POLITECHNIKA LUBELSKA

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Failures of concrete and masonry structures

Identification of damage and causes

Lublin 2016

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Identification of damage and causes

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Introduction

Construction works (buildings and other civil engineering works) failures have been as old as the art of construction. In the Hammurabi’s Code (2200 B.C.), the situation of a house failure was already foreseen: “If a builder build a house for a man and do not make its construction firm and the house which he has built collapse and cause the death of the owner of the house – that builder shall be put to death” and “If it destroyed property, he shall restore whatever it destroyed, and because he did not make the house which he built firm and it collapsed, he shall rebuild the house which collapsed at his own expense” [17].

The first well-known structural failure was the collapse of the Colossus of Rhodes, one of the Seven Wonders of the Ancient World (227 B.C.). This collapse was caused by an earthquake.

The worst building failures in history were also related to force majeure events, i.e. natural (geological or weather) phenomena or violent events (wars, terrorist attacks, gas explosions and fires). The failures mentioned above, the reasons of which are obvious, are not the main focus of this book however. The authors rather intended to show the activities or procedures which should be undertaken when damage is visible, but its reasons are to be found.

The degree of damage and material losses, the number of dead or injured people are different for each type of failure. That is why any construction work failures should not be identified with catastrophes only. For the same reason, in this book ’failure’ is rather construed according to the definition by the American Society of Civil Engineers (ASCE), i.e.: “Failure is an unacceptable difference between expected and observed performance” [17]. This means that excessive, unacceptable deformations or cracks are a failure, too.

Regardless of failure extent, three important questions must always be answered:

- What was the reason(s)?
- What is the range of destruction?
- Is it safe to use the building or structure after failure, and what repairs should be undertaken?

Answers should be based on a detailed inspection of the damaged structure. A very important issue is the personal experience of the expert assessing the structure and analysis of similar cases. The diagnosis should be based not only on the technical background but, in the more difficult cases, also on scientific assumptions and research.

This book addresses two initial questions; the repairs are not considered. The authors’ intent was to systematise information about failures and their symptoms, and to provide updated information about building structures diagnostics. For purposes of this book, diagnostics shall mean inspections leading to identification of failure and causes of damage.

Authorship contribution:

- Anna Halicka – conception of the book, its arrangement, chapters 1, 2, 3.1-3.4, 3.5.1–3.5.3 and 4.
- Marek Grabias – chapters 3.5.4–3.5.6.

The first co-author shares her professional experience gathered during years of working as an expert; the second co-author presents the diagnostic methods.

The text is illustrated with sketches and photos depicting real-life examples of damage the first co-author and her expert partners encountered in their professional work Therefore, the authors would like to thank the following individuals for making their drawings available:

E. Błazik-Borowa, associate prof. – author of the FEM calculations of vaults (Fig. 99),

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J. Fronczyk, Eng. – conducting the tank inspection (sketches in Fig. 84, 85),

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J. Podgórski, associate prof. – specialist of dynamic measures (charts in Fig. 14),

J. Szerafin, PhD – author of dwelling house inspections (sketches in Fig. 18, 22).

All other sketches were drawn by Anna Halicka (the sketches of crack patterns in concrete and masonry structures are based on pictures taken from [32]).

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P. Borowski, MSc – Fig. 49,

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All other photos were taken by Anna Halicka.

All careful readers are very welcome to send any their comments or queries to: a.halicka @pollub.pl.

Anna Halicka
Marek Grabias

1. Construction works failures

1.1. The essence and main reasons for construction work failures

Any construction work failure always affects people: those who lose their lives, those who lose people close to their hearts, their health, possessions or money. The degree of destruction may vary from damage of secondary members, which has a small range and exerts a minor influence on the stability and bearing capacity of the entire structure or building, to complete collapse or demolition. The only consequence of small damage may be material costs that must be spent on repair, which is solely the owner's problem. Collapses on the other hand, may involve casualties – killed or injured people. Such collapses have been described in newspapers or on the Internet.

The worst building failures in history were caused, directly or indirectly, by force majeure or violent events.

Force majeure encompasses natural, i.e. geological or weather phenomena. Earthquakes have destroyed entire towns or quarters, like San Francisco in 1906, Tangshan, China in 1976, Mexico City in 1985, and Haiti in 2010. Earthquakes generating tsunamis ruined, e.g. Messina harbour in 1908 and large areas of Japan in 2011. Mine subsidence may also be classified as force majeure. Catastrophes caused by a hurricane destroyed entire towns, e.g. Katrina in New Orleans in 2005. The abovementioned events are, of course, only examples of many similar events taking place every year.

Violent events include in particular:

- wars,
- terrorist attacks, like the collapse of two World Trade Center towers in 2011,
- gas explosions, like the explosion in a 22-storey Ronan Point tower block in London in 1968 (4 people killed), a bank in Warsaw in 1979 (49 killed), Hotel New World in Singapore's Little India in 1986 (33 killed) and an 11-storey dwelling house in Gdańsk in 1995 (23 killed),
- fires, e.g. the Great Fire of Rome in the year 64, a three-day long Great Chicago Fire in 1871, fire in the Church of the Company of Jesus in Santiago in 1863 (more than 2,000 people died) and more recent events, like the fire at Kings Cross tube station on the London Underground in 1987 or the fire in the sports hall of Gdańsk Shipyard in 1994.

In this book however, failures emerging due to force majeure and violent events are not the main focus. The authors are primarily interested in another group of failures – those occurring mainly due to human error. Usually there is no specific reason, but a coincidence or a sequence of causes that lead to failure:

wrong decision(s) and/or wrong action(s) of humans, and abnormal operation conditions. Therefore, the problem of failure causes is of holistic nature [35].

In this group of failures, the causes may be divided into primary reasons (defects in design or execution) and operational reasons.

Primary reasons include:

- errors in the original (basic) documents: during the design process, the designer relies on codes and technical specifications for building materials and products – wrong or unchecked information contained therein, as well as the lack of general knowledge concerning the analysed problems may mislead the designer and result in wrong design solutions,
- design errors: neglecting some loads in structural calculations (e.g. thermal or dynamic loads, accidental loads, shrinkage of concrete), inaccurate determination of subsoil parameters and the level of underground water, choosing an inadequate structural model or improper solutions for connecting the elements, errors in calculations or in drawings,
- execution faults: use of structural materials of inferior quality (e.g. concrete of lower strength, soaked timber), poor quality workmanship (e.g. insufficiently compacted concrete mix, rebar displacement, inattentive bonding of bricks in brick masonry, bad quality welding), unsafe works (e.g. improper fixing of the formwork or scaffolding).

Failures during operation may occur due to incorrect operation (e.g. adding new floors or increasing loads without strengthening the structure), accidental load (e.g. washing out of the soil from beneath the foundation, abnormally heavy snow) and environmental factors whose impact is aggravated by improper maintenance.

1.2. History of construction works catastrophes and their main reasons

Until the 19th century, failures due to human error mainly reported in historical sources included collapsed churches, e.g. St. Peter's Church in Riga in 1666 (8 persons buried under the rubble), nave of the Dom Church in Utrecht in 1674, a wall of the Collegiate Church of St. Mary Magdalene in Poznań in 1777 (during the construction works), Chichester Cathedral Spire in 1861 or St. Mark's Campanile in Venice in 1902.

From the 19th to 21st centuries, different types of building structures were destroyed due to different causes. The largest collapses in history are compiled below, based on the list of the largest structural failures and collapses [14] and following other Internet sources (bridges, tunnels and broadcast towers were omitted). The numbers of fatalities are given, because this may to some extent reflect the catastrophe range, although in some cases deaths were evaded, yet the range of destruction and financial losses remained huge.

Failures of buildings of different types and at different destinations may be illustrated by: the collapse of the Broadway Central Hotel in New York (1973, 4 killed), Sampoong Department Store in Seoul, South Korea (1995, 502 killed, reason – overload and reduced dimensions of columns in relation to the originally designed), Highland Towers apartment building in Kuala Lumpur, Malaysia (1993, 48 killed, reason – weak saturated subsoil), Palace II residential tower in Rio de Janeiro (1998, 8 killed, reason – poor quality materials), Versailles wedding hall in Jerusalem (2001, 23 killed, reason – insufficient bearing capacity of the floor), a 21-storey building in Lagos, Nigeria (2006, collapse of the building weakened by a fire that took place a month earlier), Historical Archive of the City of Cologne in Germany (2009, 2 killed, failure due to construction of a subway tunnel), Lotus Riverside Block 7 in Shanghai, China (2009, 1 killed, reason – excavations of earth near the building), 45J Ma Tau Wai Road in Hong Kong (2010, 4 killed, reason – destruction of one column), Lalita Park in New Delhi (2010, 67 killed, reason – adding new floors on the existing ones), Vieira Fazenda office block in Praça Floriano, Rio de Janeiro (2012, progressive collapses of three 22-storey office buildings; 17 killed), a building in Philadelphia, Pennsylvania (2013, a 4-storey building undergoing demolition collapsed onto the neighbouring building, 6 fatalities), a 5-storey building in Mumbai (2013, 74 killed, reason – adding a new floor), a 22-storey residential building in Medellin, Colombia (2013, reason – bad construction and questionable materials) and Synagogue Church in Ikotun Egbe Lagos, Nigeria (2014, 115 killed, reason – design errors).

The deadliest structural failures of factories and production facilities are the following: fall of the Pemberton Mill in Lawrence, Massachusetts (1860, more than 100 killed, reason – overloading with heavy machinery), Rana Plaza in Savar Dhaka in Bangladesh (2013, 1127 killed, reason – overloading with machinery causing vibrations and addition of 4 extra floors).

In halls and hangars used for various purposes, it was mainly the roofs that collapsed. This very often happened due to load exerted by heavy snow on light roofs, e.g. roof of Kugaiza Cinema, Tokamachi, Niigata, Japan in 1938, roof of the Sophia Gardens Pavilion concert hall in Cardiff, Wales in 1982, roof of an auditorium of Mauna Ocean Resort in Gyeongjuin, South Korea in 2014 (10 killed), roof of Katowice Trade Hall in Chorzów, Poland in 2006 (heavy snow as well as design errors, 65 killed). Roofs overloading was also caused by: parked cars (the Station Square in Burnaby, British Columbia in 1988) and stored building materials (Maxima superstore in Riga, Latvia in 2013, 54 killed). Other reasons included design errors, e.g. concrete roof-top parking deck of Algo Centre Mall in Elli Lake, Ontario (2012, 2 killed), poor quality of structural members e.g. roof of Terminal 2E at the Charles de Gaulle Airport in Paris (2004, 4 killed) or poor maintenance, e.g. roof of Marja store in Tallin, Estonia (1994, 5 killed) or roof of the Baikonur Cosmodrome in Kazakhstan (2002).

The collapses often involved stadiums and sports halls. These were mainly roof collapses due to snowfall or heavy storm, e.g. Hartford Civic Center in Connecticut (1978), space-frame roof of Kemper Arena in Kansas City, Missouri (1979), Bad Reichenhall Ice Rink in Germany (2006, 5 killed), as well as poor design or workmanship, e.g. glass dome of Knick-Ei in Halstenbek, Germany (1997 and 1998), roof of Transvaal Water Park in Moscow (2004, 28 killed), fiberglass fabric roof of Hubert H. Humphrey Metrodome in Minneapolis, Minnesota (2010), stadium roof in Enschede, the Netherlands (2011, 2 killed), Sultan Mizan Zainal Abidin Stadium in Terengganu, Malaysia (2009), roof of the indoor water park at the Thumper Pond Resort in Ottrertal, Minnesota (in 2015), Hu Fa Kuang Sports Centre at the City University of Hong Kong (in 2016). Roofs were not the only members that collapsed in the stadiums, e.g. in 1902 a wooden terracing in Ibrox Park in Glasgow (26 lookers-on killed) and in 1992 terrace of Armand Césari Stadium in Bastia, France (18 killed).

The collapsed engineering structures (silos, tanks, etc.) are represented by: Transcona Grain Elevator in Canada consisting of 90 reinforced concrete chambers (fall in 1913 due to excessive and non-uniform settlement of the subsoil), molasses storage tank in Boston, Massachusetts (burst in 1919 killing 21 people); cooling tower in Bouchain Power Plant in France in 1979, and in Turów Plant in Bogatynia, Poland in 1987 (collapses due to workmanship errors).

Sometimes failures occurred already during the construction stage. Structures that collapsed under construction include in particular: Willow Island Cooling Tower in West Virginia (in 1978, reason – scaffolding bolted onto not cured concrete, 51 killed); chimney of the Matla Power Station in Mpumalanga, South Africa (1982, 4 killed), L'Ambiance Plaza in Bridgeport, Connecticut erected using the lift slab construction technique (1987, 28 killed), Korba chimney in Chhattisgarh, India (2009, 45 killed), Dar es Salaam apartment building in Tanzania (2013, reason – poor design, 36 killed); commercial building in Sao Paulo, Brazil (2013, 6 killed), Thane building in Mumbai, India (2013, reason – poor quality design and construction, 45 killed), a building in Lagos, Nigeria (2016).

1.3. Failure aspect in design, execution and maintenance

1.3.1. General information

Construction works failures and damage may be of different range: from destruction of non-structural elements, secondary structural members, primary structural members to partial or complete collapse of construction work. The consequences may also vary: from financial losses, the rebuilding cost, to injured or killed people.

In order to avoid failures or limit their consequences, special requirements should be met. These involve:

- avoiding, eliminating or reducing hazards to which the structure can be exposed – this should be considered at the design stage and observed by users during operation,
- appropriate design procedures and detailing, especially selecting a safe structural form and choosing suitable materials,
- obeying rules of proper maintenance and performing the required repairs during use.

According to Eurocode [S6], a safe structural form ensures that the construction work has low sensitivity to the hazards considered and will not be damaged by events such as explosion, impact and the consequences of human errors, to an extent disproportionate to the original cause. What is more, the structural systems that may collapse without warning should be avoided.

A safe construction work is a reliable one. Reliability is expressed mainly in probabilistic terms. The present-day rules of designing construction works include the reliability aspect, specified in detail in the Eurocode [S6]. Nevertheless, it should be remembered that the reliability of structures is achieved not only by good design but also by appropriate execution, quality management and maintenance measures.

Determination and application of relevant control procedures for design, production, execution and use, facilitates meeting of the requirements specified above.

1.3.2. Design with regard to reliability

The demanded level of reliability may be achieved during design by using the ultimate states method or reliability analysis. Designing based on ultimate limit state consists in ensuring that the specified criteria (ultimate limit states and serviceability limit states) are fulfilled for structural members and for the entire structure. Designing based on the reliability analysis, consists in ensuring that the measure of reliability (reliability index) is of demanded value.

The reliability index β is defined as the inverse function of the failure probability P_f :

$$\beta = \Phi^{-1}(P_f) = \frac{\mu_R - \mu_E}{\sqrt{\sigma_R^2 + \sigma_E^2}} \tag{1}$$

where

μ, σ – mean value and standard deviation of R (resistance, bearing capacity) and E (effect of loads).

Reliability assessment procedures are not the subject of this book, therefore only general information relating to failures shall be given below.

Eurocode [S6], specifies three different levels of reliability: RC1, RC2 and RC3. The choice of the level should take into account any relevant factors, including: the possible cause and/or mode of failure, the possible consequences of failure in terms of risk to life, injury, potential economic losses, the public's aversion to failure and the expense and procedures necessary to reduce the risk of failure.

The reliability level corresponds to the range of failure consequences. To that end the consequences classes were introduced in Eurocode [S6], that may be associated with the reliability classes:

- CC3 class: high consequence for loss of human life, or economic, social or environmental consequences very great (grandstands, public buildings where consequences of failure are high, e.g. concert hall),
- CC2 class: medium consequence for loss of human life, economic, social or environmental consequences considerable (residential and public buildings where consequences of failure are medium (e.g. an office building),
- CC1 class: low consequence for loss of human life, and economic, social or environmental consequences small or negligible (e.g. agricultural buildings where people do not normally enter, storage buildings, greenhouses).

Attribution of the reliability class to a structure results in assigning the appropriate level of reliability and subsequently, choosing the appropriate values of partial factors, if the ultimate states method is used or appropriate values of reliability index, if the reliability analysis is carried out.

1.3.3. Durability of construction works

The criteria mentioned in chapter 1.3.1 should be met and the reliability index must be of demanded value in the assumed 'design working time'. According to Eurocode definition, it is the assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary. With reference to the subject of this book, it can be said that during the design working time no failure occurs.

The design working time is related to durability – deterioration over the design working life should not impair the performance of the structure below that intended level. The level of maintenance must correspond to its environmental conditions. In order to achieve an adequately durable structure, it should be designed with regard to the expected environmental and subsoil conditions, and especially, the protective measures should be planned. The workmanship must be of good quality. The structure must be used in accordance with the intended or foreseeable manner, the intended maintenance must be performed. The controls during all stages (design, execution and use) must be performed.

In Eurocode [S6], structures are divided into categories related to design working life and indicative periods of use are recommended:

- Category 1 (temporary structures) – 10 years,
- Category 2 (replaceable structural parts, e.g. gantry girders, bearings) – 10 to 25 years,
- Category 3 (agricultural and similar structures) – 15 to 30 years,
- Category 4 (building structures and other common structures) – 50 years,
- Category 5 (monumental building structures, bridges, and other civil engineering structures) – 100 years.

1.3.4. Control as a factor influencing reliability

As mentioned above, the control procedures at each stage of construction work's service life help avoid failures or limit their consequences. Therefore, Eurocode [S6] introduces design supervision types and levels of inspection during execution. The higher the consequences class and the reliability class, the higher the level of design supervision and the level of inspection.

Design supervision consists in checking the calculations, drawings and specifications. Three possible design supervision levels are linked to the reliability class. They should be implemented through appropriate quality management measures. Design Supervision Levels are the following:

- extended supervision DSL3, relating to RC3: third party checking – checking performed by an organisation different from that which has prepared the design,
- normal supervision DSL2, relating to RC2: checking by persons different from those originally responsible and in accordance with procedure of the organisation,
- normal supervision DSL1, relating to RC1: self-checking – checking performed by the person who prepared the design.

Execution of structures requires inspection focused on avoiding the failures due to poor workmanship and materials used. Three inspection levels are introduced:

- extended inspection IL3, relating to RC3: third party inspection,
- normal inspection IL2, relating to RC2: inspection in accordance with the procedures of the organisation,
- normal inspection ILl, relating to RC1: self-inspection.

1.4. Construction works failures as the subject of scientific research

In order to gather knowledge of construction works failures, a scientific and technological discipline called *building pathology* was introduced [12, 38]. According to [12], this term was introduced by CIB (International Council for

Research and Innovation) in 1993. Building Pathology is defined as the systematic study or treatment of building defects, their causes, their consequences and their remedies. Such terminology is metaphorical and relates to an ’illness’ in medical terms. Just like in medicine, this term covers: assessment of the ’diseased‘ condition, determining its aetiology and proposing remedies. The approach may be described as holistic – the failure is a result of various associated mechanisms.

In building pathology, analyses of particular cases are very important. They provide the knowledge of failure causes. One can learn to which agents designers and executors should pay special attention, and what should be avoided. They also inspire the researchers to find new solutions not likely to be affected by human error. Works by [3, 4, 8, 15, 19, 27, 31, 34, 35, 42], may serve as just a few examples of rich bibliography of case study analyses. Sometimes such analyses were performed even several years after the failure [27].

The basics of construction failures and diagnostics were given, among others, in books by [6, 17, 38], as well as in the papers by, e.g. [4, 39, 35, 45]. Statistical analysis of failure causes may be found in e.g. [1, 2].

Reports and analysis of construction works failures are found in the journals devoted to safety issues and failures – “Safety Science”, “Engineering Failure Analysis”, “Case Studies in Engineering Failure Analysis”.

Conferences on structural failures have been organized for many years now. In 2016, the Seventh International Conference on Engineering Failure Analysis was held in Leipzig. In 2004, International Conference on Structural and Foundation Failures took place in Singapore. In 2017, the 28th Structural Failures conference will be organized in Międzyzdroje, Poland, and the 19th International Conference of Structures and Failures will be held in Paris.

Diagnostics of construction works, especially after-failure diagnostics, is also based on scientific findings. The contemporary increasingly advanced diagnostic techniques, based on scientific findings are involved. A lot of scientific research has been focused on the non-destructive methods of structure investigations [18, 21, 26]. The most useful are described in chapter 3.5.

2. The essence of construction works diagnostics

2.1. The focus of construction works diagnostics

Diagnostics (assessment) of buildings and other structures includes all activities focused on testing, identifying damage and evaluating technical condition and performance capability, as well as predicting future performance of examined structures. Diagnostics should be performed by experienced civil engineers with the use of diagnostic equipment.

Structures and buildings are diagnosed in the following situations:

- in the case of structural failure or damage (not only during operation but sometimes during the erection stage), in order to determine the cause of failure and to identify the safety level of the structure after failure,
- assessment of the level of structural deterioration due to time-dependent actions, especially caused by corrosion or fatigue,
- in the case of anticipated change of use (e.g. an apartment building expected to be changed into a library) or erection of additional storeys of the building or its parts, focused on checking the capability of bearing the increased or otherwise applied load,
- to calculate the price of the building or other structure put up for sale or intended to be insured,
- reliability check (e.g. for earthquakes, increased traffic on a bridge) as required by the authorities or insurance companies,
- periodic inspection of buildings and structures under the national law (e.g. in Poland, inspections of different scope should be performed every year and every five years), in order to check for any damage posing hazard to the safety of the building or structure in question.

In the above description and later chapters, the following terms are used in accordance with ISO 13822 [S12]):

- damage – unfavorable change in the condition of a structure that can affect structural performance,
- deterioration – process that adversely affects the structural performance, including reliability over time due to: naturally occurring chemical, physical or biological actions, repeated action such as those causing fatigue, normal or severe environmental influences, wear due to use, or improper operation and maintenance of the structure,
- structural performance – qualitative or quantitative representation of the behaviour of a structure (e.g. load bearing capacity, stiffness) in terms of its safety and serviceability.

2.2. Diagnostics in the case of construction work failure

From the point of view of a civil engineer, each failure is a challenge. As mentioned in the Introduction, an expert must answer the questions concerning failure reasons, condition of the structure and its safety after failure. Work of an expert may be compared to detective work – on the basis of the facts (damage), investigations and tests, as well as interviews with persons involved, the real failure reason must be found.

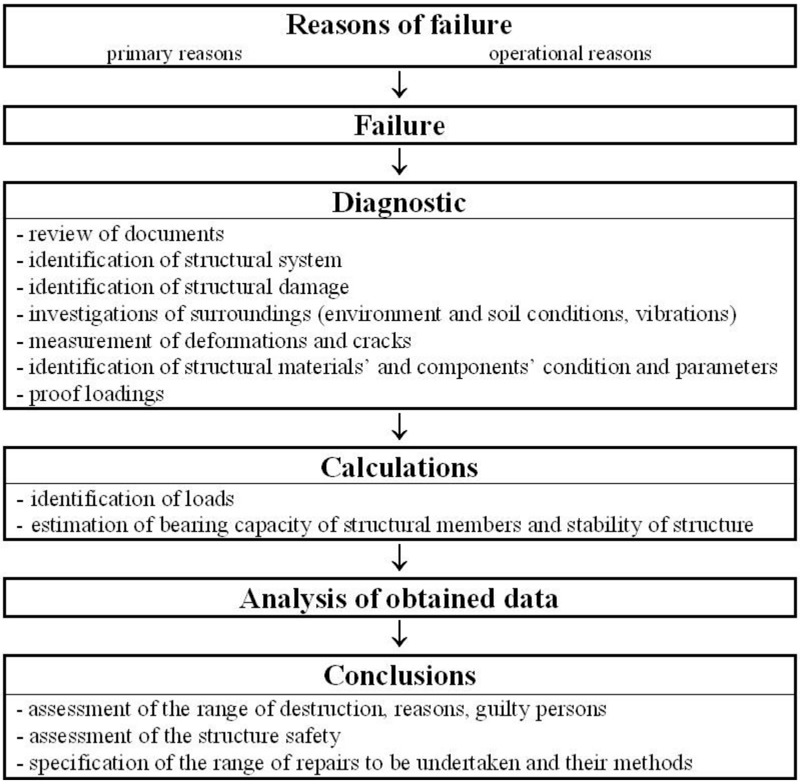


Fig. 1. Sequence of affairs and activities relating construction work failure

The process of finding answers to the above questions should consist of all or selected activities shown in Fig. 1. It begins with diagnostics, including: review of documents, identification of the structural system and structural damage, investigation of the surroundings (environmental and subsoil conditions, presence of vibration and its parameters), investigation of deformations and cracks, and identification of structural materials' and components' condition and parameters. Sometimes proof loading is necessary. The next step are structural calculations using identified loads and parameters of examined materials'. After the analysis of diagnostic and calculation results, the conclusions are formulated concerning the range of destruction, failure reasons and sometimes, especially in forensic opinions – responsible persons.

In the end, structural safety is assessed and the range of necessary repairs, as well as relevant repair methods are specified. It should be emphasised that one can only select the proper repair method warranting safety and reliability of the construction work in the future, if actual reasons of the failure are found and eliminated.

The activities specified above are described in greater detail in chapter 3. They are illustrated by photographs and by examples taken from the practice.

2.3. General assessment of existing structures in accordance with ISO 13822

Flowchart for the general assessment of existing structures set out in ISO 13822 [S12] is quoted in Fig. 2.

An expert enters the site and proceeds with the assessment of the existing structure at the request of its owner, the authorities, insurance company, etc.

The first step before commencing the assessment is to identify the objective of the assessment in terms of the required structural performance in the future – the focus of the assessment should be determined by the owner (possible focuses are specified in chapter 2.1). The second step before the assessment is to identify scenarios related to structural conditions or actions to identify possible critical situations. The scenarios are characterized by processes or actions reported in the past (being the cause of damage) or anticipated in the future.

The assessment is conducted in two stages: a preliminary assessment and, if necessary, a detailed assessment.

