitted in low & moderate, with low q
- Mean values used for material properties both in model and verifications
- No mix of uncalibrated confidence factor and γ_{c}, γ_{s} but reliability-based $\gamma_{\mathrm{Rd,KL}}$

Data for assessment

- Three distinct knowledge levels for Geometry, Details and Materials
- After information on Geometry is collected, preliminary analysis can be used to direct inspections on Details and tests on Materials
- Mix of destructive and non-destructive techniques allowed
  - Details: preference for DT
  - Materials: larger proportion of NDT
- Total number of surveyed sections, inspected details and material tested increases less than linearly with structure's size