The preliminary assessment involves: the study of documents and other evidence, a preliminary inspection and check. As a result of these activities, ISO 13822 [S12] specified two possible solutions: decisions on taking immediate actions to reduce the danger with respect to public safety or presenting recommendations for detailed assessment. In practice, also a third way is possible – right after the preliminary inspection, the expert may conclude that the objective of the inspection formulated by the client has been achieved.

The detailed assessment involves: detailed documentary search and review, detailed inspection and material testing, determination of actions and loads, structural analysis, verification of structural safety and serviceability on the basis of the reliability theory. Detailed assessment is provided in chapter 3.

Sometimes in order to assess changes in the condition of the structure over a longer period of time, it is necessary to repeat or extend the scope of inspections and assessment.

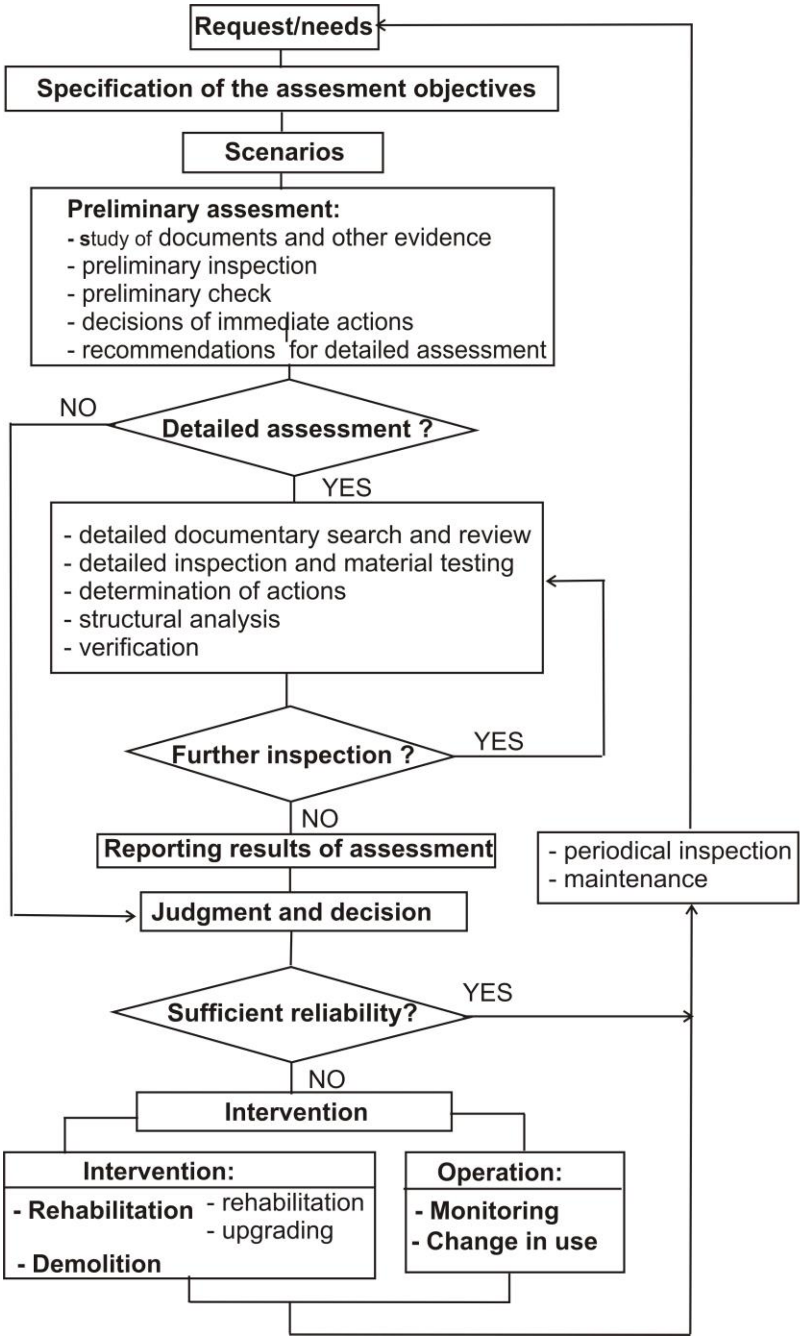


Fig. 2. Flowchart for the general assessment of existing structures in accordance with ISO 13822 [S12]

On the basis of the analysis of test results and the structural analysis, the report should indicate further activities.

If the structural safety or serviceability is shown to be inadequate:

- the repair, rehabilitation or upgrading may be recommended in order to ensure safe structure performance during the remaining working life; the report may give detailed solutions for the repair, rehabilitation or upgrading or only outline the general concepts, leaving the detailed solution for separate design (it depends on the contract with the owner),
- demolition may be recommended, if the repair, rehabilitation or upgrading is impossible or too expensive.

In the above paragraphs, in accordance with ISO 13822 [S12], 'rehabilitation' means work required to repair and possibly upgrade an existing structure, 'repair' – improving the condition of a structure by restoring or replacing existing components that have been damaged, and 'upgrading' – modifications of an existing structure to improve its structural performance (the term 'strengthening' may also be used).

If a structure is in a good condition, two methods are possible:

- if safety level for the remaining working life is acceptable, only maintenance (routine interventions to preserve the appropriate structural performance) and periodical inspections are recommended,
- if bearing capacity is not sufficient, the change in use may be recommended, e.g. reduction of loads (provided however that this is accepted by the user).

3. Detailed assessment – activities to be undertaken and their characteristics

3.1. Review of documents

The expert should carefully analyze all documents connected with the assessed structure. The most important are the preliminary documents:

- structural calculations, analyses and drawings coming from design documentation,
- subsoil conditions and groundwater level value adopted by the designer,
- specifications of material and product properties,
- regulations and by-laws, codes of practice and standards that were used for constructing the analyzed structure.

Documents drawn up in the erection period (construction records, sketches of modifications in the designed materials and structural solutions) may show the real characteristics of a structure with alterations introduced with respect to design solutions.

Documents drawn up in the service period, such as periodic inspection and maintenance reports, as well as previous assessment reports, provide information about behaviour of the structure throughout its use, especially about the deterioration rate and incidence of accidental situations and loads.

It should be emphasized, that the above-mentioned documents may not be available. This may especially be the case with heritage structures, as the design documents usually do not exist at all or the owner has no documents from the erection stage.

In the case of heritage structures, in accordance with ISO 13822 [S12], an expert should use the historical report and heritage report prepared before the assessment as the basis for assessing the structure.

A historical report is usually prepared by cultural resource specialists (such as historians or archaeologists). This report presents summary information derived from heritage documents. Where no such specialist is employed, the assessment report prepared by the engineer should include historical information. The historical report should identify the nature of the original construction, all subsequent alterations and any significant events that may have influenced the emergence of any structural damage and deterioration. It should be emphasized, that historical reports are not produced strictly for structural purposes – therefore, they generally provide useful information, but as they can also be misleading and contain information difficult to interpret, they need to be carefully evaluated.

Heritage records must be produced by specialists (mostly architects specializing in heritage buildings and structures). Currently, heritage records have been made to describe the existing and earlier conditions of heritage structures. This includes drawings presenting arrangement of rooms, as well as arrangement of structural members (walls, vaults, domes, ceiling beams) and identification of materials.

At the stage of investigating the documents, the interviews with persons involved may be added. The owners, responsible persons and users may provide information that is not reported in documents.

3.2. Identification of the structural system and structural damage

The structural system and damage of the structure is identified by visual observation with simple tools.

Identification of the structural system is based on determination of the structural materials (concrete, masonry, steel or timber). Next, type of structure (e.g. frame structure or wall structure) and type of cover (dome, vault, floor: beam-framed floor, rib-and-slab floor, reinforced concrete slab etc.) should be determined.

Fig. 3. The photographs documenting damage to the rib-and-slab floor in a two-chamber water tank, drawn in Fig. 4; arrows indicate the cracks in the slab

Then, visible damage and defects should be identified. The following aspects should be considered:

- deformations,
- surface characteristics: existence and condition of plasters and coatings; areas of dampness, salinity, mould or fungi,
- decrease in member cross-sectional area and loss of mortar between bricks,
- cracks and leakage,

- dropping off or spalling of concrete cover in reinforced members,
- corrosion of steel and bio-corrosion of timber members.

Such damage should be described verbally, shown in the sketches and evidenced with photographs. In order to draw attention to the most important damage, e.g. cracks, arrows may be added to the photographs. Examples of such documentation are shown in the figures from 3 to 8.

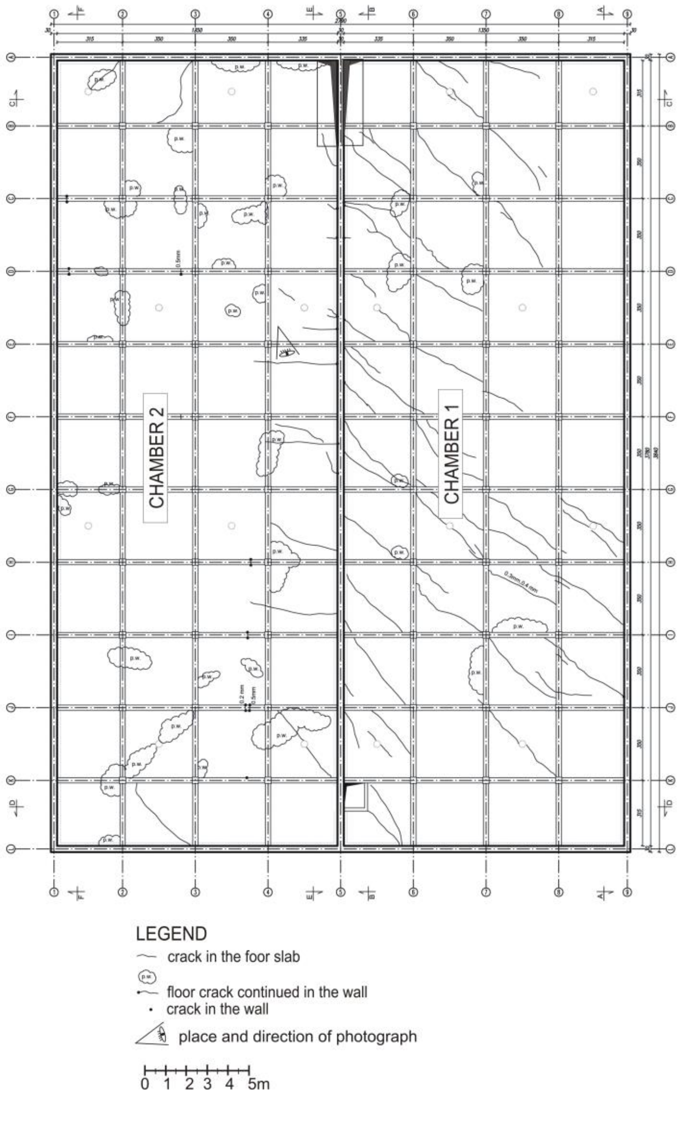


Fig. 4. Documentation of damage to the rib-and-slab floor in a two-chamber water tank, the view looking up

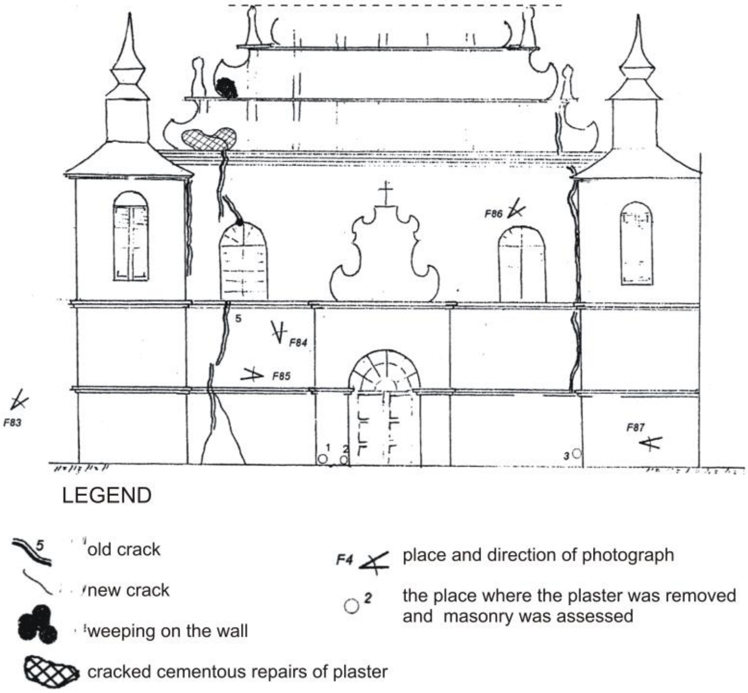


Fig. 5. Sketch of damage in the façade of the Dominican church in Lublin

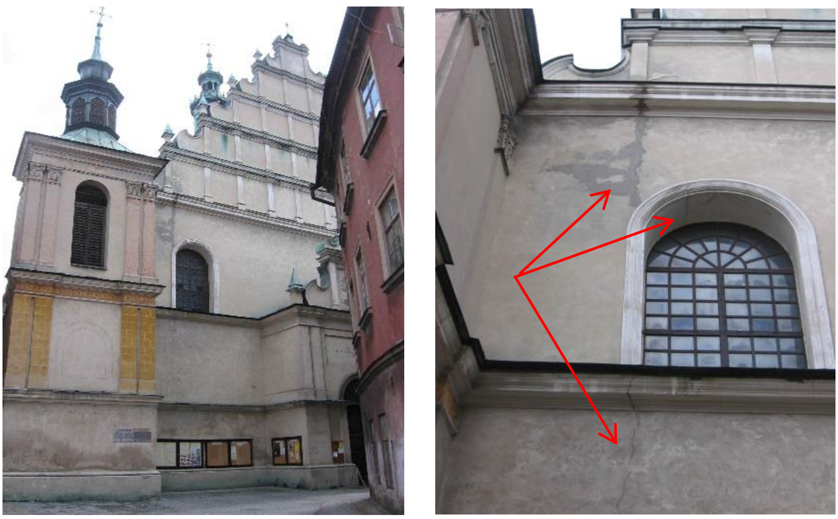


Fig. 6. Photographs of the façade shown in the sketch in Fig. 5: general view and zoomed fragment of the wall, arrows indicate the cracks

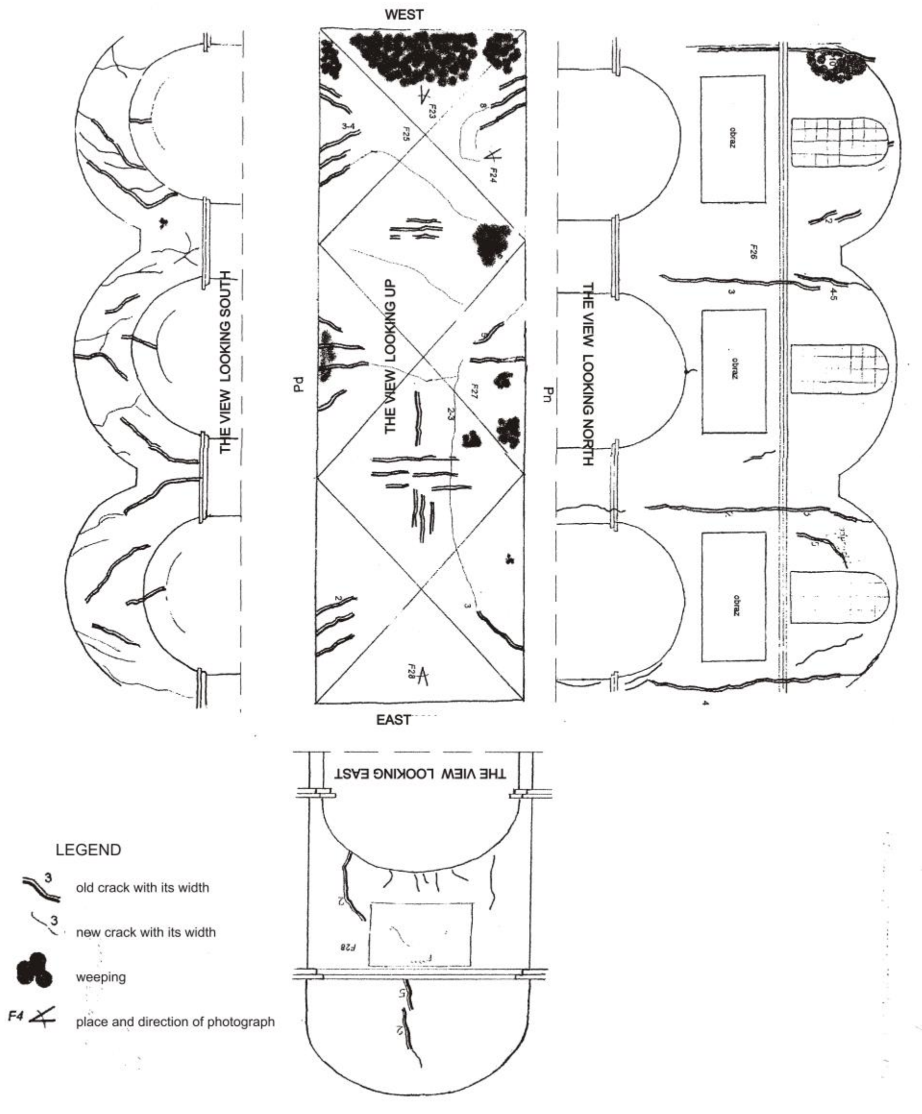


Fig. 7. Documentation of damage in the central aisle of the Dominican church in Lublin: sketch of a view looking up and views looking south, north and east

Fig. 3 and Fig. 4 present the evidence of damage identified in the slab of a two-chamber water tank – examples of photographs are shown in Fig. 3, while the sketch of damage is presented in Fig. 4.

Figures from 5 to 8 present the evidence of damage found in a historical church (examples of sketches and photographs). Fig. 5 and Fig. 6 demonstrate the façade, whereas Fig. 7 and Fig. 8 – one of the church side-aisles.

Fig. 8. Photographs of the cracks and damp area of the vault of the Dominican church in Lublin

3.3. Investigation of surroundings influencing structure performance

3.3.1. Environmental conditions

Environmental conditions influence the long-term behaviour of buildings and structures. Aggressive agents present in the surroundings cause structural deterioration (corrosion). For this reason, the environmental conditions should be carefully considered.

Environmental conditions constitute:

- chemical composition of the air, e.g. acid rain, presence of sulfate ions, chloride ions (e.g. in seaside locations), nitrite ions,
- CO_2 content accelerating the carbonation processes in concrete cover of the reinforcement,
- temperature variations causing freeze/thaw attack,
- humidity and temperature (Fig. 9) – high humidity and high temperature accelerating the corrosion processes.

Aggressive conditions may also result from the contact with aggressive liquids – soil water, sewage or other waste, liquids used in technological processes or waste from such processes, and even solution of chloride-based deicing agents in the snow. They may produce acid reaction. Sulfate, nitrite or magnesium ions, as well as organic compounds may initiate the corrosion process in the internal structure.

Fig. 9. Measurement of air temperature and humidity

Identification of aggressive and corrosion-enhancing agents is very important. It allows to determine the exposure class of a given structure. Classification of structures according to specific exposure classes results in the need to satisfy the design code requirements for this structure (e.g. minimum thickness of reinforcement concrete cover).

3.3.2. Subsoil characteristics

Subsoil conditions or their variations during operation of the structure are often the cause of damage or failure. Therefore, they must be carefully considered.

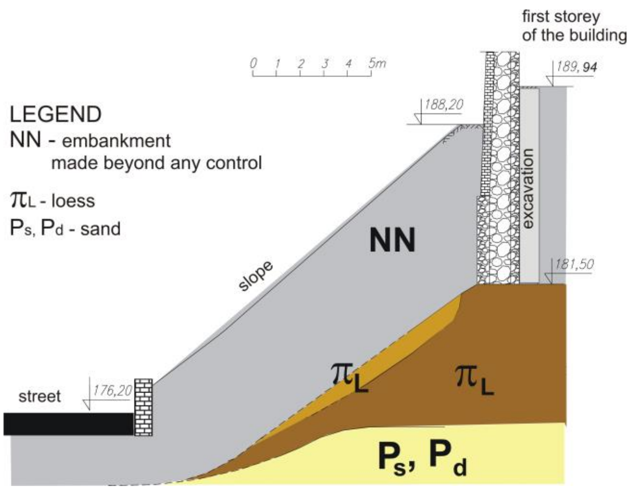


Fig. 10. A fragment of subsoil cross-section under the corner wall of the Dominican monastery and the adjacent slope in the Old Town in Lublin

Firstly, documents coming from the design period should be studied to determine subsoil layers and their parameters, as well as the groundwater level. In the absence of such preliminary documentation or in case of suspicion that the damage was caused by subsoil conditions, new subsoil testing should be performed. The control drilling and probing in the most critical sites allows to determine the subsoil layers characteristics – their thickness and parameters (strength, modulus of deformability), as well as the groundwater level.

Fig. 11. Photographs of the drilling site near the wall of the church shown in Fig. 10: drilling (left), excavated soil (right): embankment material (dark) and loess (fair)

Fig. 12. Examples of test excavations

Based on original and updated data, subsoil cross-sections should be prepared including the foundation levels (depth beneath the ground level). In order to establish the foundation levels, test excavations adjacent to the foundation should be performed and documented. These excavations additionally allow to find the condition of the foundation and of the underground part of the wall or column**.**

Fig. 10 presents a subsoil cross-section based on results of new tests prepared for one of the churches in the Lublin Old Town, while Fig. 11 shows the drilling site adjacent to this church; drilling was executed to obtain the updated soil parameters. Fig. 13 presents a sketch of excavations in the same monastery, and Fig. 12 shows the photo record of sample excavations.

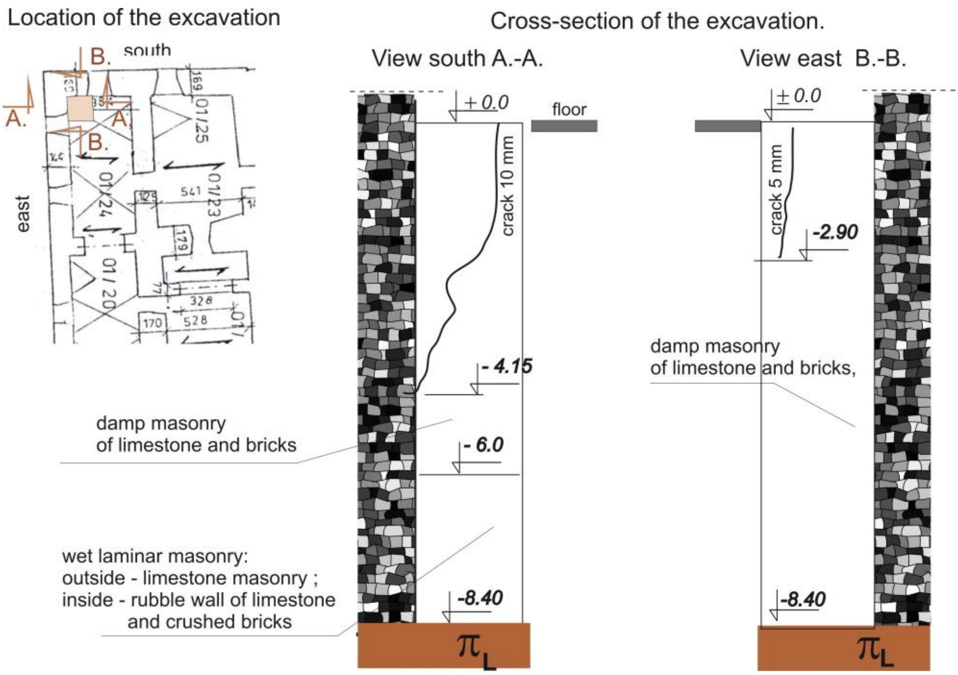


Fig. 13. Excerpt from the documentation of the excavation site

3.3.3. Dynamic effects

The source of dynamic load, dangerous for the structure, may be the passing-by heavy trucks, trams or trains, vibration-generating machines located in a neighbouring factory, or soil compactors. Impulses generated by a pile-driver working in a short distance from the structure may also be the cause of damage.

If dynamic effects are believed to be the cause of structural damage, dynamic response to dynamic load should be tested. These tests are performed by an expert using appropriate measuring instruments – vibration analysis equipment. An example of such equipment – Spider8 amplifier manufactured by Hottinger Baldwin Messtechnik – is shown in Fig. 15. This amplifier allows to save the measured signals from multiple sensors. The most relevant dynamic analyses are performed by sensors called accelerometers used to measure acceleration. Control measurements and results are recorded by an amplifier coupled to a portable computer. Registered signal is then analyzed by special computer programs that allow to determine important dynamic parameters, such as acceleration value and the frequency of vibration (Fig. 14).

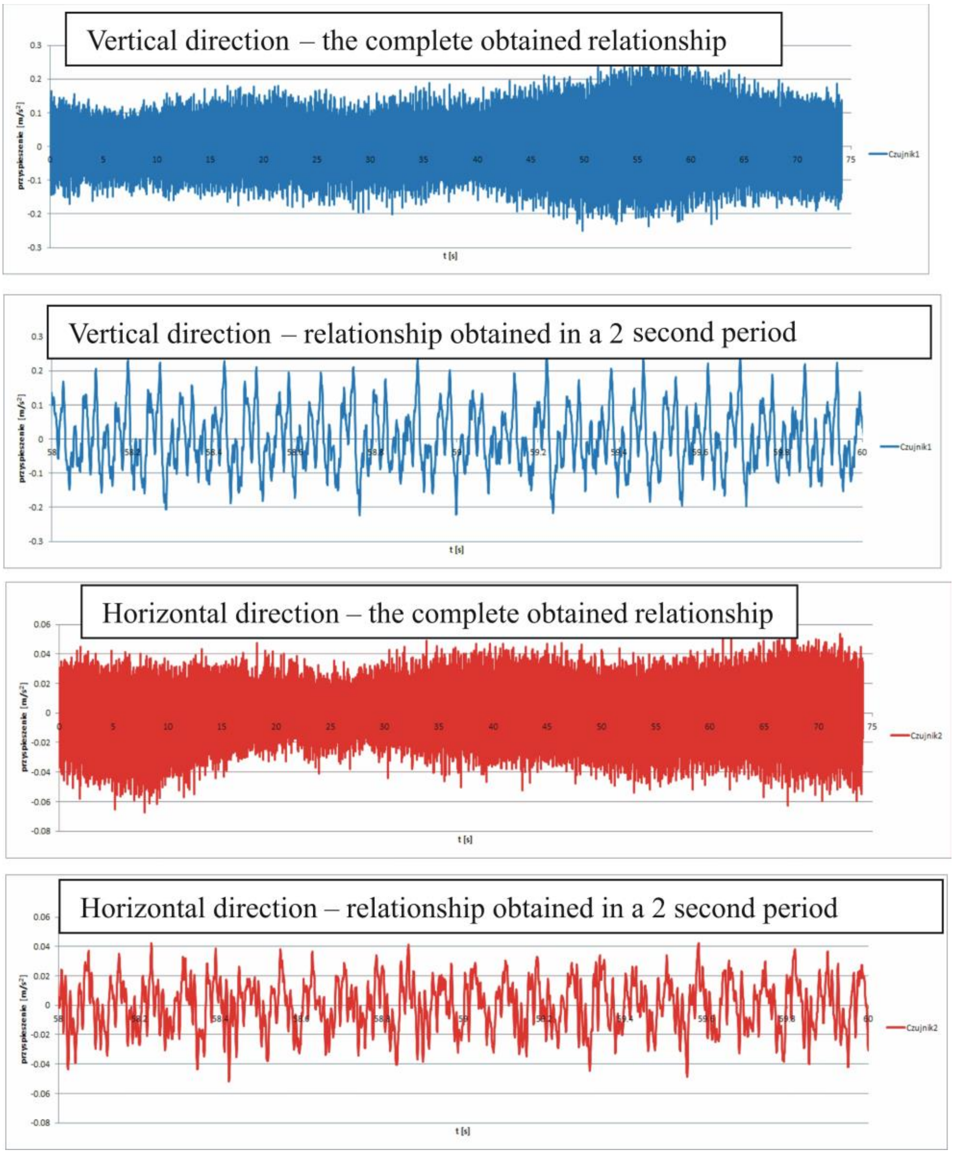


Fig. 14. Examples of 'time-acceleration' relationships obtained in dynamic tests

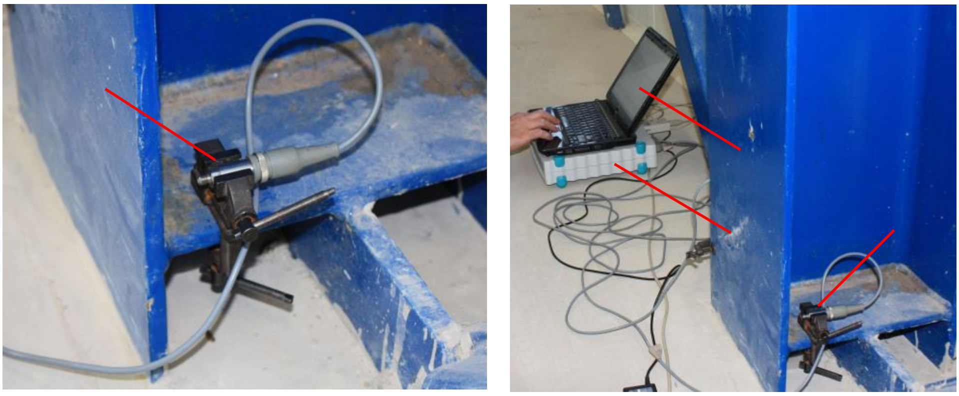


Fig. 15. Equipment for dynamic measurements: 1 – a sensor fixed to an element on a steel frame, 2 – an amplifier, 3 – a laptop computer with diagrams visible on the screen

3.3.4. Seismic and quasi-seismic effects

Structures located in seismic regions are subjected to specific seismic loads. Possible impact of such loads should be taken into account during structure assessment.

A similar situation may take place in mining regions. Here, buildings and structures may be subjected to quasi-seismic loads due to roof collapse in the extraction gallery in the mine.

3.4. Measurement of structural deformations and cracks

3.4.1. Deflection of beams and slabs

Deflection of beams and slabs is a response to load that may be used to assess the load value.

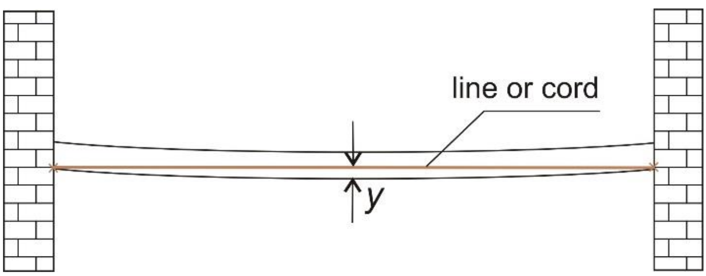


Fig. 16. Simple method of measuring deflection using a line or a stiff cord stretched between two ends of the beam

The simplified *in situ* method may be used provided that the beam can be easily accessed. Ends of the line or of a stiff cord should be fixed to the ends of the beam. The distance from the line to the lowest point of the beam is measured. This may be treated as deflection value (Fig. 16).

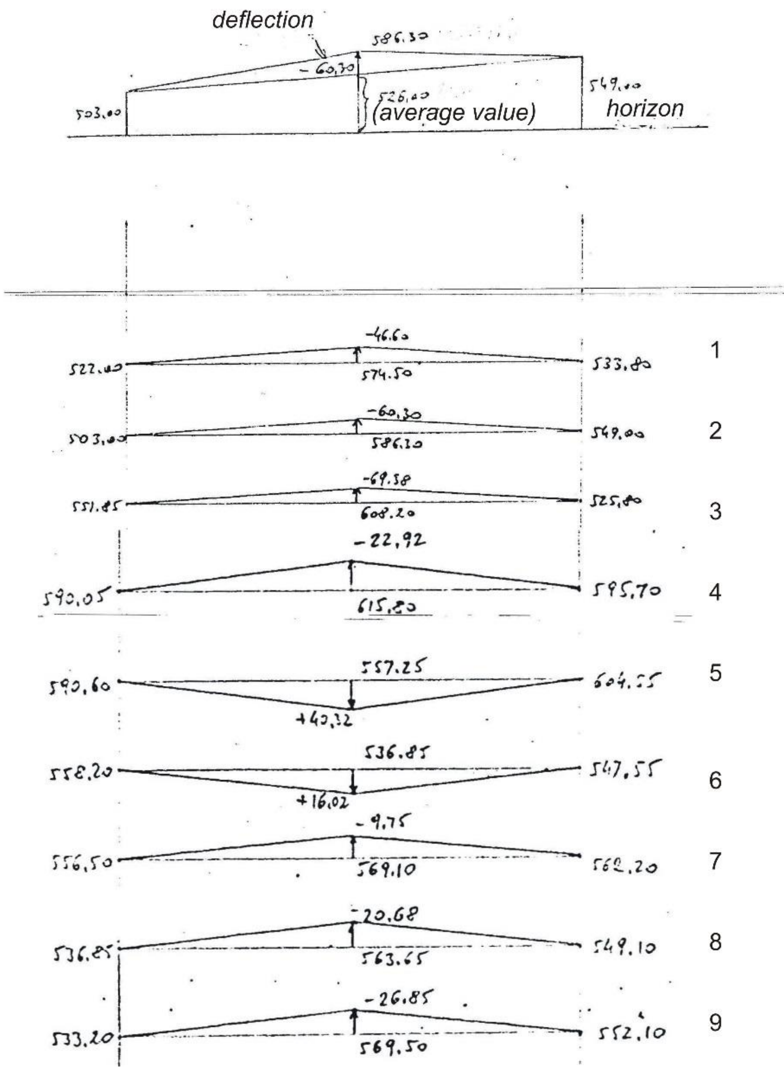


Fig. 17. An example of a surveying report presenting deflection of nine pre-tensioned girders of the roof construction in an industrial building

Surveying methods involving the use of special surveying equipment (e.g. levelling instruments, photogrammetry or 3D scanning) are more accurate. An example of a surveying report presenting deflection of nine pre-tensioned girders, being elements of the roof structure in an industrial building, is demonstrated in Fig. 17. It may be noticed that girders No. 5 and 6 are characterized by downward deflection, whereas the remaining girders are characterized by upward deflection. This may reflect reduction of the prestressing force in these girders. Nevertheless, it should be emphasized that a single measurement cannot serve as sufficient evidence. Only periodic measurements and analysis of deflection variations may provide grounds for such conclusions.

3.4.2. Structural displacement

Weak subsoil may cause excessive settlement of structure. In the case of non-homogeneous subsoil, different parts of the structure may be settled differently, leading to a deflection in relation to the vertical axis. Consequently, stability of the building may be lost. The above-mentioned displacement is measured using the surveying methods (e.g. levelling instruments, photogrammetry or 3D scanners).

Sketches of displacement due to non-uniform settlement of a high apartment building, prepared on the basis of a surveying report, are presented in Fig. 18.

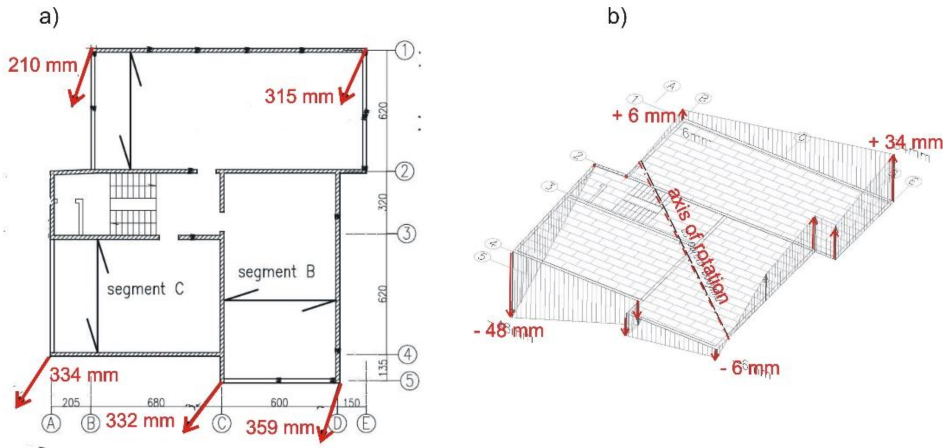


Fig. 18. Sketches of displacement due to non-uniform settlement of a high dwelling house prepared on the basis of a surveying report [40]: a) vectors of horizontal displacement of the roof, b) vectors of vertical displacement and axis of rotation of the building established on the basis of these vectors

3.4.3. Crack measurement

When cracks are found in a masonry or concrete structure, the most important thing is to find out whether these cracks are active. An active crack means that its width continues to increase. This suggests that the phenomenon causing the crack appearance has not been completed, e.g. ground settlement continues.

The easiest way to find out whether a crack is still active, is to fix a gypsum or glass seal (Fig. 19) on it. The seal should be fixed with glue on both sides of the crack on the brick or concrete, after removing the existing plaster or covers. After some time (e.g. after winter or after spring snowmelt), the seal should be examined. If the crack is active – seal cracking shall be observed.

The best seals are ‘eight’-shaped gypsum seals about 5 mm thick (the left-hand photo in Fig. 19). The narrowing area is also the zone where thickness is reduced up to 2–3 mm. If the crack is active, this narrow and thin zone will crack.

If the above-described gypsum seal is not available, a slip of glass no more than 2 mm thick may be used (the right-hand photo in Fig. 19). Unfortunately, a glass seal tends not to crack but to unstick. This is difficult to interpret: the crack may be active or the glue was not strong enough.

The seals should be marked with a date of fixing or a number.

Fig. 19. The seals for determining crack activity: gypsum seal (left), glass seal (right)

Crack width may be assessed using a ruler with a scale, or standard lines of different widths for comparison with the actual crack width (Fig. 20).

Crack measurements may be more accurate when using special equipment allowing to assess changes in crack width on the basis of periodic measurements of the same crack. Principles of operation of majority of the devices are based on slide caliper mechanism. Two parts (rulers) of such instruments are movable in relation to each other. The first part is fixed on one side of the crack and the second on the opposite side. The initial reading should be set on ‘zero’. Changes in crack width are reflected by positive or negative readings (different from ‘zero’). Reading accuracy is usually of 0.05 mm.

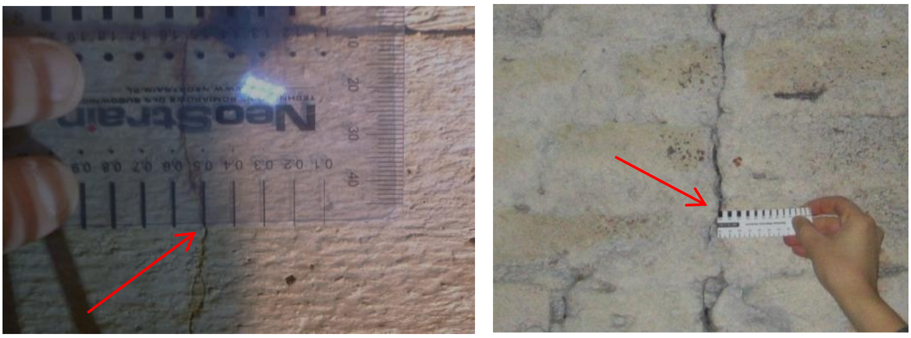


Fig. 20. Assessment of crack width with a ruler

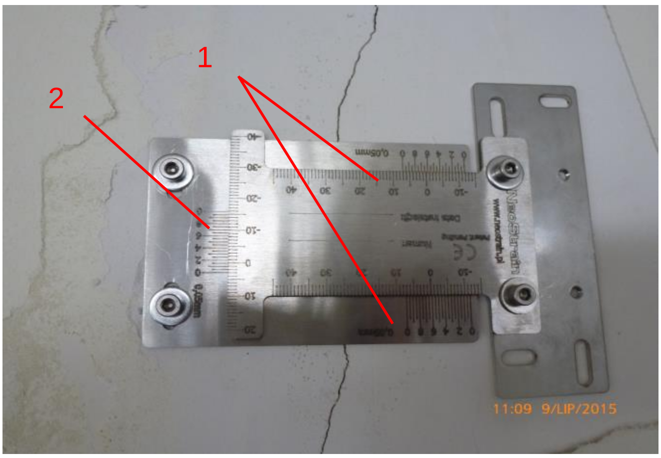


Fig. 21. Device for measuring crack variations (produced by NeoStrain): 1 – two horizontal linear scales for crack width, 2 – vertical linear scale for displacement of crack edges

There are many types of such devices:

- instruments expandable for one crack or used repeatedly after disassembly and assembly on other cracks, instruments measuring the crack width only (in one direction, perpendicular to the crack) or instruments allowing to measure crack width as well as displacement between two crack edges (measuring in two perpendicular directions), sometimes the angle of rotation of two parts of the structure separated by the cracks,

- instruments arranged as a flat ruler fixed on one flat surface or instruments fixed on surfaces perpendicular to each other, allowing to measure cracks near the corners.

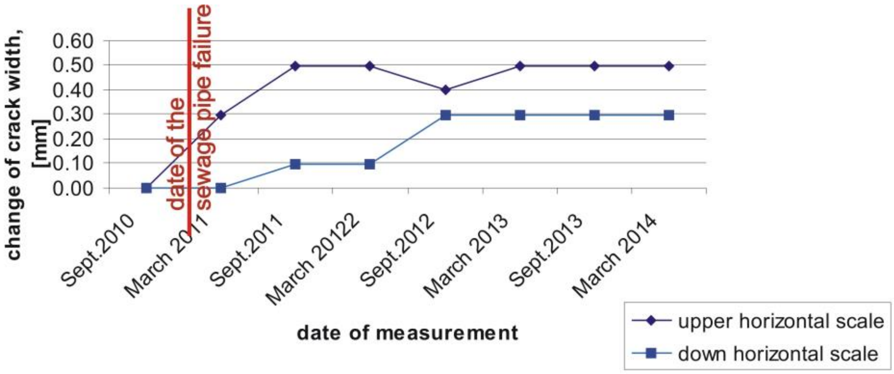


Fig. 22. A model chart illustrating the results of crack width variations measured with the device shown in Fig. 21 [40]; the measurement confirmed the impact of sewage pipe failure under the building on crack width

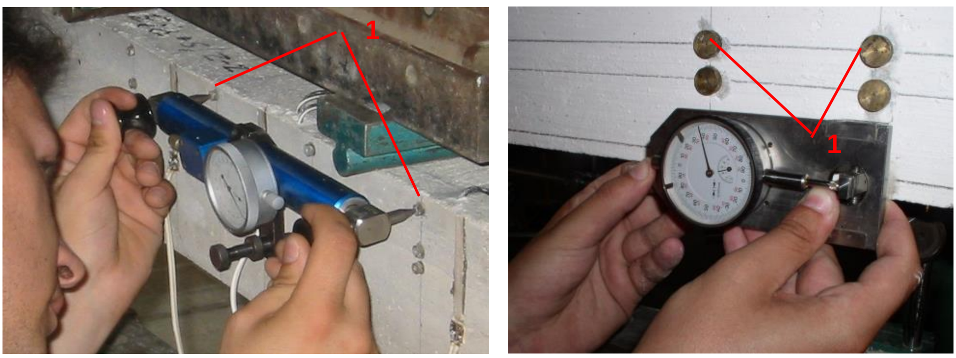


Fig. 23. The measurement of distance between bench marks using an extensometer with a dial indicator; 1 – bench marks

An example of an instrument measuring crack width changes is shown in Fig. 21. It is equipped with two horizontal linear scales for measuring crack width changes (two scales allow to assess the rotation angle) and the vertical linear scale perpendicular to them for measuring crack displacement. Fig. 22 presents a model chart illustrating changes in crack widths**.**

Variations in crack width may also be measured with an extensometer equipped with a dial indicator (Fig. 23). It usually allows for reading accuracy of 0.01 mm or even 0.001 mm. In order to use this method, the bench marks must be glued on both sides of the crack and the actual distance between the bench marks is measured (Fig. 24).

Figure 24. Devices for crack observation: 1 – gypsum seal, 2 – a couple of bench marks for crack width measurement, 3 – a couple of bench marks needed for consideration of temperature fluctuations in the results

3.4.4. Strain measurement

In order to assess load level of the structure under the increasing load, strain measurements may be necessary. It may be performed using the system of electrical strain gauges.

An electrical strain gauge takes advantage of the physical property of electrical conductance and its dependence on the conductor's geometry. When an electrical conductor is stretched, its electrical resistance increases. Therefore, measurement of electrical resistance of the strain gauge allows to assess the amount of induced stress and, consequently, the amount of strain. If the gauge is glued to the structure, structure strain may be regarded as equal to the gauge strain.

A typical strain gauge arranges a long, thin conductive strip in a zig-zag pattern of parallel lines attached to the foil (Fig. 25 on the left) and connected to the measuring device (the so called ‘Wheatstone bridge’ – Fig. 25 on the right).

Figure 25. Measurement of concrete structure strains: electrical dial gauges glued to concrete (left), ‘Wheatstone bridge’ produced by Hottinger Baldwin MessTechnic (right)

3.5. Identification of condition and parameters of structural materials and components

3.5.1. Groups of methods

Methods used to assess the characteristics of structural materials may be divided into three groups: visual or organoleptic methods, destructive methods and non-destructive methods.

3.5.2. Visual and organoleptic methods

Inspection of structural member from the outside

Visual and organoleptic methods are used to assess geometric shape of an element, losses in the material or cavities, surface quality including presence of plaster or coatings and their condition, presence of dampness or salinity, presence of mould, fungi or pests (the latter group especially including insects in timber members).

Simple methods for estimating the condition of the structural material using simple hand tools (e.g. hammer, chisel) may be also classified in this group (Fig. 26, 27).

Fig. 26. Estimation of the strength of structural material using simple tools: estimation of concrete strength with a hammer (left), estimation of masonry integrity with a hammer (right)

Hammer strokes produce scratches or pits in the surface of weak concrete or brick, whereas tough concrete remains unaffected. When concrete or brick is very tough, the strokes may even produce sparks. The sound of a hammer is also different – in the case of weak concrete or brick, the sound is dead, when they are tough however – the sound is top.

While assessing hardness of the material one may use a chisel (weak material is scratched off). This tool chisel may also be used to try to pick the mortar out of the masonry, which confirms weakness of the mortar and lack of integrity of the masonry structure (Fig. 27 on the left). A chisel may also be used to estimate the condition of wood elements – it can easily penetrate weak bio-corroded wood causing damage to wood fibers and appearance of the yellow powder (Fig. 27 on the right).

Fig. 27. Chisel work: determination of limestone toughness by scratching (left), a chisel penetrating weak bio-corroded wood (right)

Uncovering of the internal layers of structural members

Uncoverings allow to inspect inner layers of structural members. They are carried out in order to:

- determine the thickness of plaster and thermal insulation,
- determine whether the cracks visible in the plaster are also present in the brick or concrete (Fig. 28),
- identify the materials used for internal layers (Fig. 29) and their technical condition.

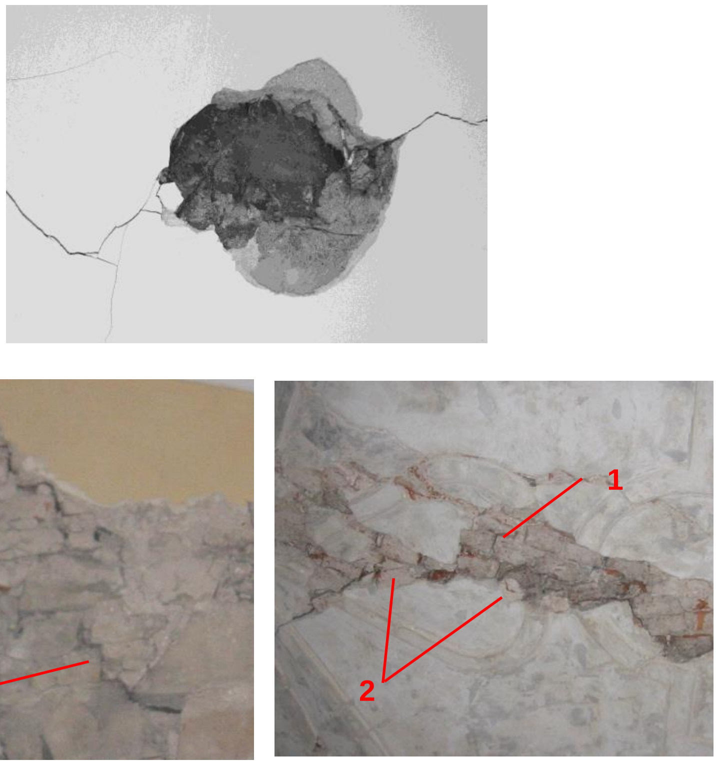


Fig.28 Uncovering of masonry structures intended to find out whether the crack exists only in the plaster or it goes through the masonry as well: no cracks under the plaster (top), cracks existing under the plaster (bottom) – cracks in the joints between bricks (1) and crack going through the brick (2)

Regions to be uncovered should be carefully chosen and they should be as small as possible for their intended purpose. They should not affect the safety of the structure due to excessive reduction of the structural members' cross-sections. Once the examination is completed, they should be filled with the repair material.

Fig. 29. Uncovering of the internal layers of the wall: 1 – outer calcium-silicate brick layer, 2 – foam glass layer, 3 – concrete wall

Uncovering of the reinforcement

The reinforcement is uncovered in order to: measure bar diameters, identify the reinforcing steel grade, and determine rebar condition in terms of corrosion (Fig. 30).

Rebars diameters and yield stress of reinforcing steel are used in calculations of bearing capacity and reliability of reinforced concrete structures, described in chapter 3.7. The results of the calculations are only reliable when the actual bar diameters and yield stress values are determined properly.

The actual bar diameters should be measured with callipers. If the corrosion degree is significant, the bar should be cleared off rust before the measurement, in order to reflect how much diameter is reduced due to rust.

Yield stress value may be assumed on the basis of ribbing of the uncovered bars, which allows to identify the steel grade. The ribs shape is attributed to the steel grade by standards and technical approvals. Fig. 31 presents examples of bar ribbing. Many more ribbing patterns of different grade are to be found e.g. in the work by [13].

In old reinforced structures, one can come across quite different steel types. Three of them (from the first half of 20th century) are presented in Fig. 32 and Fig. 33.

If it is impossible to determine steel grade based on ribbing, or if the expert would like to have more accurate steel parameters, the bar sample may be cut out from the structure (Fig. 34) and investigated in a laboratory tensile test.

Fig. 30 Uncoverings intended to determine bar diameter, thickness of the concrete cover and degree of bars corrosion in the slab (top), wall and column (bottom)

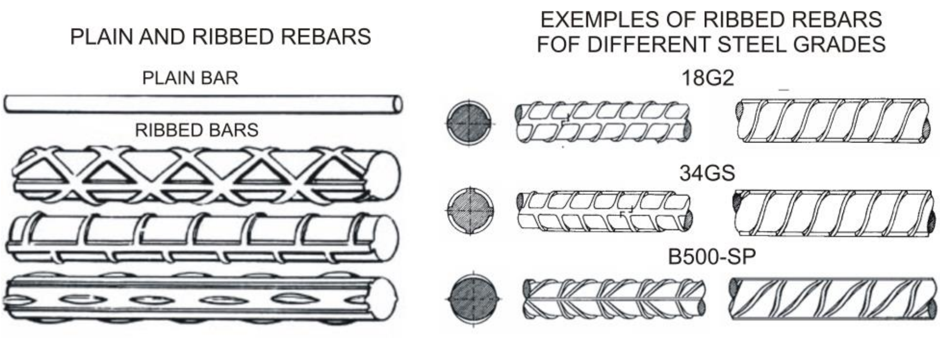


Fig. 31 Exemplary ribbing of bars as the method of steel grade identification

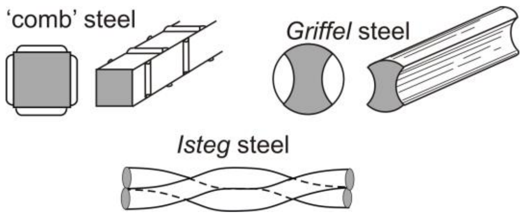


Fig. 32 Old steel grades: 'comb steel' (yield stress about 440 MPa, *Griffel* steel (yield stress about 430 MPa), *Isteg* steel (yield stress about 370 MPa)

Fig. 33 *Isteg* steel in the rib-and-slab floor from the 1930s

Fig. 34 Measurement ob bar diameter (left), the place remaining after the bar sample was cut off (middle) and a sample of rusty bar before laboratory tests (right)

Uncovering of bars allows to measure thickness of the concrete cover and assess whether it is sufficient for the actual exposure class. They may also serve for rough estimation of concrete cover carbonation and pH. The most popular test is the use of phenolphthalein solution (1 g of phenolphthalein dissolved in 70 ml of ethyl alcohol and 100 ml of water) [10, 46]. The solution should be sprayed onto the uncovered concrete surface (if the concrete is dry, distilled water should be sprayed first). The place where the phenolphthalein solution was sprayed, should turn reddish-violet in about 30 seconds. If the change of color does not occur or it is slow, it can be concluded that the carbonation has started.

Not only the phenolphthalein solution may be used in carbonation assessment. Other similar indicators are commercially available, including those which indicate the pH value with colors.

Fig. 35 The phenolphthalein test of concrete cover carbonation – the violet colour is visible near the surface but deeper (near the bar) the colour disappears

Inspections of the internal parts of structural members

Internal parts of a structure may be inspected via drilled holes using an optical device called a borescope (Fig. 36). Such inspections may also be conducted using the existing cracks in concrete or masonry structures.

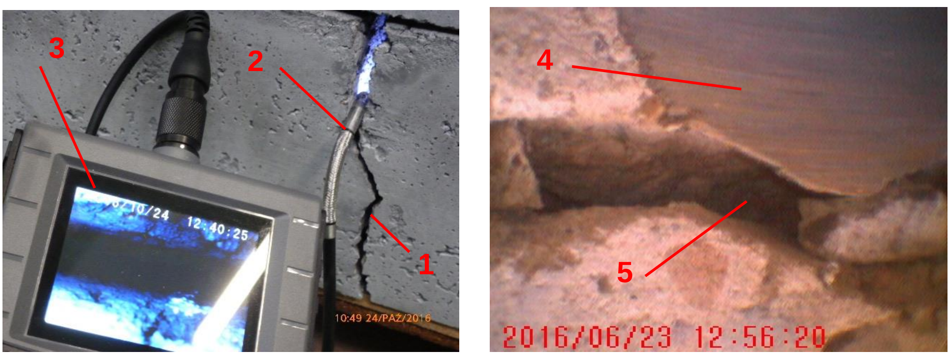


Fig. 36 Inspecting interior surfaces of the crack with a borescope (left) and a view of the delamination in the concrete wall as a result of borescope inspection (right): 1 – crack, 2 – borescope, 3 – imaging device, 4 –steel reinforcing bar, 5 – crack under the bar

A borescope consists of a flexible tube with an objective lens on one end and an imaging or video device on the other, linked together by a rely optical system in between. The optical system is surrounded by optical fibres illuminating the inside of a hole or crack.

3.5.3. Laboratory identification of the structural materials parameters using samples from structural members

Laboratory testing – purposes of taking samples

Parameters of the materials may be identified most accurately by taking samples of the material out of the structure and testing them in a laboratory using standard laboratory methods. This is mostly done to find the following features of the structural material: density, humidity, water absorbability and water penetration, porosity (e.g. of concrete, stone, brick) and strength (e.g. of concrete, stone, brick, as well as steel, including reinforcing steel).

The size and nature of the sample must match the type and aim of the test. Location from which the sample is collected should be carefully chosen – it should not affect the safety of the structure by excessive reduction of the structural members' cross-sections. Once the examination is completed, holes should be filled with the repair material.

Fig. 37. Drilling the cores out of the concrete wall for strength tests (left), and a hole in a concrete structure left after extraction of a concrete core (right)

To estimate the compressive strength, concrete cores of several centimetres are drilled out with a rotary cutter – a drill with a hollow barrel tipped with industrial diamond bits (Fig. 37). The rules for determining concrete strength based on drilled cores are given in chapter 3.5.4. There are also other trials using the similar method to estimate strength of the masonry [20].

Chemical tests on the other hand, require powder to be taken from the structure. The powder may be obtained directly from the structure by disintegration of bigger samples. In the case of concrete, it may be taken out with a drill of several millimetres in diameter (Fig. 38). In the case of masonry, the mortar for testing may be obtained by picking out with a chisel and the brick – by hammer use. Chemical tests allow to determine content of destructive salts or chloride ions in bricks and mortar or in the concrete cover, and pH of the solution prepared from the concrete cover.

Fig. 38. Drilling concrete powder out of a concrete structure (on the left) and holes left in a concrete structure (on the right)

Samples of the structural material, taken from the structure or obtained from the drilled cores after testing, may be even put to structural tests (e.g. scanning electron microscopy, SEM).

Laboratory testing of concrete strength

Concrete compressive strength may be most accurately assessed by core tests. The method is described in chapter 3.5.4.

Chemical testing of the masonry and concrete cover

Chemical tests are performed on solution of powder obtained from concrete, mortar or bricks, prepared especially for that purpose.

Detailed descriptions of chemical testing of the concrete cover, i.e. sample preparation, testing and results interpretation procedure, may be found in books by [10, 46]. Below, only essential information is given regarding three corrosive agents: pH, as well as chloride and sulfate ions.

The main factor responsible for protecting rebars against corrosion is alkaline concrete cover, which ensures passivation of steel. The concrete cover protects the bars, if pH is not lower than 12. The pH value may decrease due to ongoing carbonation process. It is posited that the concrete cover protective ability diminishes, if pH is lower than 11.5. They disappear completely when pH < 10. The pH value is determined in tests of concrete powder solution with devices called pH meters.

The second factor influencing steel corrosion is presence of chloride ions. If in close proximity of the bar (2–3 mm) the amount of these ions exceeds the critical value, the electro-chemical cell may arise leading to pinhole corrosion. The critical value is regarded as 0.4% in relations to cement mass in reinforced concrete structures and 0.2% in prestressed structures. The chloride ions amount is determined by using chemical methods, given e.g. in EN 480-10 [S5]. Also, there are commercially available sets of chemical reagents accompanied by a electro-chemical meter, which assess the content of chloride ions in an easy way. It should be emphasized that chemical tests are conducted for concrete or mortar, not on cement only and the results are related to concrete or mortar mass. Therefore, in order to compare them with the critical values, the results should be recalculated in relation to cement mass, taking into account percentage of cement in concrete mass.

The presence of excessive soluble sulfate ions may cause damage of concrete by chemical reactions resulting in ettringite or gypsum production. These products have bigger volume than the substrate used in reaction, therefore they burst concrete from the inside. The natural amount of sulfate ions is less than about 3.5–4% in relations to cement mass. Such percentage may be regarded as safe. If the tests results are higher, this may be the symptom of corrosion risk by sulfates. The sulfate ions amount is determined using chemical methods, given e.g. in EN 196-2 [S4]. The results need to be recalculated, just as in the case of chloride ions.

Tab. 1. Classification of salts content in masonry in accordance with WTA recommendations [S16]

amount of ions	low	medium	high
chloride	< 0.2 %	0.2 – 0.5%	> 0.5 %
nitrite	< 0.1%	0.1 – 0.3 %	< 0.3%
sulfate	< 0.5%	0.5 – 1.5 %	< 1.5%

The chemical mortar and brick testing is usually carried out with the use of commercially available sets of reagents designed to determine salts content. The chloride, sulfate and nitrite ions amount is classified as ‘low’, ‘medium’ or ‘high’ on the basis of the percent value. The WTA recommendations [S16], cited in Tab. 1, although dedicated to heritage structures, are usually used for this purpose for all masonries. Identification of salts content determines proper choice of repair methods and materials.

3.5.4. Destructive methods for concrete strength assessment

Introductory information

Assessment of concrete compressive strength in existing structures may be performed using a direct method – core test or indirect tests (rebound hammer tests, ultrasonic pulse velocity tests and pull-out tests). Where indirect tests are used, the uncertainty associated with the relationship between the indirect test and core test should be carefully considered.

The best results of assessing concrete strength in large members or entire structures are obtained by a combination of various testing methods.

Core specimens taken out of the structure – direct destructive method for compressive strength determination

Concrete compressive strength may be most accurately assessed by core sampling and testing. The rules of taking, examining and testing the cores, as well as interpreting the obtained results are given in EN 12504-1 [S8] in Europe and in other standards, e.g. by ACI Committee 214.4-03 [S1].
Cores are usually cut out using a rotary cutter – a drill with a hollow barrel tipped with industrial diamond bits. The whole rig has to be rigidly positioned – firmly fixed in position against other parts of the structure (Fig. 37). Cutting of the existing reinforcement should be avoided. Unless specifically required otherwise, cores should be drilled perpendicular to the surface.

Fig. 39. Core drilled out of concrete before trimming (left) and a trimmed core with height equal to its diameter during testing (right)

In this manner, a cylindrical specimen is obtained, with uneven ends and sometimes embedded pieces of the reinforcement. Immediately after cutting, each core is clearly and indelibly marked, to indicate its location and orientation within the member. The cores should be described and photographed, with attention paid to compaction, distribution of aggregates, presence of steel etc.

Before testing, the cores have to be trimmed to the proper length, and their ends need to be made flat and perpendicular to the longitudinal axis. This is achieved by grinding or capping the cores with cement mortar. Caps should be made as thin as possible, and their thickness should not exceed 10 mm at any point.

The preferred core diameter should be at least 3 times the maximum aggregate particle size. The rotary cutting tool for extracting *in-situ* concrete cores allows to obtain diameters of 50 mm to 150 mm (most common sizes are 75, 96, 120 and 150 mm). Smaller cores are also permitted, however cores having small a diameter exhibit higher variability in results than larger cores, hence their use is generally not recommended.

Concrete classes are estimated using standard specimens – cylinders with diameters of 150 mm and height of 300 mm, or cubes of 150 mm. For this reason, conversion of test results obtained for different core diameters should be performed. According to EN 13791 standard [S12]:

- testing a core of length equal to diameter and a diameter of 100 mm gives the strength value equivalent to the strength value of a 150 mm cube manufactured and cured under the same conditions;
- testing a core of a diameter of at least 100 mm and not larger than 150 mm and with a 2.0 length to diameter ratio gives strength value equivalent to the strength value of a 150 mm × 300 mm cylinder manufactured and cured under the same conditions;
- conversion of test results for cores with diameters of 50 mm up to 150 mm and other length to diameter ratios shall be based on conversion factors of established suitability.

Conversion factors of established suitability for other specimen sizes and length to diameter ratios may be specified in provisions valid in the country of use. The ACI 214.4-03 standard gives the following relationships:

$$f_{\text{core}} = 1 - (\beta - 4.3 \cdot 10^{-4} \cdot f_{\text{core,test}}) \left(2 - \frac{l}{d} \right)^2 , \qquad (2)$$

where:

f_{core} – compressive concrete strength converted for the cylinder of height twice its diameter,

$f_{\text{core,test}}$ – tested compressive strength,

l – cylinder height,

d – cylinder diameter,

β – value depending on sample dampness: $\beta = 0.130$ for a sample tested right after it was obtained, $\beta = 0.117$ for a sample soaked for 48 hours and $\beta = 0.144$ for an air-dried sample.

Data for different diameters and *l*/*d* ratios compiled in the book [33] may be also useful for the above-mentioned conversion.

The diameter and the height-to-diameter ratio are not the only factors influencing the obtained core strength. Others include: moisture, reinforcement in the core, drilling direction in relation to the direction of concreting, age of concrete [28, 33, S1].

Tab. 2. Minimum characteristic *in-situ* compressive strength for compressive strength classes in accordance with EN 13791 standard [S12]

Compressive strength class	Minimum characteristic in-situ strength	
	$f_{ck,is,cyl}$, MPa	$f_{ck,is,cube}$, MPa
C8/10	7	9
C12/15	10	13
C16/20	14	17
C20/25	17	21
C25/30	21	26
C30/37	26	31
C35/45	30	38
C40/50	34	43
C45/55	38	47
C50/60	43	51
C55/67	47	57
C60/75	51	64
C70/85	60	72
C80/95	68	81
C90/105	77	89
C100/115	85	98

EN 13791 [S12] standard sets out the rules for determining the concrete class based on core tests (Tab. 2). The basic values are $f_{ck,is,cyl}$ (characteristic *in-situ* compressive strength expressed in the equivalent strength of a 150 × 300 mm cylinder) and $f_{ck,is,cube}$ (characteristic *in-situ* compressive strength expressed as the equivalent strength of a 150 mm cube).

Estimation of the above values is based on the mean *in-situ* compressive strength of *n* test results ($f_{m(n),is}$) and the lowest *in-situ* compressive strength test result ($f_{is,\,lowest}$). The rules are as follows:

- if there are at least 15 specimens, the estimated *in-situ* characteristic strength is the lower of the two values:

$$f_{ck,is} = f_{m(n),is} - k_2 s\,, \tag{3}$$

or

$$f_{ck,is} \le f_{is,lowest}\,, \tag{4}$$

where: *s* is standard deviation of test results (if $s \le$ 2.0N/mm^2, the value used is fixed as 2.0MPa), k_2 = 1.48.

- if the number of available cores ranges from 3 to 14, the estimated *in-situ* characteristic strength is the lower of the two values:

$$f_{\mathrm{ck,is}} = f_{\mathrm{m}(n)\mathrm{,is}} - k\ , \tag{5}$$

or

$$f_{\mathrm{ck,is}} = f_{\mathrm{is,lowest}} + 4\ , \tag{6}$$

where k depends on the number n of test results (k = 5 for 10–14 samples, k = 6 for 7–9 samples, k = 7 for 3–6 samples).

Pull-off test – destructive method for tensile strength testing

Pull-off test was originally developed to measure bond strength between two materials. Therefore, major applications of this method in concrete structures include:

- checking bond strength of repair materials (coatings, paints, repair mortars),
- assessment of concrete or mortar tensile strength.

To conduct the test, a simple, mechanical, hand-operated loading equipment was developed (Fig. 40), having an integrated digital manometer and providing constant jerk-free load increase through the use of an easy-running crank (an optional, electric drive unit is available). A circular steel or aluminum disc having a diameter of 50 mm is glued to the surface of the concrete using epoxy or polyester resin. The force required to pull this disc off the surface, together with an attached layer of concrete, is measured. On the basis of the ‘pull-off’ value obtained during testing, concrete tensile strength is estimated. The rules of ‘pull-off’ testing are set out in EN 12504-3 standard [S10]. The usefulness and reliability of this method in assessing the concrete strength were confirmed in a body of previous research, e.g. [7].

The ‘pull-off – tensile strength', and consequently ‘pull-off – compressive strength’ relationships depend on the type of aggregate used and thus, on concrete modulus of elasticity. Failure is expected to occur while applying lower load as the modulus of concrete increases.

In accordance with EN 13791 standard [S12], a specific relationship between *in-situ* compressive strength obtained in the ‘pull-off’ (f_{F}) test and the ‘pull-off’ test result (F) may be established:

- by calibration using the core tests: the relationship should be based on at least 18 pairs of results, 18 core test results and 18 ‘pull-off’ test results,
- using the basic curve given in the operating manual of the device or in EN 13791 standard, where the following expression is given:

$$f_{\mathrm{F}} = 1.33(F - 10) \qquad \text{for } 20 \le F \le 60, \tag{7}$$

although the above basic curve should be corrected due to at least 9 results of core strength.

Fig. 40. Pull-off method: a circular steel disc glued to the tested concrete (top); testing using the Dyna Pull-Off Tester (left); obtained result and surface of the concrete after testing (right)

3.5.5. Non-destructive methods of estimating strength parameters

Introductory information

The group of non-destructive tests intended to estimate strength parameters employ the principles of mechanics. A spring-loaded mass strikes against the surface of the sample with a defined energy or velocity. The rebound causes a change (reduction) in energy or velocity. If the tested structural material is soft, the energy is lost by plastic deformation of the material near the surface. The harder the material, the smaller the reduction. The measured ‘rebound value’ can be used to determine hardness or compressive strength by reference to the conversion expressions or the conversion chart.

Estimation of concrete compressive strength using a Schmidt hammer

Concrete Test Hammer is a mechanical device used for performing non-destructive testing of materials, mostly concrete. It was invented by Ernst Schmidt, a Swiss engineer, therefore it has been known as a Schmidt hammer or a Swiss hammer. The device is distributed by Proceq company. It measures the rebound of a spring-loaded mass impacting against the surface of the sample. The hammer hits the concrete with a defined energy and displays the rebound value *R*. There is a specific relationship between this value and the hardness and strength of the concrete.

The Schmidt hammer has an arbitrary scale ranging from 10 to 100 and is available in different energy ranges: type L – with impact energy of 0.735 Nm, type N (most popular) – impact energy of 2.207 Nm and type M – 29.43 Nm. N and NR models can be used in testing of concrete items with thickness of 100 mm or more, with the maximum particle size of the aggregate ≤ 32 mm. L and LR models can be used in testing items with smaller dimensions (e.g. thin-walled items with thickness ranging from 50 to 100 mm).

Fig. 41. Schmidt hammer of type NR: ready to test, with the visible impact plunger (left) and during the test – impact plunger hidden inside the hammer (right)

LR and NR subtypes include embedded recorders – rebound values are recorded as a bar chart on a paper strip which has a capacity for 4000 test impacts without reloading (Fig. 41).

An example of the newer generation Schmidt hammers are SilverSchmidt (Fig. 42) and DigiSchmidt. The DigiSchmidt rebound hammer combines the classic Schmidt hammer with an external LCD display unit that allows the direct reading and display of test results. SilverSchmidt has the inner LCD display not affected by the angle of impact – the rebound value requires no angular correction. Velocity-based detection of the rebound value results in smaller dispersion than in the classic concrete test hammer. The impact plunger is made from aerospace alloy and equipped with a hardened steel cap.

Fig. 42. SilverSchmidt hammer of type N with an external LCD display: ready to test, with the visible impact plunger (left), during the test – impact plunger hidden inside the hammer (right)

Prior to testing, the Schmidt hammer should be calibrated using a test anvil supplied by the manufacturer. Twelve readings should be taken, ignoring the highest and the lowest, and the average should be calculated from the remaining ten readings. A typical anvil is made of steel with hardness of H_B=5000 N/mm^2 according to Brinell scale, and the rebound on that type should be 80±2.

Testing should be performance in accordance with EN 12504-2:code [S9]. Each tested surface should receive at least 9 impacts and the individual points must be spaced at least 25 mm apart and located in the same distance from the edges of a member.

Tab. 3. Correction coefficient for non-horizontal position of the Schmidt hammer

Rebound value	Impact plunger up		Impact plunger down	
	$\alpha = +90^o$	$\alpha = +45^o$	$\alpha = -90^o$	$\alpha = -45^o$
20	-5.1	-3.5	+2.5	+3.4
30	-4.7	-3.1	+2.3	+3.1
40	-3.9	-2.6	+2.0	+2.7
50	-3.1	-2.1	+1.6	+2.2
60	-2.4	-1.6	+1.3	17

While conducting the test, the hammer should be held at proper angles in relation to the surface which should be flat and smooth (grindstone can be used to smoothen the test surface). The rebound reading will be affected by the orientation of the hammer; when used in a vertical position (on the underside of a slab, for example), gravity will increase the rebound distance of the mass, whereas the reverse phenomenon will be observed for a test conducted on a floor slab. For this reason the rebound values need angular correction by adding or subtracting the correction coefficient shown in Tab. 3.

Besides the direction of the hammer, the following factors must be also taken into account when establishing rebound values *R*:

- local variation of concrete strength – to minimize this, it is recommended to take a selection of readings and calculate an average value,
- water content in the concrete – wet material will give different (lower) results than the dry one,
- age of concrete – if concrete is tested at 28 to 100 days, the obtained rebound values are regarded as conclusive but older concrete has a more hardened surface due to carbonation; therefore, the correction coefficient due to the age of concrete may be used, e.g. compiled in Tab. 4.

Tab. 4. Correction coefficient due to age of concrete according to [36]

Age, days	Value of the coefficient
10	1.20
20	1.04
28–100	1.00
150	0.92
200	0.86
300	0.78
360	0.75
500	0.70
1000	0.63
> **1000**	0.60

In the paper by [5], more influential factors are mentioned: the value of load, surface finish, distance from free edges of the member.

Once testing is completed, the average strength of concrete is estimated on the basis of the average corrected rebound value R_m, using the conversion curve. It is necessary to choose the relevant curve from curves available in the Schmidt hammer manual or provided by standards or recommendations. It should be remembered that the obtained average compressive strength is subject to dispersion (±4.5 N/mm^2 to ±8.0 N/mm^2).

In accordance with EN 13791 standard [S12], the specific relationship between *in-situ* compressive strength obtained by the Schmidt Hammer (f_R) test and the rebound value (R) may be determined:

- by calibration with core tests: the relationship shall be based on at least 18 pairs of results, 18 core test results and 18 rebound test results,
- using the basic curve given in the operating manual of the device or in the EN 13791 standard, where the following expression is given:

$$f_R = 1.25R - 23 \quad \text{for} \quad 20 \le R \le 24\,, \tag{8}$$

$$f_R = 1.73R - 34.5 \quad \text{for} \quad 24 \le R \le 50\,, \tag{9}$$

the above basic curves should be corrected due to at least 9 results of core strength.

In the Instruction No. 210 of the Building Research Institute [36], the following conversion curve is given:

$$f_R = 0.0409R^2 - 0.915R + 7.4\,. \tag{10}$$

Testing metal hardness using Leeb rebound

The Leeb rebound hardness test method was developed in 1975 by Dietmar Leeb and Antonio Brandestini at Proceq SA to provide a portable hardness test for metals. It was developed as an alternative to the traditional hardness measuring equipment, e.g. Rockwell, Vickers and Brinell, which is stationary, requires fixed workstations and destructive tests on samples. Hardness value measured using the Leeb hammer is calculated from the energy loss of a defined impact body after impacting on a metal sample. The impact body rebounds faster from harder test samples than it does from softer ones. A magnetic impact body permits the velocity to be deduced from the voltage induced by the body as it moves through the measuring coil.

Fig. 43. Leeb hardness tester, Time TH 130

The Leeb rebound hardness unit is referred to as *HL*. The *HL* values are often converted into traditional hardness scales: Rockwell *HRC*, Brinell *HB* and Vickers *HV*, mainly for comparison. The requirement to convert the results from one hardness test scale to another is a common practice and is covered by various International Standards – ASTM E140 [S3] and ISO 18265 [S14]. These standards state that due to differences in various types of hardness test methods, it is not possible to show a fixed relationship across all materials. As such, the standards provide approximate conversion tables for different materials.

Most of the modern Leeb testers automatically compute all Vickers, Brinell, Rockwell or shore hardness values (like Time TH 130 model shown in Fig. 43).

3.5.6. Non-destructive methods for assessing structural member condition

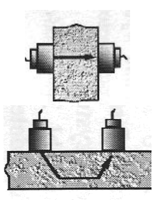
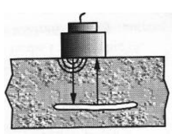
Introductory information

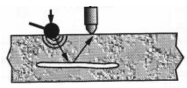
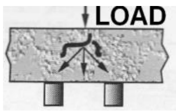
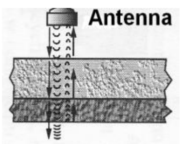
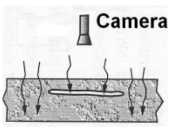
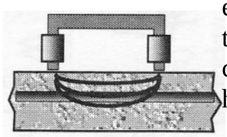
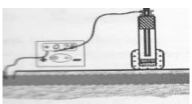
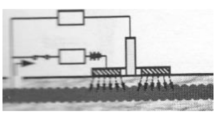
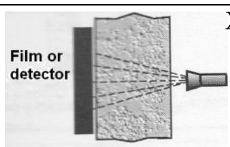
Non-destructive methods of assessing the condition of structural members employ the generation of different wave types (acoustic, ultrasonic, electromagnetic, electric), their propagation into the tested structural material and recording of their parameters [26]. The measured wave parameters depend on properties of the tested material. The non-destructive methods are compiled in Tab. 5.

They are mostly used to assess:

- thickness of elements (especially, of steel elements),
- moisture level and depth,
- homogeneity of the structural material: presence of voids, inclusions, depth of the cracked zone, thickness of coatings as well as diameter of the reinforcement in concrete structures and thickness of the concrete cover,
- risk of reinforcement corrosion.

Tab. 5. Non-destructive methods used in structural diagnosis using generation of different wave types (based on [18])

	Sketch of measurement	Measured parameter	Purpose
1	2	3	4
Ultrasonic pulse velocity UP-V		• time of ultrasonic wave propagation between the emitting and receiving heads • calculation of wave velocity	• assessment of material homogeneity • estimation of strength and modulus of elasticity
Ultrasonic pulse echo UP-E		• time of ultrasonic wave propagation from emitter to reflection plane • calculation of wave velocity	• detection and localization of defects, voids and cracks • assessment of element thickness

Impact-Echo I-E and impulse response I-RS		time and frequency spectrum of sound wave caused by the impact (generated by the hammer) reflected by flaws and external surfaces	• detection and localization of defects, voids and cracks • assessment of thickness, • detecting voids under the plates on the ground • depth of surface cracking
Acoustic emission		surface registration of sound elastic waves generated by internal cracking	fracture and crack analysis
Ground-Penetrating Radar GRP		parameters of electromagnetic waves (emission and recdording without direct contact with structure surface)	• detection of defects, • localization of the reinforcement • thickness of pavements, • estimation of humidity and chloride content
Thermovision IT		thermograph preparation on the basis of measurements of temperature of the surface	localization of defects near the surface
Electromagnetic		parameters of electromagnetic flux transferred between poles changed by the lack of homogeneity	• assessment of material homogeneity • localization of the reinforcement • assessment of concrete cover depth
Electric	 	electrical conductivity parameters: • resistance • electrochemical potential • current density	• assessment of/penetration of damped zones • estimation of the possibility and rate of reinforcement corrosion
X-ray		X-radiation weakened after transition via the element (impulse counter method or radiogram method)	• localization of defects • localization of the reinforcement

Short descriptions of selected non-destructive methods are presented below. They are based on the works by [13, 26, 21, 46] and operating manuals of the devices.

Detection of the reinforcement and depth of concrete cover

In order to determine the bearing capacity of reinforced concrete structures, the amount and location of the reinforcement should be known. Sometimes it is impossible to uncover the bars, as shown in Fig. 30. In such cases, non-destructive methods of reinforcement detection may be used. What is important, these methods should be used before drilling the cores for strength tests – cutting of the reinforcement should be avoided. Next, it is essential to find out the thickness of the concrete cover to evaluate its corrosion protection ability – for this purpose, the non-destructive methods of reinforcement detection may be used.

There are different devices for detecting the reinforcements. Their function is to localize the bars, determine their diameters and measure the concrete cover.

One of the most easy-to-use devices is a Profometer (Fig. 44) – a portable, battery-operated magnetic device. The underlying principle of the method it utilizes is that steel affects the strength of electromagnetic field. Profometer's probe unit consists of a magnetic core on which two coils are mounted. Alternating current is passed through one of these coils, and the current induced in the other coil is measured.

Fig. 44. Detection of the reinforcement and measurement of a concrete cover using Profometer 5+: search head (left), display of the main unit (right) with the measured value of concrete cover of 25 mm

The reinforcement is detected through bar magnetization and induction of eddy current in it. When the impulse is ended, the eddy current dies away, creating a weaker magnetic field as an echo of the initial impulse. The strength of the induced field is measured by the search probe as it dies away, and this signal is processed to give the depth measurement. The eddy current echo is determined by the depth, size and orientation of the bar. The device has a built-in function of taking the impact of the neighbouring bars and voids into account.

There are various factors affecting test results: arrangement of the reinforcement, variation of cement iron content and the use of aggregates containing iron or displaying magnetic properties.

More advanced models, like Profometer 5+ Scanlog or other devices (e.g. Ferroscan by Hilti), have a mobile probe – carriage with an integrated path measuring device that allows to make a map of the reinforcement (measuring areas of 0.5 × 0.5 m, 1.0 × 1.0 m or even 2.0 × 2.0 m) or display concrete covers over a large area using a grey scale or colour shades.

Other non-destructive methods were also employed for reinforcement detection, e.g. thermography [41].

Estimation of material homogeneity and thickness using ultrasonic flaw detector

Ultrasonic flaw detection is a well-established testing method used in many basic manufacturing process and service industries, especially in applications involving welds and structural metals. Since the 1940s, the laws of physics governing propagation of sound waves through solid materials have been used to detect hidden cracks, voids, porosity and other internal discontinuities in metals, composites, plastics and ceramics.

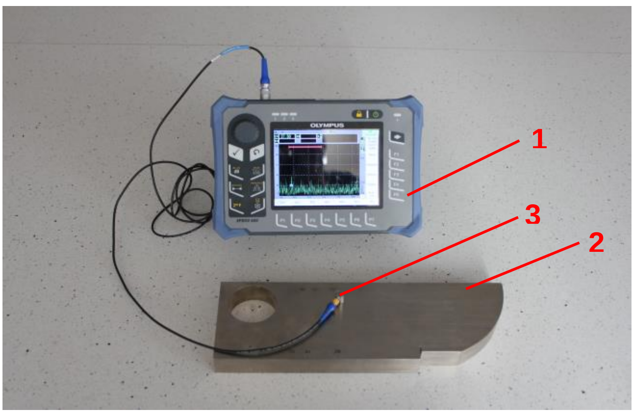


Fig. 45. Ultrasonic flaw detector Olympus EPOCH 600:
1 – detector, 2 – calibration plate, 3 – contact transducer

At high frequencies (from 500 kHz to 10 MHz), sound energy does not travel efficiently through air or other gases, but it travels freely through most liquids and common engineering materials. Sound waves of such frequencies reflect flaws in predictable ways, producing distinctive echo patterns that can be displayed and recorded. In ultrasonic flaw detection, the generally accepted lower limit of detection for small flaws is one-half wavelength – anything smaller than that will be invisible. An example of the above-mentioned devices is Olympus EPOCH 600 flaw detector (Fig. 45). It allows to estimate material homogeneity (especially, of metals) and to measure thickness of typical structural members having only one face exposed. It is a small, portable microprocessor-based instrument that generates and displays an ultrasonic waveform that is interpreted by a trained operator, often with the aid of analytical software, to locate and categorize flaws in test pieces. It is equipped with a Multicolour liquid crystal display (LCD) calibrated in units of depth or distance and the calibration plate.

An important component of the ultrasonic flaw detector is a transducer – a device that converts one form of energy into another, e.g. electrical energy into energy of high-frequency sounds, and vice versa. A typical transducer for ultrasonic flaw detection utilizes an active element made of piezoelectric, composite or polymer. When this element is induced by a high voltage electrical pulse, it vibrates across a specific spectrum of frequencies and generates a burst of sound waves. When it is set into vibration by the incoming sound wave, it generates an electrical pulse. The front surface of the element is usually covered by a wear plate that protects it from damage, and the back surface is bonded to backing material that mechanically dampens vibrations once the sound generation process is complete.

There are five types of ultrasonic transducers commonly used in flaw detection applications:

- contact transducers (Fig. 45) – used in direct contact with the test piece introducing sound energy perpendicular to the surface; they are typically used for locating voids, porosity and cracks or delaminations parallel to the outside surface, as well as for measuring thickness,
- angle beam transducers – used in conjunction with plastic or epoxy wedges (angle beams) to introduce shear waves or longitudinal waves into the test piece at a designated angle with respect to the surface; they are commonly used in weld inspection,
- delay line transducers – incorporating a short plastic waveguide or delay line between the active element and the test piece; they are used to improve near-surface resolution and also in high temperature testing, where the delay line protects the active element from thermal damage,
- immersion transducers – designed to couple sound energy into the test piece through a water column or water bath; they are used in automated scanning

applications and also where a sharply focused beam is needed to improve flaw resolution,

- dual element transducers, commonly known as tandems – they use separate transmitter and receiver elements in a single assembly; they are often used in applications involving rough surfaces, coarse grained materials, detection of pitting or porosity, and they offer good high temperature tolerance as well.

Estimation of homogeneity and compression strength of concrete using ultrasonic pulse velocity test

The main objective of the ultrasonic pulse velocity method (UPV) is to evaluate homogeneity of material, presence of cracks, voids and other imperfections. This method may also be used to find values of concrete dynamic modulus of elasticity and concrete compressive strength. For this purpose, the ultrasonic pulse velocity method is used, yet essentially less frequently when compared to impact hammers, as measurement results are influenced by many factors, e.g. concrete moisture, type of aggregate or the reinforcement. The rules of concrete strength assessment with UPV are set out in EN 12504-4 standard (S11).

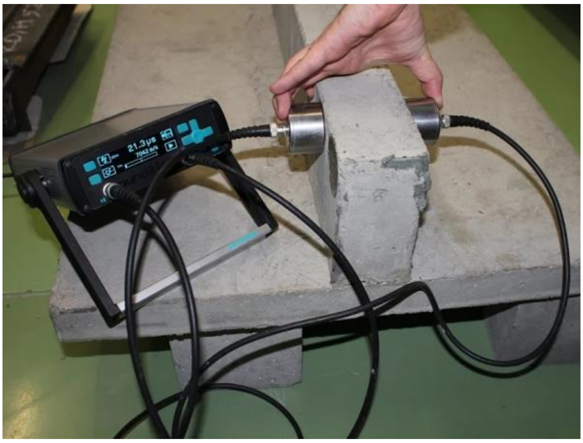


Fig. 46. Pundit Lab – the device used in ultrasonic pulse velocity method with transducers L40 (frequency 54 kHz)

Transducers with natural frequencies ranging from 50 to 100 kHz are the most common (50 to 60 kHz are useful for most common applications). There are three ways of measuring pulse velocity in concrete:

- direct transmission (cross probing) via structural member (Fig. 46) – transducers are held on the opposite faces of the tested specimen; this method is used most commonly and is preferred to the other two methods because of maximum sensitivity and providing the well-defined path length,

- semi-direct transmission via concrete – when one of the concrete specimen faces is not accessible, transducers are applied on adjacent surfaces (horizontal and vertical ones) of the tested element; the sensitivity of this method is smaller than that of cross probing, and path length is not clearly defined,
- indirect transmission (surface probing) via element – used when only one face of the element is accessible; it is the least satisfactory of the three methods (it only indicates the quality of structural material near the surface and does not give information about deeper layers).

The Pundit Lab device produced by Proceq may serve as an example (Fig. 46). It measures the time of ultrasonic wave propagation via tested element with microsecond accuracy. In the screen, the *V* value of pulse velocity is displayed. It can also measure the width of the tested element (i.e. the distance of ultrasonic wave transmission).

In order to use the ultrasonic pulse velocity method for assessing concrete strength, in accordance with EN 13791 [S12], a specific relationship between the *in-situ* compressive strength obtained by ultrasonic pulse velocity method (f_V) test and the pulse velocity (V) may be determined:

- by calibration using the core tests: the relationship should be based on at least 18 pairs of results, 18 core test results and 18 'pulse-velocity' test results,
- using the basic curve given in the operating manual of the device or in the EN 13791 code, where the following expression is given:

$$f_{\mathrm{V}} = 62.5V^2 - 497.5V + 990 \quad \text{for } 4 \le \mathrm{V} \le 4{,}8 \text{ [km/s]}, \tag{11}$$

the above basic curve should be corrected due to at least 9 results of core strength.

Estimation of material humidity

The principle 'the higher the humidity of porous building material – the lower strength of such material and the higher the possibility of corrosion and biocorrosion processes' causes the need to check the humidity of porous building materials such as concrete, masonry or timber. Devices intended to measure water content in the material are called moisture meters.

Moisture meters with a pin sensor are intended to measure timber humidity, although they are sometimes used for other materials as well (Fig. 47 upper row photos). After the two probes, or 'pins', are inserted into the wood, a small electrical current is passed between them. Moisture is a good conductor of electricity, therefore 'the wetter the material, the less electrical resistivity of the tested material' and vice versa, and the amount of resistance is correlated with moisture content reading. Pin-meter accuracy is affected by variations in the natural chemical composition of the material, but is not sensitive to density variations.

The second popular method of moisture measuring is capacitance technology also known as Electromagnetic Wave Technology. EMW meters measure the moisture content in wood without piercing the wood with pins. The electromagnetic waves of a designated frequency are generated by a sensor which is pressed against the material (Fig. 47 bottom). These waves cause an electromagnetic field in the zone up to 25 mm deep. The capacity of wood to store energy (capacitances) is measured, and it depends on the amount of moisture in the wood. Pin-less meters typically ‘scan’ a much larger area than pin-style meters and give a more complete moisture content picture. This method is sensitive to density variations.

Fig. 47. Moisture meters: with a pin sensor (top), with a spheroid sensor (bottom)

For measuring humidity of concrete or masonry, pin-less devices are used as well. They contain a metal spheroid sensor (Fig. 47 bottom). This sensor is placed into a hole drilled in the tested material or left in fresh concrete during casting. What is important – if the sensor is in contact with the surface of the tested material only, it can measure the humidity of the outer layer of an element, up to about 25 mm. The most difficult to test are reinforced concrete structures, because steel bars may produce false results. Also, variations of concrete density and other variable chemical and physical characteristics, as well as type, size and amount of aggregate may produce false readings.

Non-destructive methods for evaluating corrosion of the reinforcement

Non-destructive methods for evaluating corrosion in reinforced concrete are described in [13, 46] and only mentioned below. They are based on half-cell potential or resistivity technologies.

Half-cell potentiometer is based on the phenomenon of electrical half-cell. The electrical activity of the steel reinforcement and the concrete makes them be considered as a weak battery cell (the so called half-cell), with steel acting as one electrode and concrete as the electrolyte. The electrical potential of the steel bar surface is measured with reference to a potential copper-copper-sulphate reference electrode placed on the concrete surface. Therefore, the electrical potential between the reinforcement and the concrete surface is measured. In practice, this is achieved by fixing the first wire connected with the terminal of a voltmeter to the reinforcement and the second one to the reference electrode. Then, readings are taken using the grid of 1.0 × 1.0 m – the so called corrosion mapping. This allows to evaluate the corrosion activity, as well as the condition of the concrete cover layer. The risk of corrosion is evaluated by means of the potential gradient obtained; the higher the gradient, the higher the risk of corrosion. The test does not evaluate the corrosion rate, neither does it specify whether corrosion activity has already started, but it indicates the probability of corrosion activity depending on the actual surrounding conditions. If this method is used in combination with resistivity measurements, the accuracy is higher.

The second method is based on the statement that there is a direct correlation between resistivity and chloride diffusion rate, and even the early compressive strength of concrete. Resistivity meter is designed to measure the electrical resistivity of concrete and allows to estimate the possibility of corrosion and the corrosion rate. The device consists of a display unit and resistivity probe. Measurements can be done using a grid to represent the resistivity value for a large area. The grid of a suitable size is marked on the surface and measurements are taken in cross-points. If the concrete is dry, measurements cannot be conducted as its surface should be moistured for the current to be carried by ions in the pore liquid. For this purpose, a foam pad with good contact with concrete is necessary. There are various factors affecting the results of resistivity measurements, such as moisture content, carbonation and chloride content, temperature, connection between the probe and concrete. If this technique is used together with half-cell potential measurements, it will give more accurate results and the corroded zone can be monitored more efficiently.

3.6. Proof load testing

Proof load tests are the only way to learn about the actual behaviour of the structure under load. They are focused on finding the 'load-deflection' relationship. This allows to read out the generalized stiffness of the structure as the proportionality ratio from 'load-deflection' curve. The values of ultimate load and the permitted variable load may be inferred from the extrapolation of the 'load-deflection' curve.

Proof load tests are performed in order to:

- verify the design assumptions – structural analysis is performed using simplified models of materials 'stress-strain' relationship, the simplified model with support conditions and the simplified load values; that is why, the actual behaviour of a given structure may differ from the designed one and the assessment of the structure at the stage of design capacity may be misleading,
- assess structure behaviour and predict its capacity when the parameters of the built-in materials are not the same as the designed ones (e.g. lower concrete strength or lesser amount of steel) or when these parameters are unknown.

Proof load tests are most commonly used in bridge testing, and trucks are used as the load. Nevertheless, in some situations there may occur the need to conduct proof loadings of building structures, especially of the floor slabs.

The proof load in buildings is applied as the gravity-based load: with sand, cement or other building materials placed on the slab. In the case of garages, cars may be used. Sometimes basins filled with water are used [37], but there is a risk of basin wall leaking (Fig. 48). Another type of proof loading, especially dynamic loading, is forcing structure displacement by means of hydraulic devices.

Slab deflection may be measured by dial indicators (Fig. 48 bottom), inductive gauges, or using geodesic methods.

It is very important to make a proper decision about the proof load level. It must not be too large or cause permanent damage to the structure. That is why the proof load should be limited and the load increase above the limit of elastic performance is forbidden. The maximum (the so called – target) proof load should be applied in several steps, and response of the structure should be observed during each step. Each step should last long enough to stabilize the structure and stabilize the readings of measured parameters. In accordance with ACI 437.2M standard [S2], the concrete structure may be considered as stabilized when the difference between successive readings taken, with intervals of not less than 2 minutes, does not exceed 10% of the initial value recorded for the current step. In the last step load should be left for 24 hours. After 24 hours, the structure should be unloaded and the final readings should be taken after another 24 hours period.

Fig. 48. Proof loadings of reinforced concrete slab, using water: filling in the basin with water and the basin full of water (top); the measurement of slab deflection (bottom): 1 – dial indicators under the slab

3.7. Calculations

3.7.1. Calculations focused on proving the suspected reason of failure

The first type of calculations of existing structures is focused on proving the suspected reason of failure. In such calculations, the Ultimate States method, in which the safety coefficients are involved, is not applicable. These calculations should be performed using the actual dimensions of structural members, actual minimum strength parameters of structural materials (f_{min}) and loads which were actually exerted onto the structure (G_{act}, Q_{act}). For instance, the real crack appearance is explained only by exceeding the actual tensile strength of concrete by actual tensile stress caused by actual load.

The condition ensuring that the damage occurred due to exerted load may be written as follows:

$$E(\sum_i G_{\mathrm{act},i}, \sum_j Q_{\mathrm{act},j}) > R(f_{\min}) \tag{12}$$

where

$E(\sum_i G_{\mathrm{act},i}, \sum_j Q_{\mathrm{act},j})$ – the effect of actual loads (internal force, stress),

$R(f_{\min})$ – actual resistance (bearing capacity) of the structural member.

3.7.2. Calculations focused on assessment of the safety of damaged structure

The second type of calculations is focused on the assessment of the reliability level (safety) of the existing structure. Two approaches may be used here.

The reliability index approach (similarly as mentioned in chapter 1.3.2) relies on the statement, that the real reliability index decreases with the passage of time and due to failures, but it should never drop under the acceptable value. These values, recommended for a 10 year reference period, are given in the ISO/CD 13822 standard [S13] depending on severity of failure consequences (very small consequences, small consequences, moderate consequences and serious consequences. In the SIA 269 code [S15], the requirements for target values of the reliability index depend on cost calculation.

Referring to the Ultimate States method, the condition ensuring that the reliability is of the demanded level, may be written down as follows:

$$E(\sum_i G_{\mathrm{k},i}\gamma_{\mathrm{G},i}, \sum_j Q_{\mathrm{k},j}\gamma_{\mathrm{Q},j}) > R(\tfrac{f_{\min}}{\gamma_{\mathrm{m}}}), \tag{13}$$

where

$E(\sum_i G_{\mathrm{k},i}\gamma_{\mathrm{G},i}, \sum_j Q_{\mathrm{k},j}\gamma_{\mathrm{Q},j})$ – the effect of design values of loads (internal force, stress),

$R(\frac{f_{\min}}{\gamma_{\mathrm{m}}})$ – designed value of resistance (bearing capacity) of the structural member.

If the above Ultimate Limit States condition is satisfied, the member or structure is safe. Nevertheless, the opposite situation (condition not fulfilled) should not be identified with the actual risk of failure. This merely means that the reliability level is below the value set out in the standard. The assessment whether such structure is safe is a task performed by expert.

An easy way to show a margin of structural safety is to use the global safety factor, which was similar to that used in the ’limit stress method’ before the

Ultimate States method was used. While adopting the global safety factor, the actual dimensions of structural members should be used and actual material strength should be taken into account. The expected values of loads multiplied by safety coefficients should be used. The actual global safety coefficient may be calculated as:

$$\gamma = \frac{E(\sum_i G_{k,i}\gamma_{G,i}, \sum_j Q_{k,j}\gamma_{Q,j})}{R(f_{\min})}, \qquad (14)$$

$\gamma < 1$ means that there is direct risk of failure.

3.7.3. Calculations for designing the repairs

Calculations focused on designing the repairs require typical use of Ultimate Limit States. The actual dimensions of structural members should be used, and actual material strength and designed repair material strength should be taken into account with material safety coefficients. The expected load values multiplied by safety coefficients should be used.

4. Structural failures and their reasons

4.1. Introductory information

Searching for reasons of construction work failures is largely based on analysis of specific failure symptoms. Type of damage, its range and location suggest whether the cause is connected with soil, overload or corrosion. Crack pattern and displacement analyses are particularly valuable for inferring the reasons. They are especially clear in the case of concrete and masonry structures. Therefore, mainly such structures are described below. Damage of timber members, as elements of floors or roofs in masonry buildings, is mentioned only.

This chapter discusses failure reasons – subsoil conditions, overload, shrinkage and environmental conditions, and their symptoms,

As mentioned in chapter 2, only proper recognition of failure reasons allows to chose and design relevant repair methods. First symptoms of damage should already be alarming for the user. Careful assessment of the construction work and drawing up of proper conclusions allow to avoid large failures or collapses.

4.2. Failures resulting from subsoil conditions

4.2.1. Failures due to excessive or non-uniform settlement of parts of the construction work

Cracking, displacements and deflection of the construction work may be caused by excessive or uneven settlement of individual parts of the structure. These may result from design errors, defects in execution or improper operation.

Faults in foundation trenches

The most frequent faults occurring during digging the foundation trenches include:

- digging deeper trenches than originally planned by the designer, and filling them up to the foundation level with the dug-out soil, instead of lean concrete or the mixture of sand and cement,
- ignoring the removal of weak soil pockets,
- conducting soil works in the rain, snow or frost without trench protection, causing damage of the original soil structure.

The above-mentioned faults lead to greater than predicted if not non-uniform settlement of parts of the construction work.

Settlement caused by weak or non–homogeneous subsoil

Some relatively frequent settlement-related design errors come from inaccurate identification of subsoil conditions, especially soil parameters and thickness of its layers, as well as the level of underground water. This may lead to designation of an insufficient foundation area, or designing continuous footing instead of plates or piles. In the case of heritage buildings, such situation is not caused by design errors but rather by the lack of awareness of the builders. Laying of foundations not matching the weak or non-homogeneous subsoil may result in tilting (Fig. 49) or cracking of the construction work (Fig. 50).

Fig. 49. The 'tilting house' in Toruń, Poland – an example of non-uniform settlement

Cracks in a wall may also appear in the case of non-uniform foundation, an example of which is presented in Fig. 51 [25]. The corner of this building is founded partly on clay subsoil and partly on the older masonry work. This caused the appearance of cracks crossing the wall along its entire height.

A good design practice is to assume the width of footing in relation to its load, so that stresses under all foundations are of similar value. If this rule is not observed, this may also lead to non-uniform settlement of the foundations.

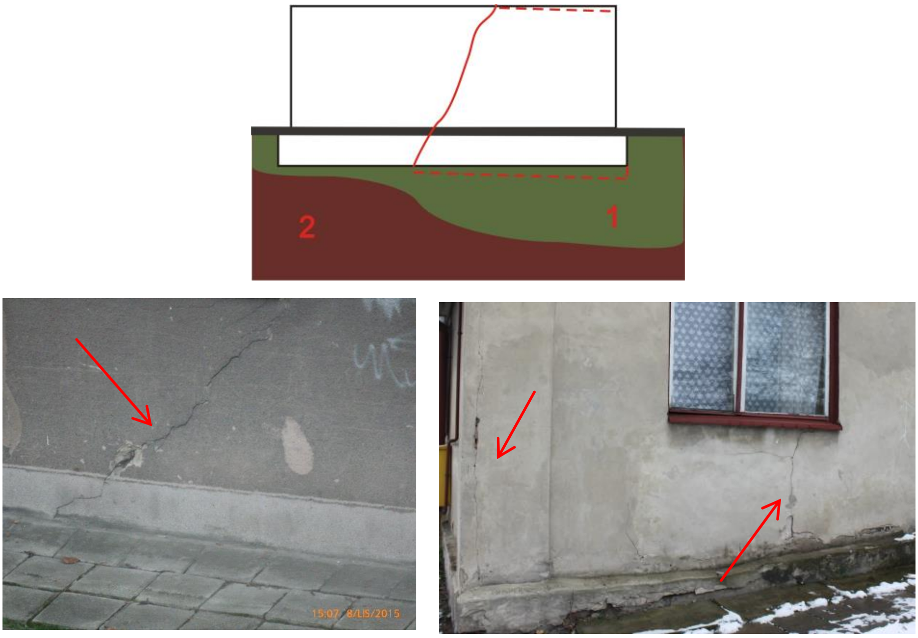


Fig. 50. Cracks in a masonry wall: top – a sketch of cracks caused by settlement of the foundation lying on a thick layer of weak subsoil (1 – weak soil, 2 – soil of good parameters); bottom – photographs of cracks caused by settlement of the building on weak deformable subsoil

Non-observance of the construction rules

The next group of foundation-related errors is connected with construction gaps. There are certain rules governing the division of a building or structure into structural parts. A construction work, including its foundations, should be divided into parts with the gaps, if parameters of the subsoil under specific parts of the building are significantly different, and if the height, load or construction type is different in different parts of the building. If these rules are not observed (the designer did not plan any gaps or the gaps were not made carefully enough), different settlement of non separated parts may cause cracks in the entire structure in the zones where gaps should have been designed (the structure will fall into parts by itself).

Similar damage may occur when the foundation level is not the same under the entire building and the design provides for significant differences in foundation levels (e.g. the cellar is designed only under some part of the building). Transition between foundation levels should be gradual. Otherwise, a crack dividing the structure may emerge above the region where levels change rapidly.

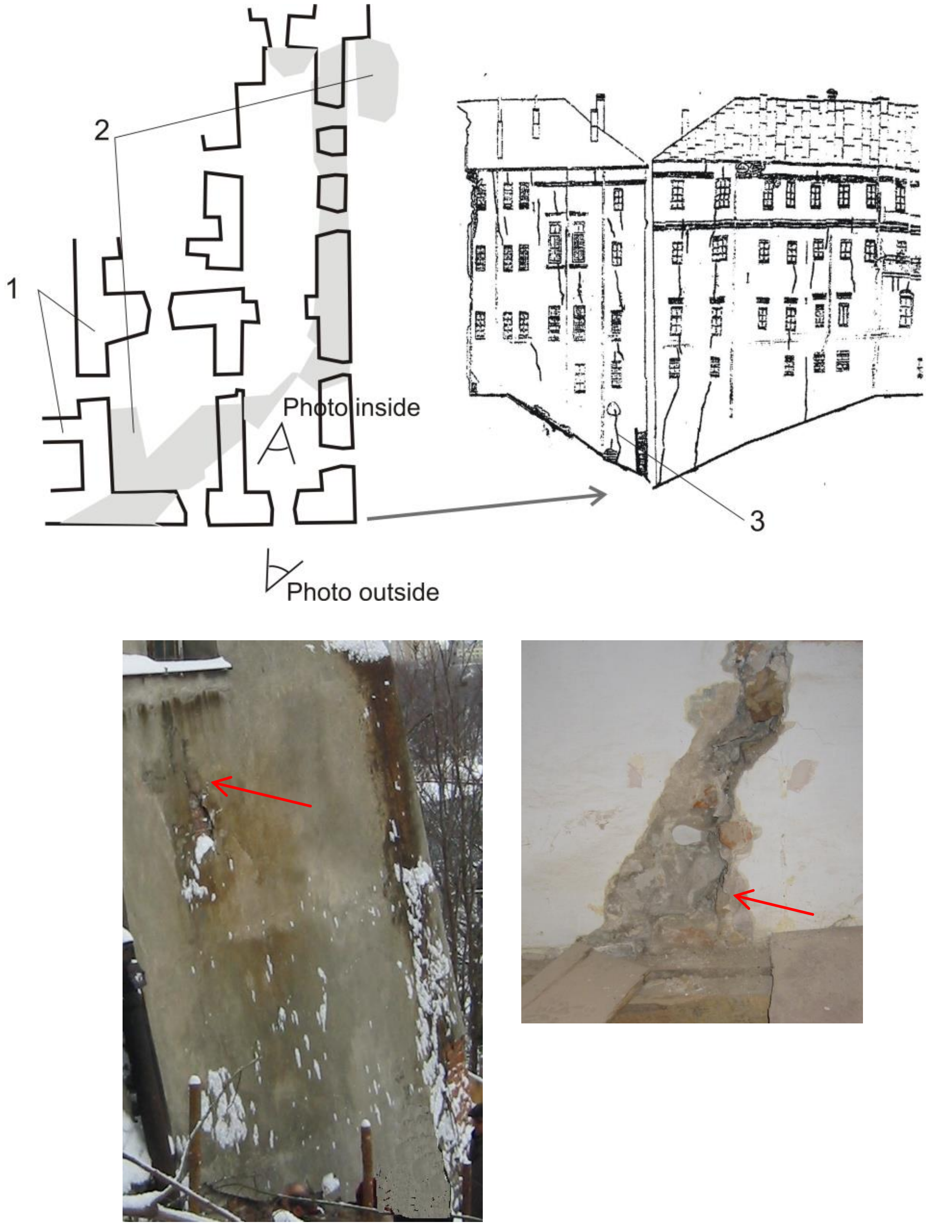


Fig. 51. Cracks in the corner wall of building of the Dominican Monastery in Lublin: top – the projection of the first floor (1 – walls in the first floor, 2 – older masonry work) and the cracks pattern (3 – crack shown in bottom photos), bottom – one of the cracks cutting through the corner walls (view outside on the left, view inside on the right)

4.2.2. Errors in foundation works near existing buildings

Strict rules have been developed for conducting the foundation works near the existing buildings. Continuous foundations should be unearthed by stages. It should be divided into parts of no more than 1.0 - 1.5 meters. The parts unearthed at the same time should not adjoin each other and the total length of the unearthed foundation should not exceed 20% of the total foundation length.

When the above requirements are not met, the soil may be forced out from the edge zone under the footing, producing cracks in the walls perpendicular to the unearthed foundation (Fig. 52) if not lead to the collapse of the wall supported by the uncovered foundation.

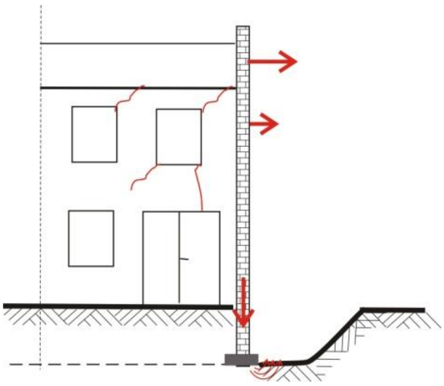


Fig. 52. Cracks resulting from soil displacement from the edge zone under the foundation due to uncovering the foundation along its full length

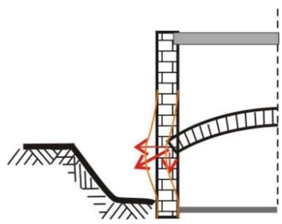


Fig. 53. Deflection of the uncovered foundation wall supporting the vault due to unbalanced thrust

If the foundation needs to be uncovered along its entire length, the excavation site must be protected by the retaining wall with ground anchors or straining beams, in order to avoid the above failure.

Uncovering of the foundation wall supporting the vault is particularly dangerous. The unbalanced thrust produced by the vault may cause deflection of the wall (Fig. 53) and cracking or even collapse of the vault.

4.2.3. Change in subsoil conditions during use

Changes in subsoil conditions while using a building or structure are most often caused by water: fluctuations in underground water level or the flowing of water into the subsoil. Flowing water may come from damaged pipes of the water supply or sewage system located near the building. Water can also flow from the improperly arranged and maintained surface of the ground around the building.

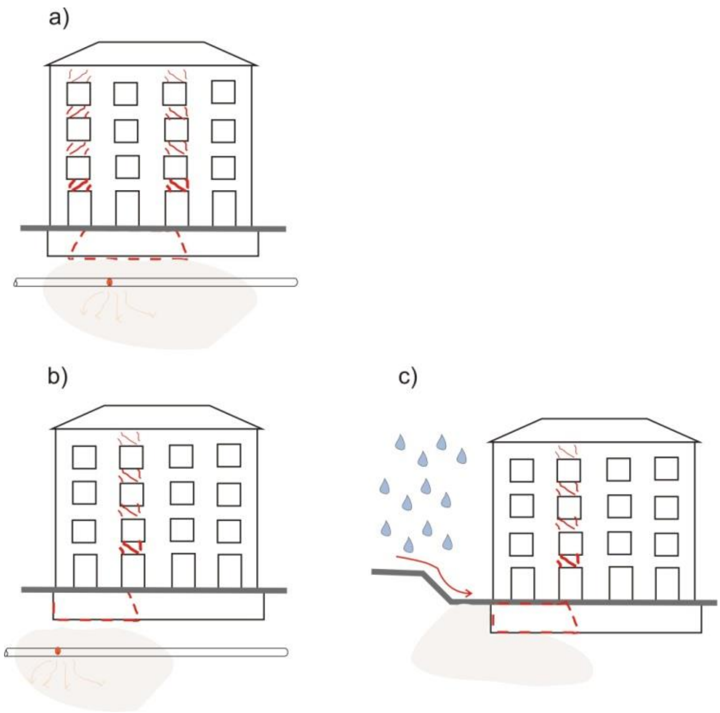


Fig. 54. Cracking of walls near the windows due to water flow and washing out of the soil from beneath the foundations: a) cracking due to pipe failure near the central part of the building, b) cracking due to pipe failure near the corner of the building, c) cracking due to flow of rain water from the slope located near the building

Fig. 54 presents crack patterns in a wall with windows and doors, caused by water flow. Crack pattern in a wall without windows is shown in Fig. 55.

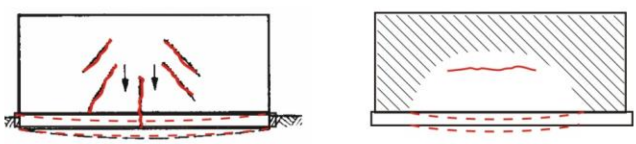


Fig. 55. Possible crack patterns due to washing out of the ground from beneath the foundation in a wall without any openings: a) washing out along the entire length of the foundation (deflection of the wall together with the foundation), cracks are vertical and inclined; b) partial washing out and creation of the compressed "arch" based on the non-deflected part of the foundation (hatched area); cracks are horizontal

4.2.4. Other foundation faults

Faults during execution may cause corrosion damage of the foundations. When the reinforcement is placed directly in the soil, without any layer of lean concrete, there is no concrete cover of sufficient thickness and the risk of reinforcement corrosion occurs. The similar deterioration may occur when the foundation is not protected against underground water penetration by any insulation layers put on foundation surfaces.

4.3. Failures of reinforced concrete structures

4.3.1. Errors in design and execution of reinforced concrete members

The main reinforced structure failures resulting from design errors include:

- choosing an inappropriate model for structural analysis,
- neglecting some loads or actions in structural calculations, especially shrinkage and thermal actions in young concrete,
- neglecting concrete creep and prestressing steel relaxation in structural calculations,
- failure to verify the limit states in transient design situations; this especially applies to pre-cast structures (stages of transport, assembly of pre-cast members) and pre-stressed structures,
- failure to verify the limit states in fire situations and design of excessively thin concrete covers in terms of fire and environmental class,
- calculation errors,
- errors in arrangement of the reinforcement (too short anchorage, too great spacing),
- errors concerning pre-cast members: too short supports of beams and plates, neglecting the support moments in calculations of partially fixed members,
- lack of expansion gaps.

The main execution errors affecting the reinforced structure failures are:

- inattentive and incorrect arrangement of the reinforcement (diameters, spacing, anchorage length or concrete cover thickness different from design assumptions), incorrect position (e.g. ‘downward’ reinforcement position in the cantilever instead of the ‘upward’ position),
- use of concrete of insufficient strength, discharging the concrete mix from considerable height, which results in segregation of its components, incorrect consolidation or treating, incorrect preparation of concrete surface after technological break,
- non-observance of technological rules while placing concrete in cold weather (frost) conditions,
- deformation of the formwork that is not rigid enough, or too prompt removal of the formwork, resulting in preliminary member deformations,

- applying the load onto too young concrete,
- inattentive assembly of pre-cast members resulting in member defects, inattentive arrangement of joints between pre-cast members.

While using reinforced concrete structures, the following damage may occur:

- excessive cracks and deflection if not collapse due to overload,
- deterioration due to environmental conditions,
- mechanical failure caused by accidental load.

4.3.2. Crack patterns due to load

Introductory information

The first sign of excessive load of a reinforced concrete structure are cracks of excessive width.

One should note that cracks are something absolutely normal in reinforced concrete structures. They appear when internal stresses exceed the tensile strength of concrete. When a crack appears, the load is carried by the reinforcement ‘bridging’ the crack. Cracks having a greater width than specified in the code for concrete structures design may suggest that load is excessive in comparison with bearing capacity of the reinforced concrete member. Ultimately, they may lead to a structural member collapse. Therefore, appearance cracks of significant size should induce a detailed assessment of the member.

In concrete structures, there usually appears a single wide crack causing their rapid collapse. Reinforced concrete structures may have many cracks and their collapse in by no means rapid. The less reinforcement there is, the more rapid the collapse. A quick and rapid collapse and the accompanying sharp explosion-like sound is also characteristic for members made of high-strength concrete.

Cracks in tensioned members

In tensioned members, cracks are perpendicular to the applied force direction. They are in the regular spacing (along the stirrups) and go across the member (Fig. 56).

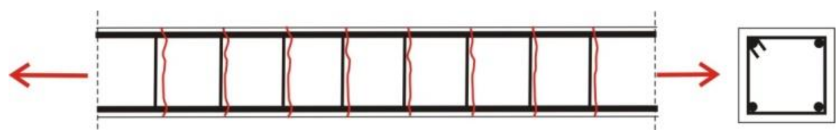


Fig. 56. Cracks in a tensioned reinforced concrete member

Cracks in compressed columns

Three types of cracking in axially compressed columns may be distinguished:

- cracks due to buckling – they are horizontal and entail crushing of concrete located in column mid-height on the opposite side of the cross-section when the load achieves the critical value (Fig. 57),
- concrete cover of the reinforcement split-off is caused by buckling of bars due to excessive spacing of the stirrups,
- cracks due to overload are located in the mid-height area and differ in slender vs. low columns: in a low column, they are parallel to the column axis and in a slender column they are slant (in a reinforced concrete column they are directed at an angle of about 45 degrees in relation to the column axis; in a concrete column without reinforcement, the angle is smaller) (Fig. 58).

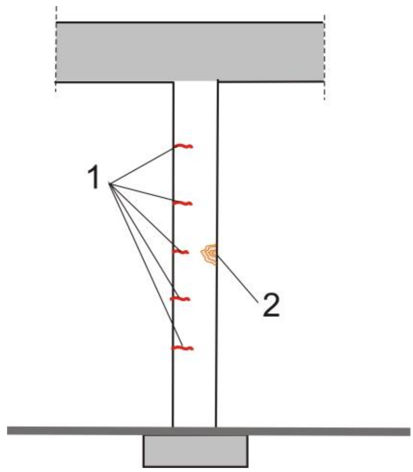


Fig. 57. Crack pattern in the case of column buckling: cracks (1) and crushing of concrete (2)

In eccentrically compressed columns being members of reinforced concrete frames, the crack pattern depends on the type of supports in column ends. Cracks appear in the tensioned zone. If the bottom end is fixed in the foundation, the cracks are concentrated near the upper joint with the beam (Fig. 59–right). If the bottom end is simple supported, the cracks are swept towards the bottom edge (Fig. 59–left). When the load achieves the bearing capacity value, concrete on the internal side of the upper part of the column gets crushed.

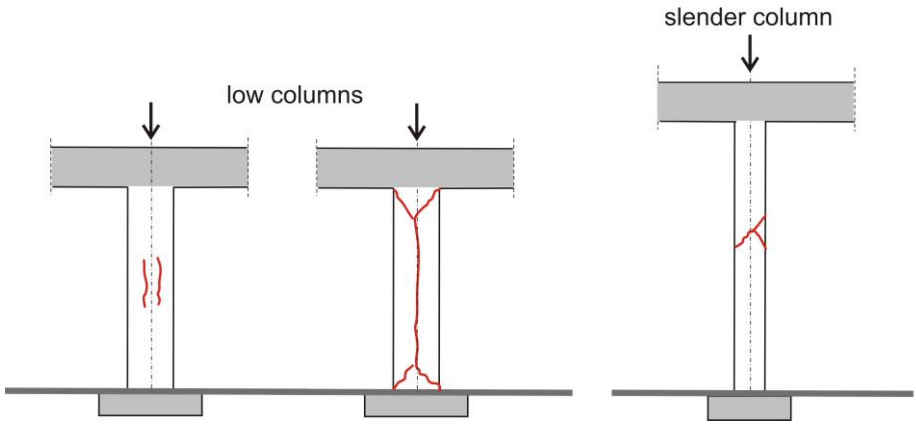


Fig. 58. Crack patterns due to load in axially compressed concrete and reinforced concrete columns

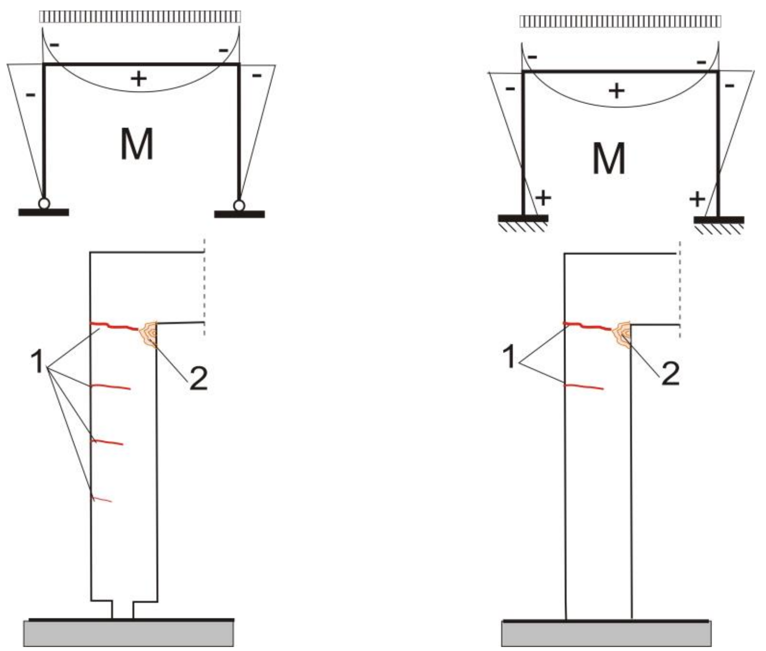


Fig. 59. Distribution of internal moments in the frames under vertical load (top) and crack pattern in reinforced concrete columns being parts of these frames (bottom): column simply supported on the foundation (left) and column fixed in the foundation (right): 1 – cracks, 2 – crushing of concrete

Cracks in reinforced concrete slabs

In slabs, cracks are generated mostly due to flexure in the zones of the largest internal moment. Therefore, in one-way slabs, cracks are parallel to the supports. They appear in the upper surface of the slab near the supports and in the bottom surface of the slab in the spans (Fig. 60).

In two-way slabs, cracks are located along the bisector line of the slab corner. In the middle of the slab, they cross each other and cracks parallel to the supports emerge, especially if the slab is rectangular (Fig. 61). In the simple-supported slabs, cracks may emerge in the corners are possible when there is no proper reinforcement (Fig. 62).

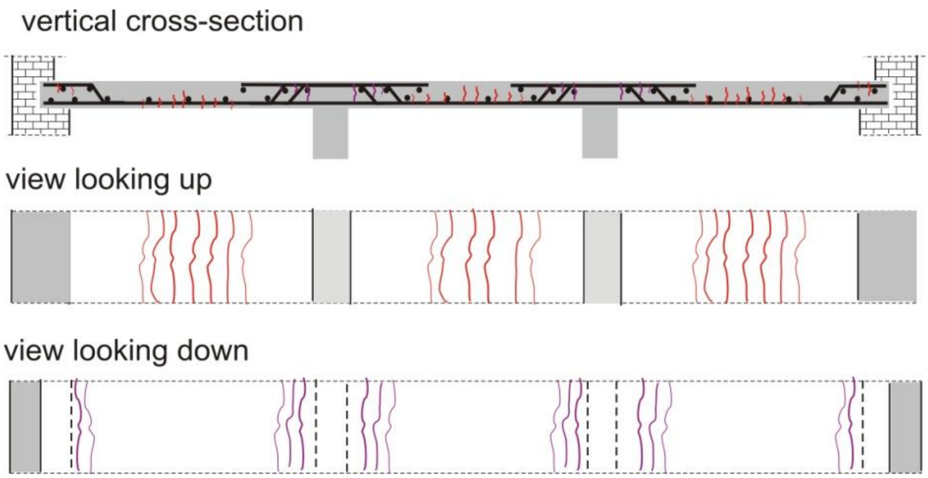


Fig. 60. Crack patterns in a one-way slab: slab cross-section and different views (view looking up presents bottom cracks and the view looking down presents upper cracks)

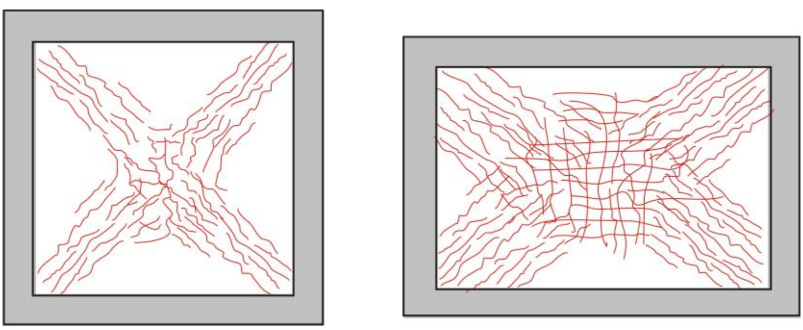


Fig. 61. Crack patterns in two-way simply supported slabs (view looking up): square slab (left) and rectangular slab (right)

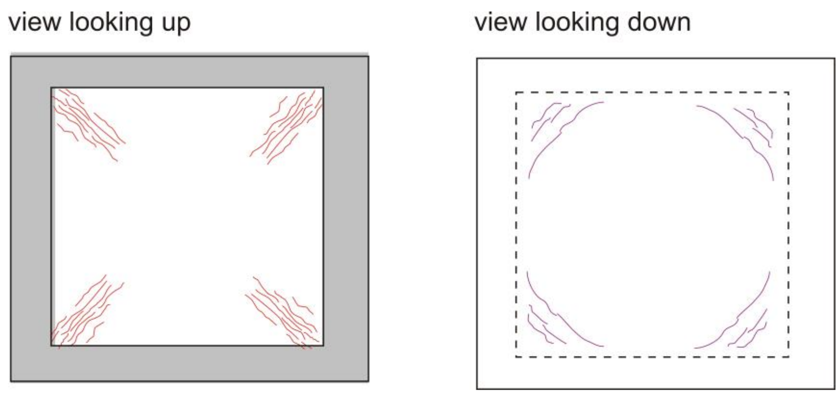


Fig. 62. Crack patterns in the simple-supported two-way slabs without corner reinforcement

Flat slab floors may be damaged due to punching shear. Relevant crack patterns are presented in Fig. 63.

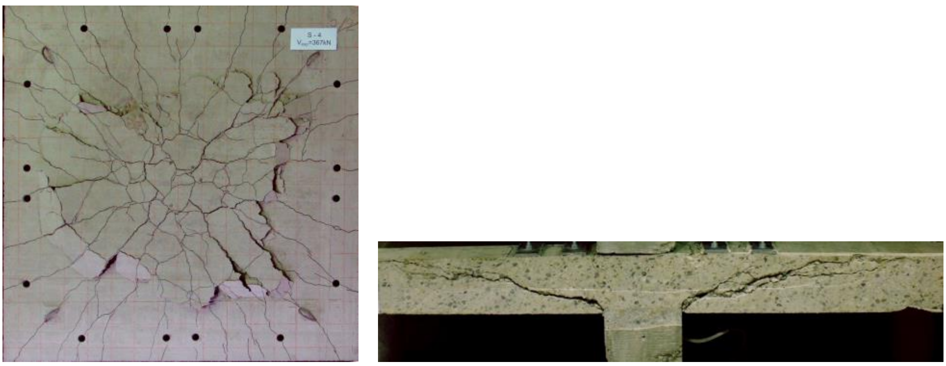


Fig. 63. Crack patterns emerged in flat slab floor due to punching shear [43, 44]]: upper cracks in the view looking down (left) and the zone cross section near the column (right)

Cracks in reinforced concrete beams

In reinforced concrete beams, cracks are caused by flexure (bending moment) and shear (transverse force), as shown in Fig. 64 and Fig. 65. Flexural cracks are perpendicular to the beam axis and they are located in the mid-span zone. Shear cracks are slant and located near the supports. The greater the influence of the transverse force (close to the support), the lesser the angle in relation to the beam axis.

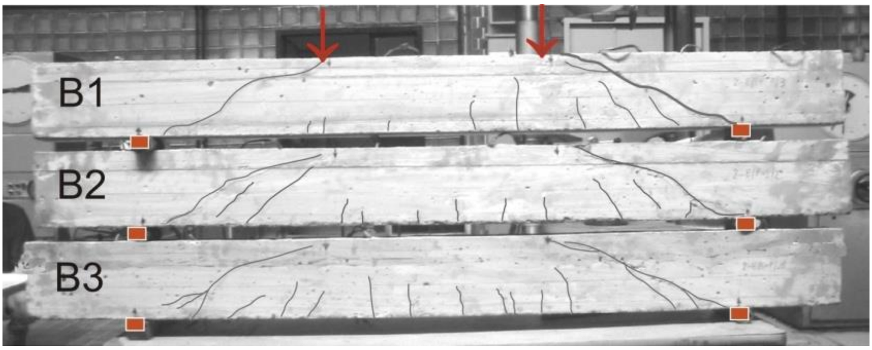


Fig. 64. Three beams after testing under four-point bending

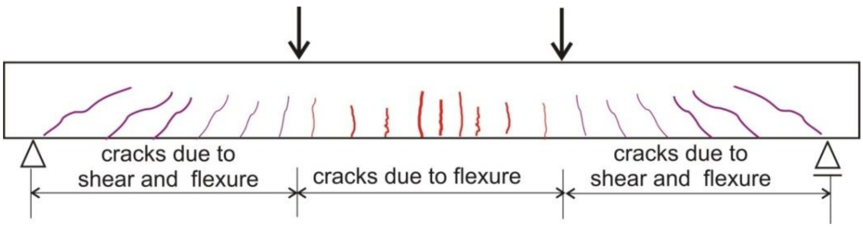


Fig. 65. Crack patterns of the free-end beam loaded with two concentrated forces

4.3.3. Crack patterns due to shrinkage and fluctuations in temperature

Shrinkage is an inherent property of concrete during curing and in the first month of concrete structure's life. If shrinkage strain is not restricted, the volume of the element decreases and no stress is generated (Fig. 66–top). However, the length of a shrunk member with fixed ends must remain the same as its length right after concreting. Shortening of concrete is therefore compensated by cracks. They appear when the tensile stress caused by shrinkage exceeds the tensile strength of concrete (Fig. 66–bottom).

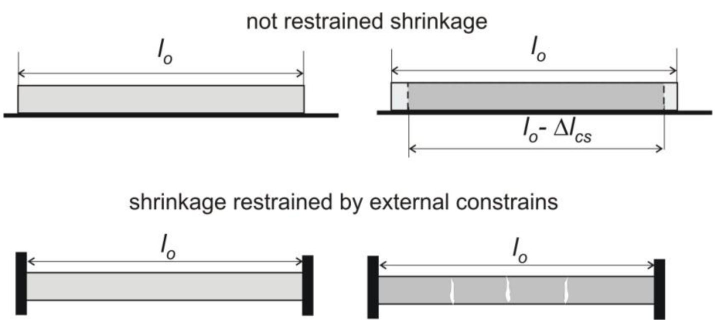


Fig. 66. The idea of non-restrained and restrained concrete shrinkage

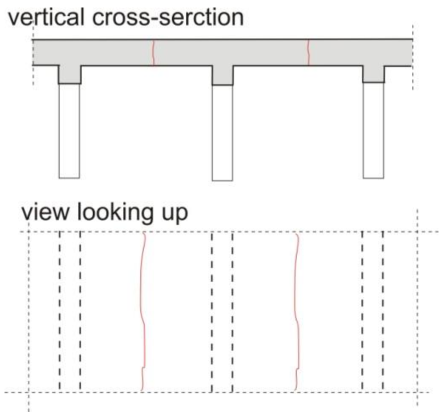


Fig. 67. Cracks due to shrinkage in a one-way slab

In slabs, shrinkage-related cracks going across the entire slab are usually located in the mid-span, parallel to the main reinforcement (Fig. 67). In two-way slabs, cracks may be parallel to one of the supporting beams or even may be somewhat irregular (Fig. 68).

In beams, shrinkage stress is an extra phenomenon to load-related stress. Therefore, shrinkage cracks are located in the cross-sections where the internal forces are the greatest (Fig. 69): in flexure-dominated beams, shrinkage cracks appear in the mid-span; in shear-dominated beams, shrinkage cracks are located near the supports. In slightly reinforced beams, shrinkage cracks go across the entire beam (Fig. 69b), in strongly reinforced beams on the other hand, there are more, shorter and narrower shrinkage cracks and they are situated between the bottom and the top reinforcement (Fig. 69c).

Fig. 68. Shrinkage cracks in the garage floor that came to light after leakage: 1 – cracks in a two-way slab, 2 – cracks in the supporting beam

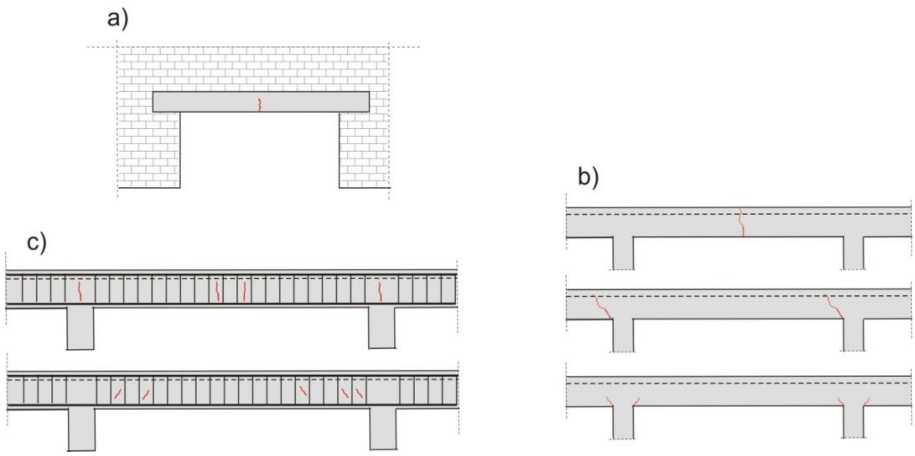


Fig. 69. Cracks due to shrinkage in beams: a) the beam above window opening, b) slightly reinforced beams, c) strongly reinforced beams

In shields and plates making the walls, the crack pattern depends on the number and arrangement of restrained wall edges, as well as on the stiffness of restricting members, e.g. previously executed foundation slab, adjacent walls or segments (Fig. 70).

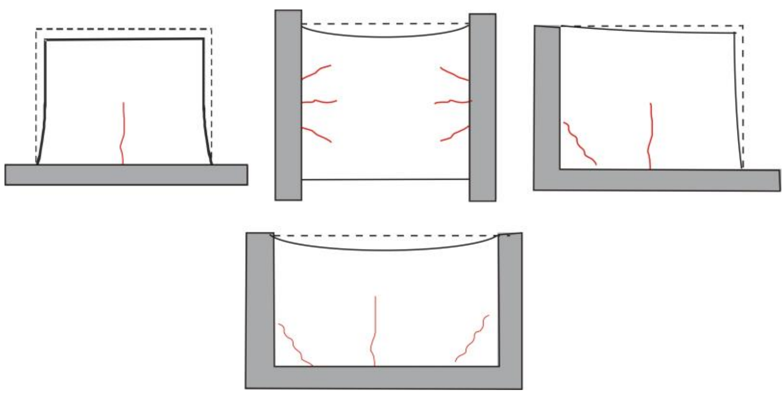


Fig. 70. Crack patterns in walls with differently restrained edges [S7]

In massive structures (retaining walls, foundation walls, bridge heads and tank walls), there is an additional factor influencing tensile stress and cracks. Concrete is self-heated as a result of hydration of cement; temperature of a massive structure may increase up to a few dozens degrees. Afterwards, young concrete begins to cool down and this effect is added to shrinkage effects Fig.71).

Fig. 71. Cracks going across a massive foundation wall that emerged a few days after concreting due to shrinkage and temperature variations connected with self-heating of concrete

Fig. 72. Cracks due to non-uniform early shrinkage

In members with only one surface exposed to sun, dry air and wind, non-uniform shrinkage occurs. The near-surface zones are subjected to greater shrinkage, as opposed to the inner parts. This produces tension near the surface and cracks in these regions. In reinforced concrete members, cracks are usually located along the reinforcement mesh (Fig. 72). The same mechanism is responsible for foundation cracking (Fig. 73).

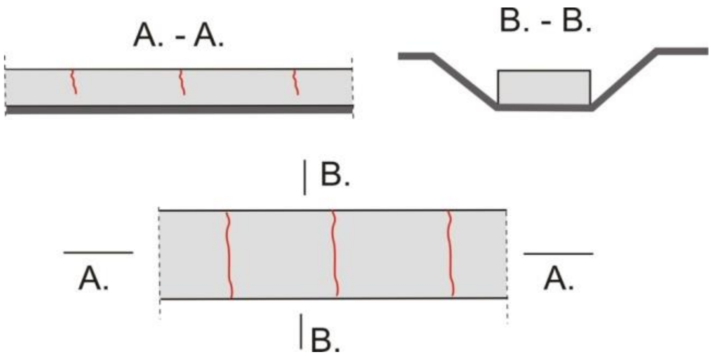


Fig. 73. Cracks due to non-uniform shrinkage of concrete foundation: cross-sections and a view 'down'

Cracks observed after some period of use may result from the joint action of shrinkage and thermal stress (Fig. 74). The latter occurs when a structural member is subject to changes in external temperature, and the precautions were not designed or were improperly executed. Relevant precautions involve designing the expansion joints (breaking the continuity of the concrete structure and enabling elongation of the divided parts within the expansion gaps) or providing for stronger reinforcement.

The cracks may occur due to large technological thermal stress, e.g. in a chimney designed with insufficient hoop reinforcement (Fig. 75).

Fig. 74. Thermal cracks in structural members without expansion gaps: long attic wall (left) and long cantilever roof (right)

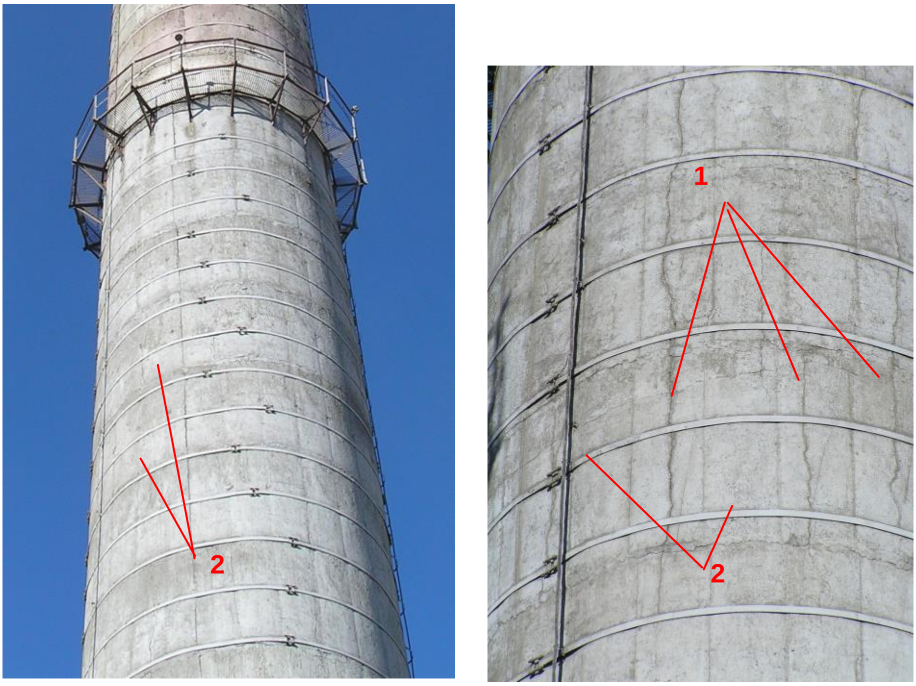


Fig. 75 The reinforced concrete chimney with vertical cracks due to thermal stress (1), repaired by steel hoops (2)

4.3.4. Damage due to environmental and operational factors

Overloading of a reinforced concrete structure causes cracking as described in chapter 4.3.2. This may lead to failure and collapse, if the load is great enough.

In this chapter, attention is paid to corrosion deterioration. Non-compliance with the requirements regarding preservation of the reinforced concrete structure against the aggressive environment leads to concrete and steel corrosion.

The major requirement is to design and produce a concrete cover for the reinforcing bars that is sufficiently thick. An excessively thin concrete cover subjected to carbonation or exposed to aggressive agents (sulfate ions, acids, ammonium etc.) may cause steel corrosion, especially if chloride ions are present. Steel corrosion products result in the ‘swelling’ of bars and concrete cover split-off. This can lead to more robust steel corrosion and reduction of effective bar diameter, and subsequently, to reduced carrying capacity of the member.

An excessively thin concrete cover may be a design error (wrong determination of the exposure class or non-compliance with code rules regarding cover thickness in relation to exposure class) or workmanship carelessness, which in more extreme cases may lead to absence of concrete cover fragments.

Fig. 76. Concrete cover faults: fragments without a concrete cover due to careless workmanship (top), split-out concrete cover due to advanced bar corrosion process (bottom)

Pictures of a thin concrete cover are shown in Fig. 76. A similar situation in water tanks and silos is illustrated in Fig. 83 and in Fig. 86 respectively.

Concrete deterioration may result from freeze/thaw conditions. In winter, water penetrating the concrete via cracks or micro-cracks freezes, subsequently enlarging its volume and bursting the concrete surface and after some time even deeper concrete layers (Fig. 77).

Fig. 77. Deterioration due to freeze/thaw attack: top - destruction of the surface zone, bottom - damage to the foundation for the heat pipe (left) and terrace slab (right)

White smudges and stalactites are the sign of the washing out of calcium carbonate from the grout (Fig. 78). Deterioration of the internal structure of concrete consisting in destruction of grout in between the remaining coarse aggregate, may be caused by sulfate attack (Fig. 79). In rooms with high humidity, yellow stalactites may appear as well.

Structural deterioration is often caused by the lack of protection, not only against aggressive environment, but also against mechanical factors. Fig. 80 presents the frame construction of an open-air coal store. The damaged column is not protected against the bulldozer impact and abrasion due to coal being moved around.

Fig. 78. Indications of calcium carbonate being washed out from the grout: white smudges and white effluents from cracks (top), white stalactites hanging from the cracks (bottom)

Fig. 79. Deterioration due to sulfate attack in a coal and coke store

Fig. 80. The column damaged due to bulldozer impacts and movement of coal (concrete split-off, broken and deformed longitudinal bars)

4.3.5. Specific damage to concrete liquid tanks

Liquid tanks are construction works sensitive to design, execution and operation faults and they are highly susceptible to damage [22]. Damage is most commonly manifested as cracks emerging due to leakage. The cracks may come to light during tightness tests or after some period of use. It is leakage that usually precludes normal use and makes the user commission tank assessment and repair. Cracks may result from exceeding the bearing capacity of structural members, but most often they are caused by not complying with tightness requirements. Other, most common types of damage include: excessive displacement of walls, especially next to thermal joints, and corrosion deteriration.

Design errors

Based on literature data gathered in [22], the most frequent design faults may be listed as follows:

- assuming the structural scheme or FEM model that improperly reflects the actual tank structure and its performance (mainly neglecting the 3D character of a tank and assuming wrong types of supports) results in obtaining wrong values and distribution of internal forces, and consequently, leads to insufficient wall thickness or inadequate amount of the reinforcement,
- assuming improper load values (e.g. assuming improper liquid level, neglecting the weight of technological devices, neglecting the combination of

filled chambers in a multi-chamber tank) also results in obtaining wrong values and distributions of internal forces and leads to insufficient wall thickness or too small reinforcement cross-section,

- neglecting or improper interpretation of the imposed actions: shrinkage and fluctuations in temperature in young concrete or subsequent thermal actions (see chapter 4.3.3, see also the example in Fig. 84 and the text below),
- neglecting the tightness analysis and errors in cracking analysis results in excessively thin walls or insufficient amount of the reinforcement,
- designing the reinforcement not complying with the minimum reinforcement conditions: excessive rebar spacing, too large rebar diameters, insufficient amount of the reinforcement,
- faults in arrangement of the reinforcement, too small lap length, excessively thin concrete cover of the bars in relation to the exposure class,

Fig. 81. Examples of tank damage in expansion joints (view down): horizontal displacement of divided wall parts perpendicular to the wall (left) and joint opening (right); 1 – sewage inside the tank, 2 – concrete wall, 3 – displacement perpendicular to the wall, 4 – the outside of the tank, 5 – opening of the joint

- faults in the foundation (e.g. neglecting the dependence between the structure and subsoil performance or assuming the improper subsoil model, assuming soil parameters departing from the actual ones, neglecting the deformability of weak soils, mixing different types of foundations in one tank) result in non-uniform settlement, sliding or rotation of tank parts,
- faults in the arrangement of expansion joints: neglecting expansion gaps or excessive gap spacing where designing of expansion gaps is justified, improper division of vast or long tanks by expansion gaps without analysing the consequences, neglecting the preservation against the displacement of divided tank parts (Fig. 81), improper sealing of gaps and breaks in concreting.

Execution errors

The most frequent execution errors may include [22]:

- changes in wall or bottom slab thickness in relation to the design objectives,
- improper preparation of the subsoil [8],
- bad quality concrete: reduced concrete strength in relation to design objectives, improper composition of the concrete mix in relation to tightness or low demands regarding hydration heat, non-uniform consolidation, improper treatment,

Fig. 82. Cracking in a prismatic water tank (left) and the reason of cracking – bars located close to the internal surface of the wall are bent without proper anchorage

- faults in the reinforcement:
  - change of steel type, diameter and bar spacing or position of bars in the cross-section (even swapping the internal and external reinforcement) in relation to design objectives,
  - laps of too small length,
  - bending of the bars located near internal surface of the wall without proper anchorage in the corners of prismatic tanks (Fig. 82),
  - excessively thin concrete cover (Fig. 83),
- negligent or not careful execution of the expansion joints assumed in the design, especially inadequate placement of the sealing tapes in expansion joints,
- leakage next to the elements stabilizing the formworks,
- lack of corrosion protection due to incorrectly matched coatings, linings and flashings or their poor quality.

Fig. 83. Roofs of tanks for drinking water [24]: left – reinforced concrete dome with corroded bars uncovered after concrete cover split-off, right – beam-and-slab floor with signs of steel corrosion and bars uncovered after concrete cover split-off

Damage due to operation

Tanks may be damaged during use due to [22]:

- change in liquid parameters (specific gravity, temperature) and liquid level (e.g. due to failure of different devices) causing increase in internal forces,
- change in subsoil conditions: continued use of a leaking tank (change in soil parameters, frost heave and washing of the soil), change in underground water level, mining damage),
- concrete and steel corrosion due to aggressiveness of liquid (chemical compound with sulfates or chlorides), high humidity (Fig. 83) and frost,
- abrasion of concrete surface due to sloshing of the liquid or movement of the movable parts of devices contacting the surface.

An example of a cracked water tank

An example of a cylindrical tank cracked before operation due to shrinkage and fluctuations in temperature of young concrete, described in detail in [23] is shown in Fig. 84. It has been used for sewage processing in a biogas plant. The wall was divided into six vertical segments about 12 meters long, with segment height equal to tank height. The segments were then subsequently concreted in two turns. At each turn, every second segment was concreted. The time intervals between subsequent segments concreting were about one week.

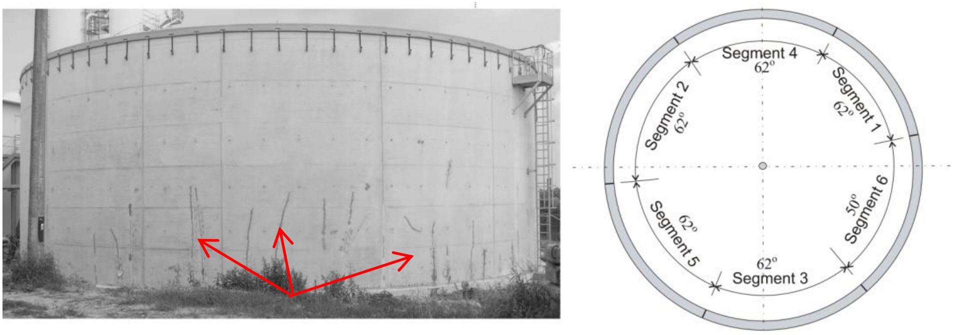


Fig.84. The tank with the wall divided into segments during concreting [23]: left - tank with cracks that appeared during tightness testing, right – division of the wall into segments

The bottom slab was cast in August, first-stage segments (No. 1–3) were executed in September, whereas segments of the second stage (No. 4–6) – at the end of September and in October. The CEM III/A 42.5N low heat cement was used for preparing the concrete mix. The concrete was treated for one week by sprinkling with water. The tightness test was carried out in May the following year. The cracks that appeared are shown in Fig. 84 and Fig. 85.

In order to identify the crack causes, FEM analysis of the tank at 7 days after concreting of segments was performed. Temperature fluctuations connected with self-heating of concrete (depending on segment massiveness), fluctuations in ambient temperature (based on meteorological data) and the average value of shrinkage strain (calculated in accordance with EC2-1-1 and modeled as cooling of the segment) were taken into account. The strength of concrete at the 7th day was estimated based on the actual concrete strength. Therefore:

- first-stage segments were modeled as concrete walls fixed in the bottom plate on the elastic subsoil of Winkler's type; the following temperature changes were assumed: segment cooling (due to shrinkage, cooling after self-heating and cooling of the surrounding air), bottom slab cooling (due to cooling of the surrounding air);
- second-stage segments were modeled as concrete walls fixed in the bottom plate rested on the elastic subsoil of Winkler's type, located between two first-stage segments, that were already cured; the following temperature changes were assumed: cooling of the first stage segment (due to shrinkage, cooling after self-heating and cooling of the surrounding air), cooling of second-stage segments and the bottom slab (due to cooling of the surrounding air).

It was stated that the main tensile stress in the bottom parts of the segment, both in the first-stage (about 6 MPa) and second-stage segments (about 4.5 MPa) exceeds the tensile strength of concrete at the 7th day. In the first-stage segments, this takes place near the edges, in the second-stage segments – in the middle and near the edges. This confirms the actual reason of crack emergence and allows to formulate a rule for designers of tanks cast in stages: static analyses for each segment during tank execution are a must.

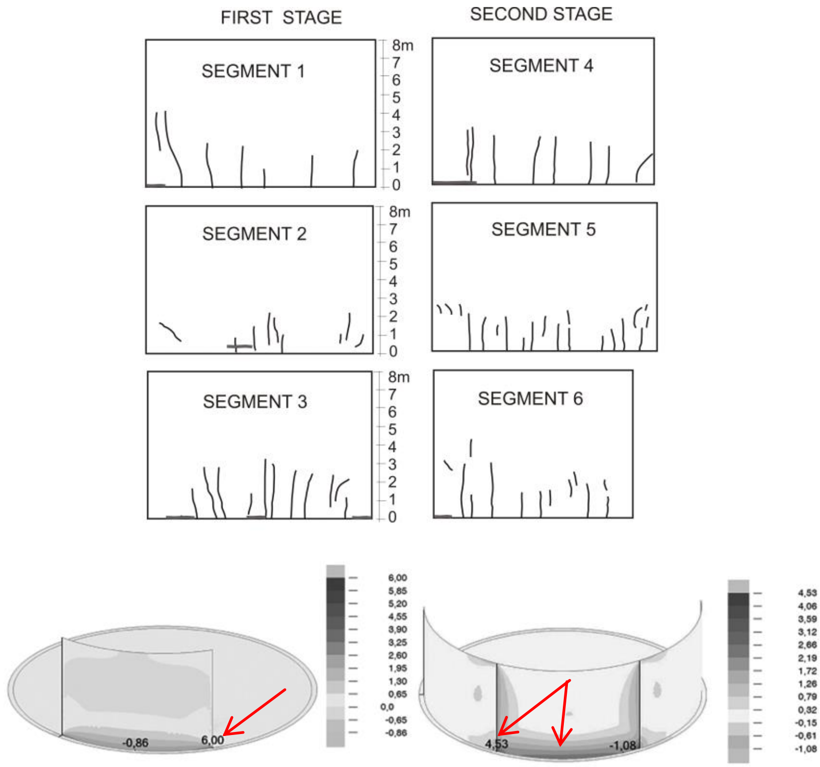


Fig. 85. The concrete tank cast in stages [23]: crack patterns of all segments (top) and maps of stress in the first-and second-stage segments, appear due to shrinkage and fluctuations in temperature of young concrete

4.3.6. Specific damage to concrete silos

Silos are designed to store particulate solids having different properties. Slender and intermediate slender silos are mainly used to hold powder and small-grain solids, whereas squat silos (bunkers) are dedicated for solids having larger particles.

The actual technical condition of a concrete silo is the result of large loads (also of dynamic nature), large dimensions and difficult work conditions on the one hand, as well as workmanship quality, used materials and structural solutions and preservation measures on the other. According to [8], the frequency of silos failures, especially of slender silos, exceeds the frequency of failures of other industrial structures. The literature presents a number of examples: conspicuous vertical cracks of several mm in width [16]), gaps in the chambers or in huge fragments of the elevators due to dust explosion [29], tilted chambers due to their non-uniform settlement [11, 29].

Design errors

A lot of serious silos failures occur due to adoption of too low values of pressure exerted by particulate solids in the calculations. It should be emphasized, that in the past they were not always the designer's fault, but they resulted from the lack of general knowledge about physical phenomena during filling, emptying and other technological operations in silos, and their influence on the pressure exerted onto the walls and the bottom.

Fig. 86. Typical damage of silo walls due to insufficiently thick rebar concrete cover

Errors at the design stage may include:

- errors in the static analysis:
  - failure to analyze all design situations, especially failure to consider various configurations of filled chambers in the case of multi-chamber silos, which leads to underestimation of the bending moments,
  - ignoring the bending moments, which may occur due to local effects or asymmetrical filling or emptying of a silo,
  - ignoring shrinkage and fluctuations of temperature in young concrete,
  - ignoring thermal actions: rapid cooling of air (causing the increase in pressure), temperature differences in different parts of the wall in the case of filling the silo with hot medium,
  - lack of analysis regarding differences in settlement between operation tower and filled chambers (especially during initial filling),
  - neglecting the dynamic actions generated in the neighboring structural members,
- lack of thermal insulation where it is needed,
- errors in arrangement of the reinforcement:
  - giving only the number of rebars and spaces between them in the drawings without specifying the thickness of the concrete cover and rules of lapping, may lead to faulty workmanship (Fig.86),
  - insufficient reinforcement around the technological openings in the walls,
  - too small spaces between bars (preventing proper concrete consolidation) or between prestressing tendons (preventing proper covering by shotcrete).

Fig. 87 Damage of outer surfaces of bunker hoppers due to excessively thin concrete cover of rebars

Faulty workmanship

Faulty workmanship arises from:

- departure from the original assumptions or careless execution as regards dimensions or location of the structural members and their supports,
- careless location of the reinforcement:
  - too large spacing, too small diameters, in relation to the original design,
  - excessively thin concrete cover (Fig. 86, 87) causing rebar corrosion and concrete cover split-off ,
  - too short laps or arrangement of all laps along one vertical line,
- poor quality concrete work (careless consolidation, treatment, use of movable formworks) leading to lack of uniformity (empty spaces, honeycombs) and low concrete strength,
- poor quality linings and insulation layers.

Fig. 88. A crack going through the silo wall, filled by the user on the spot with glue

Operational faults

The main operational fault is silo use contrary to design objectives, especially: storing media having different parameters than originally assumed, non-compliance with the storage procedure, e.g. medium humidity, emptying the silo using unserviceable aeration devices (Fig. 88), willful installation of vibrators facilitating the emptying without dynamic analysis, striking by the means of transport). Other damage include:

- abrasion of wall and hopper surfaces by particles of the material, or stroking the walls with large pieces of stored materials during filling and emptying the silo (Fig. 89),

- concrete and steel corrosion due to frost, aggressive chemical agents, carbonation (Fig. 86 and 87), which is intensified in the case of excessively thin concrete cover,
- dust explosions (Fig. 90) or fires.

Fig. 89. Damage to internal surfaces of coal bunker walls due to abrasion and striking by coal pieces

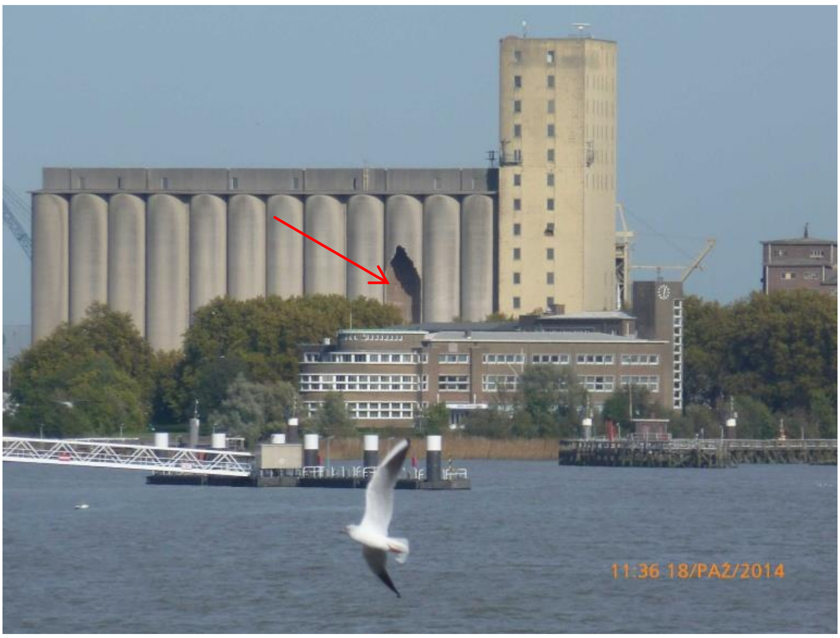


Fig. 90. The elevator with one chamber visibly damaged due to dust explosion

4.4. Failures of masonry structures

4.4.1. Failures of masonry walls

Compilation of reasons

Failures of masonry structures may result from:

- excessive or non-uniform settlement of their foundations (described in chapter 4.2),
- design calculation errors, e.g. regarding load specification, neglecting the slenderness effects for walls or pillars,
- defects in workmanship,
- thermal stress and shrinkage of adjacent concrete members,
- lack of three-dimensional rigidity of the building,
- overload,
- operational reasons,
- environmental reasons (dampness and salinity).

The above reasons are described below in greater detail.

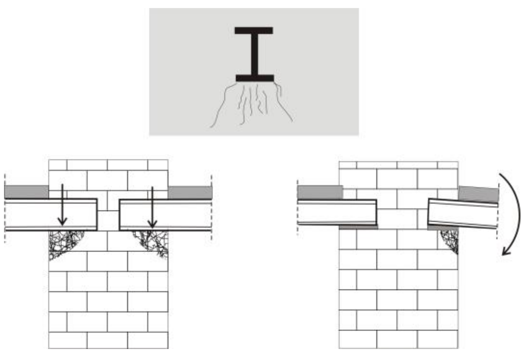


Fig. 91. Crushing of a masonry wall under the floor beam: top – view of the wall surface; bottom – wall cross-sections in: beams supported directly on brick without any sleeper (left) and beam loaded with a moment and support of insufficient length (right)

Defects in workmanship

The main defects in workmanship include:

- poor quality or improper arrangement of bricks, hollow bricks or other masonry elements,
- mortar of insufficient strength,

- excessively thick or excessively thin layer of mortar, or not filled mortar joints,
- cutting out of new openings, chases or gaps in a wall, without guaranteeing the sufficient load carrying capacity and stability,
- faults in anchoring the beams in the walls: too short anchorage (especially if a moment is applied to the beam – Fig. 91 on the right), placing the beam directly on bricks without any sleeper – layer of mortar or concrete or a steel washer – Fig. 85 on the left).

Thermal and shrinkage stress

When expansion gaps are ignored by the designer or contractor, cracks in masonry structures may appear. They may be caused by shrinkage of adjacent elements – reinforced concrete tie or beam above the window, as shown in Fig. 92.

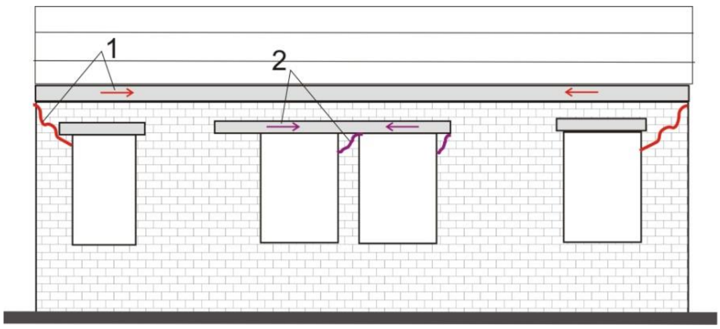


Fig. 92. Cracking in a masonry wall due to shrinkage of the reinforced concrete tie (1) and the beam above the window (2)

When a designer or contractor ignores the expansion gap around the roof structure inside the outer wall, a crack may appear above the last floor (Fig. 93). In summer, roof temperature may exceed 50^{o}C, especially due to application of dark roofing paper or roofing sheet. This leads to roof expansion, when compared to winter conditions. The expansion is greater in length than in width; the longer the roof, the greater the expansion. The gable wall (perpendicular to building length) restraining the expansion is forced out (Fig. 93 top). This is accompanied by cracking of the longer wall (Fig. 93 bottom).

In the upper edge of the wall, vertical regularly spaced cracks may appear. This is caused by the wall's expansion in summer and shrinkage in winter due to seasonal variations in roof temperature. This phenomenon consequently applies to the wall edge as well. Such cracking often occurs in historic buildings where walls are not secured with ties or bowstrings against thermal movement (Fig. 94).

Fig. 93. Cracking as a result of ignoring the expansion gaps around the roof, inside the outer wall above the last floor: forced out gable wall (top), cracked ends of the same longitudinal wall (bottom)

Fig. 94. Vertical cracking of the upper edge of the wall under the roofing

In walls containing ventilation or smoke ducts, cracks may occur at duct edges due to temperature differences in the masonry and the ducts (Fig. 95).

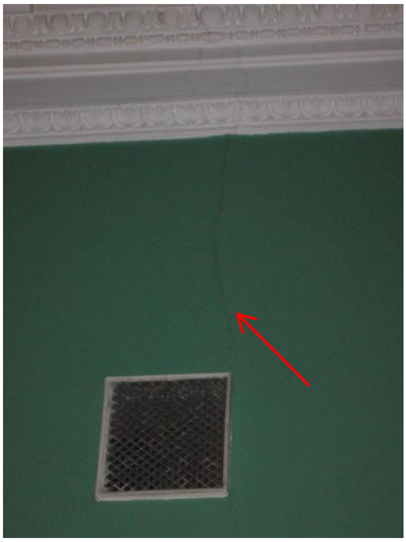


Fig. 95. Crack in a masonry wall at the edge of the ventilation duct

Fig. 96. Cracks in the facing layer of a three-layer wall (in the right-hand photo – the crack is prepared for repair)

In walls built of three layers, the proper arrangement of the facing layer in order to protect the insulating layer against environmental agents is very important. Anchors connecting the facing layer and the bearing layer should be used, especially in wall corners. Vertical and horizontal expansion gaps should be performed, in accordance with code requirements. Otherwise, cracks in the facing layer may appear (Fig. 96).

Lack of three-dimensional rigidity of the building

Any construction work should have a three-dimensional rigidity ensured by strutting and connections between the walls in the corners. Rigidity should be warranted by reinforced concrete ties, bowstrings or beam anchors.

In the absence of strutting, cracks may occur in the gable wall due to unbalanced thrust of the vault or roof truss, as presented in Fig. 97.

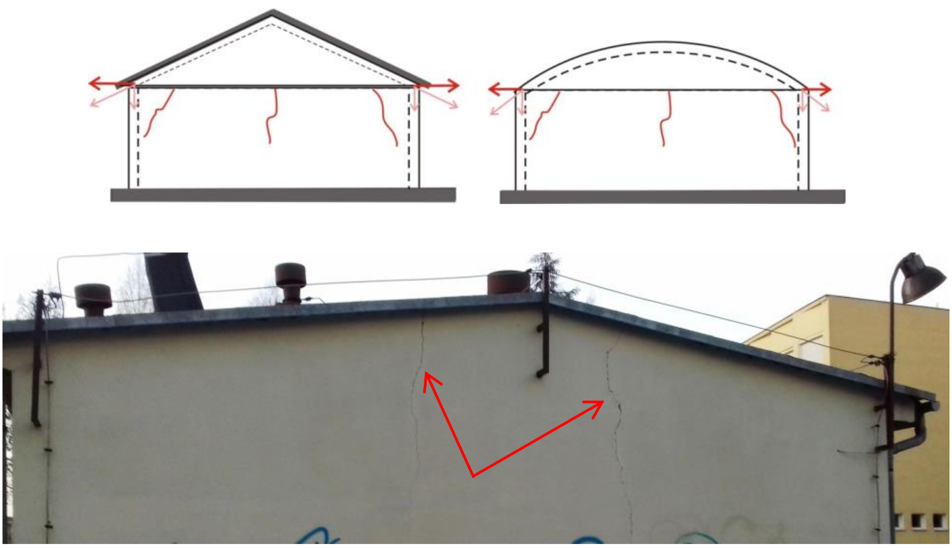


Fig. 97. Cracks in a gable wall due to unbalanced thrust of roof truss and vault

Operational reasons

Damage during operation may arise from changes in loading or change in the support conditions.

Changes in the structural loads usually follow from the change in use, e.g. a flat is adapted into a library or a shop. This may result in wall overloading, especially of the window pillars; consequently, the pillars may crack or even collapse. The crack pattern depends on pillar slenderness (Fig. 98).

Changes in support conditions may be caused by cutting of a new opening or widening of the existing ones, without securing the part of the wall above such an opening. This results in cracking or even collapse of the non supported part of the wall (Fig. 99). Floor (slab or beam) deflection changes the support conditions in partition walls situated on this floor. The wall behaviour depends on it rigidity (rigid walls are presented in the Fig. 100 and the not rigid ones in the Fig. 101). Crack patterns in not rigid wall depend on its length (Fig. 101). When the openings are present the cracks usually emerge in the neighbourhood of their corners (Fig. 103).

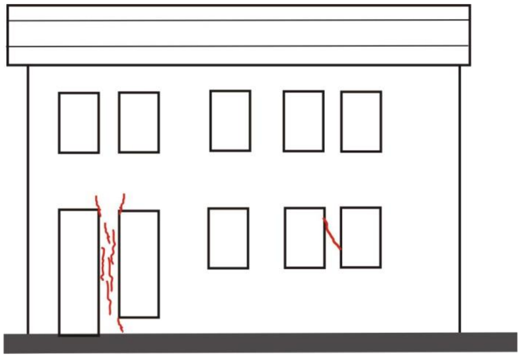


Fig. 98. Cracks in slender and low pillars between openings due to overload

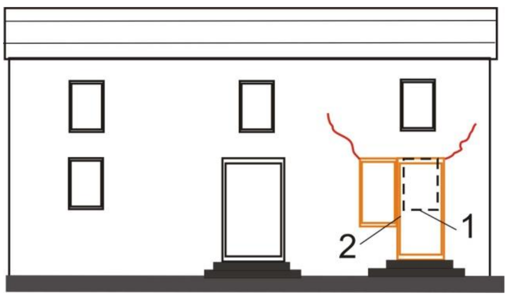


Fig. 99. Cracks caused by widening of the existing opening without securing the part of the wall above: 1 – existing opening, 2 – new opening

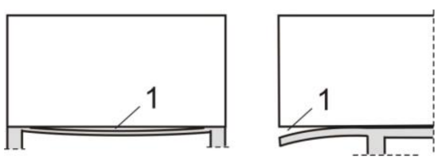


Fig. 100. Gaps (1) emerged between the deflected beam or slab and the short rigid wall

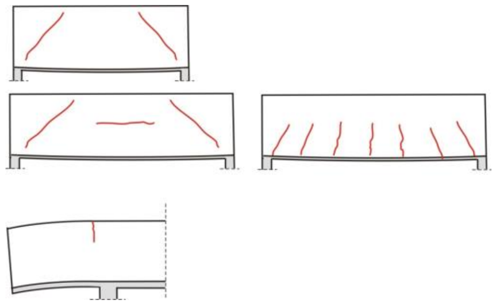


Fig. 101. Crack patterns due to deflection of the slab or beam under long or not rigid wall

Fig. 102. Cracks in a wall situated on wood beams

Fig. 103. Cracks near the door in a wall situated on a reinforced concrete deflected slab

4.4.2 Failures of arches and vaults

Arches and vaults are intended to ensure compression of possibly the largest part of the cross-section (ideally an axial compression). In practice, due to arch shape and the support conditions, bending moments exist as well, causing tension in a part of the cross-section.

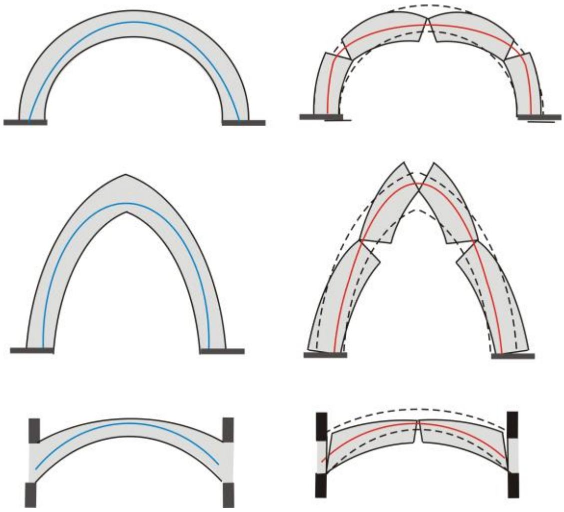


Fig. 104. Static work of arches of different height: on the left – the optimum 'compression line' (blue), on the right – the 'compression line' (red) resulting in cracks and rotation of separated arch parts

Fig. 105. An arch with key bricks falling out due to 'compressive stress line' moving downwards

While analyzing vaults and arches, the traditional notion of 'compression line' is used. It is a line including points where the resultant force of internal compressive stress is applied to particular cross-sections [30]. The location of this line depends on vault shape, as shown in Fig. 104

The optimum is reached if this line goes through the mid-zone of the cross-section's height (zone width equals 1/3 of the cross-section's height). This ensures compression across nearly the entire cross-section (Fig. 104 – left). Otherwise, tension occurs, which means that arch may crack or even bricks fall out of the it (Fig. 104–right, Fig. 105).

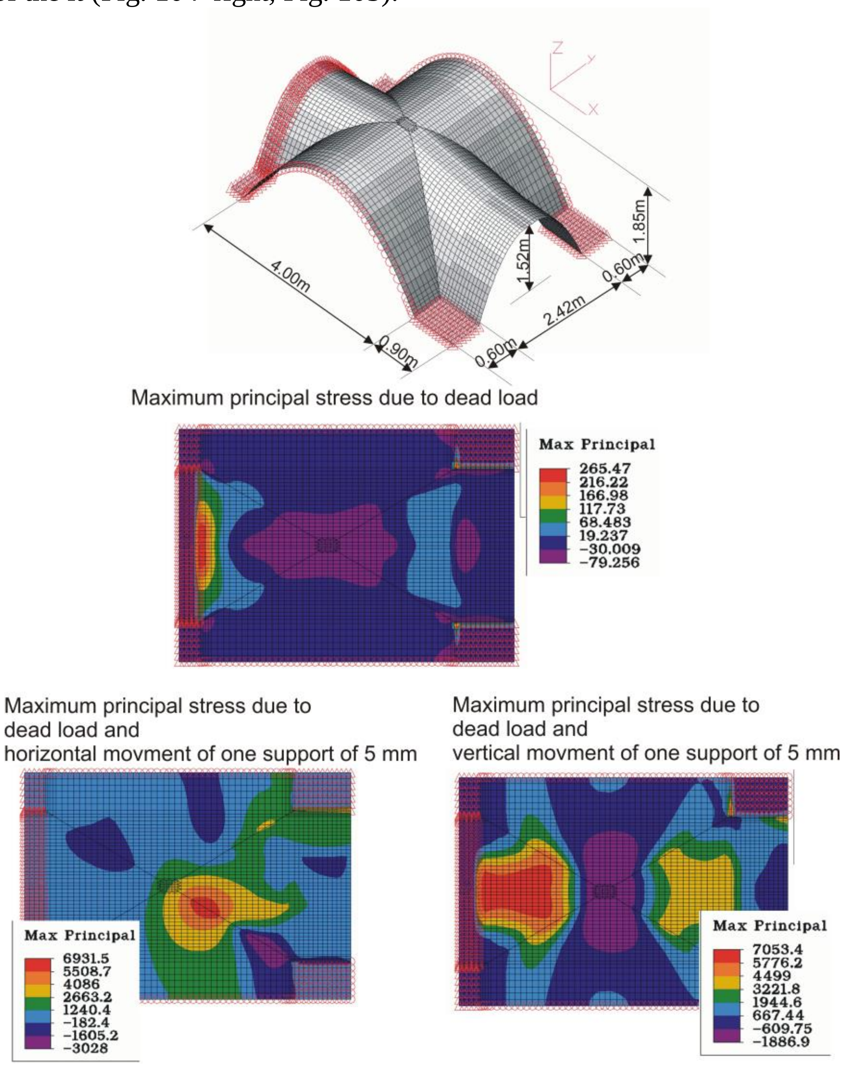


Fig. 106. FEM maps of maximum principal stress in the cross vault (values are given in kPa)

Fig. 107. Cracks in the cross vault due to movement of the supports

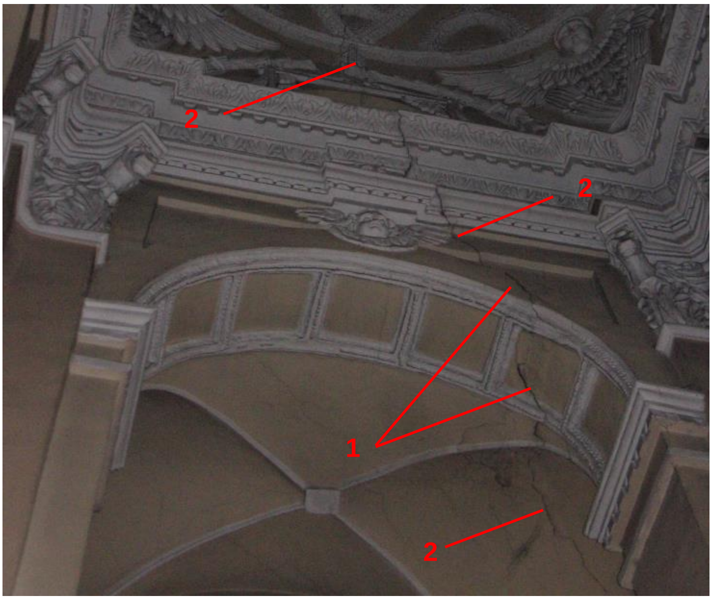


Fig. 108. Arch cracks (1), cracks in the wall above the arch and in the adjacent vault (2) caused by movement of the support

Nevertheless, the usual reason of vault and arch failure is not overloading but moving of the supports – columns or walls. Horizontal movement of the supports results usually from an unbalanced thrust. Vertical movement of the supports occurs due to settlement of the foundations. Fig. 106 shows maps of the maximum principal stress in the cross vault, supported by four columns, obtained from FEM analysis in the elastic ‘stress-strain’ state. Maximum tensile stress due to dead load (self-weight and backfill) is 265 kPa. This value is much lower than tensile strength of the mortar, and cracking is not possible. On the other hand, the maximum tensile stress assuming horizontal movement of one support is 6932 kPa, whereas assuming vertical movement of one support – 7053 kPa. These values are far greater than tensile strength of the mortar, thus giving reasons for vault cracking.

Examples of cracked arches and vaults are presented in figures from 107 to 109. Arch cracks and deformations may result in cracking of the above parts of walls or of the adjacent vaults (Fig.108 and Fig.109).

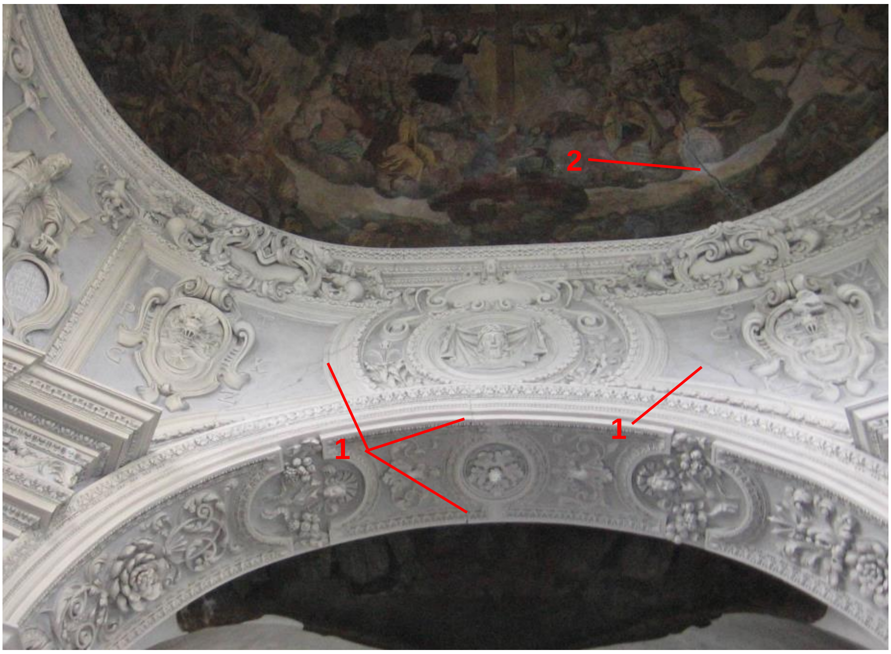


Fig. 109. Cracks in the wall and dome above the arch (2) as a result of arch cracks and deformation caused by loading (1)

4.4.3. Damage caused by the environment and the surroundings

The environment is the source of moisture, water and chemical substances likely to penetrate masonry walls.

Water in down parts of walls lacking horizontal insulation is the source of capillary action. Upper parts of the walls, upper floors and vaults are damp due to leakage of the roofing, gutters or downpipes. Dampness and salinity lead to reduced masonry strength, as well as to mould and fungi formation (Fig. 110).

Freezing-up of water inside the wall and frost bursting, is dangerous for the masonry especially under tight and rigid facing, e.g. cement-based plaster which is burst by frost (Fig. 111).

Fig. 110. Environmental factors affecting the masonry: vault dampness due to roof leakage and vault cracks (top), and dampness and salinity due to capillary action as a result of lack of horizontal insulation (bottom)

Fig. 111. Cement plaster chip off due to freezing of water present in the masonry, resulting in masonry damage

4.5 Failures of timber structures

Failures of timber structures are only mentioned here and the information is limited to traditional structures being members of floors and roofs in masonry buildings.

Failures of traditional timber structures may be caused by design errors as well as faults in workmanship and operational faults.

Design errors include:

- assumption of inadequate static models in calculations, neglecting eccentricities resulting from a given arrangement of joints,
- ill matched solutions regarding joints and contact zones between timber members, especially improper arrangement of nails and fasteners, or too small washers.

The most frequent faults in workmanship are the following:

- using timber of lower class than set out originally in the design,
- use of damp, not completely dried timber, which results in longitudinal shakes (Fig. 112),
- lack of insulation between timber members and masonry, especially at the ends of wood beams in wall pockets,
- neglecting the preservation of wood against biological corrosion and fire.

Faults in operation include overload (e.g. application of unplanned loads) and most often, inadequate maintenance resulting in biological corrosion: appearance of pest (Fig. 113) and dampness, mould and fungi formation (Fig. 114).

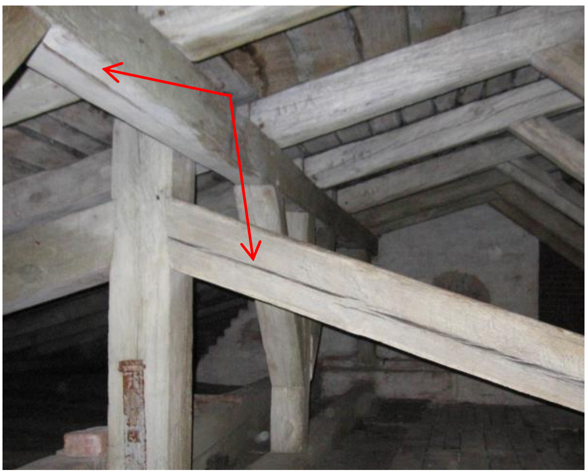


Fig. 112. Longitudinal cracks in a wood beam due to the use not completely dried timber

Fig. 113. Damage of timber members caused by pest

Fig. 114. Damage of timber members due to dampness

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Awarie konstrukcji betonowych i murowych
Identyfikacja uszkodzeń i przyczyn

Anna Halicka, Marek Grabias

Streszczenie

Książka poświęcona jest diagnostyce konstrukcji żelbetowych i murowych, które uległy uszkodzeniom. Jako podstawę dla doboru prezentowanych treści autorzy przyjęli stwierdzenie, że wybór efektywnego sposobu naprawy czy wzmocnienia i bezpieczne użytkowanie obiektu po awarii są uwarunkowane prawidłowym rozpoznaniem jej przyczyn. Dlatego szczegółowo opisano działania zmierzające do uzyskania odpowiedzi na pytania o typ i zakres uszkodzeń oraz ich przyczynę. Ponadto przedstawiono obrazy różnych uszkodzeń konstrukcji wraz z wywołującymi je przyczynami, uznając takie przyczynowo-skutkowe zestawienie za pomocne w diagnostyce konstrukcji.

Zawarte w książce treści oparto na publikacjach naukowych i zaleceniach aktualnych norm, a także na doświadczeniu w działalności eksperckiej pierwszego ze współautorów. Drugi ze współautorów prezentuje najczęściej stosowany sprzęt diagnostyczny.

W rozdziale pierwszym dokonano klasyfikacji przyczyn awarii budowlanych oraz przytoczono przykłady największych katastrof budowlanych w historii. Następnie rozważono zagadnienie awarii budowlanych jako problemu, który musi być brany pod uwagę podczas projektowania, wykonywania i użytkowania konstrukcji budowlanych. Podano najważniejsze pojęcia związane z zapewnieniem niezawodności i trwałości na wyżej wymienionych etapach „życia” konstrukcji. Podkreślono, że problem awarii budowlanych jest także problemem naukowym, czego przejawem jest wprowadzenie dyscypliny naukowej pod nazwą „patologia budowli”.

Rozdział drugi poświecony jest istocie diagnostyki konstrukcji budowlanych. Rozpoczyna się wyspecyfikowaniem celów, jakim służyć może ocena istniejących konstrukcji. Następnie rozwinięto zagadnienie diagnostyki realizowanej w przypadku awarii konstrukcji, sporządzając diagram sekwencji zdarzeń i działań związanych z taką sytuacją. Omówiono także ogólny algorytm oceny konstrukcji zawarty w normie ISO 13822.

Rozdział trzeci zawiera szczegółowy opis kolejnych działań eksperta, którymi są:

- analiza istniejącej dokumentacji technicznej obiektu,
- identyfikacja typu i schematu statycznego konstrukcji,
- inwentaryzacja uszkodzeń,

- badania środowiska (warunki cieplno-wilgotnościowe, warunki gruntowe, efekty dynamiczne, sejsmiczne i para-sejsmiczne),
- pomiary deformacji (ugięć, przemieszczeń, odkształceń) i rys,
- identyfikacja parametrów materiałowych za pomocą badań wizualnych i organoleptycznych oraz badań laboratoryjnych (wytrzymałościowych i chemicznych),
- badania wytrzymałości materiałów (niszczące i nieniszczące),
- badania stanu konstrukcji wykonywane metodami nieniszczącymi,
- obciążenia próbne,
- obliczenia wykonywane w celu: potwierdzenia przyczyny uszkodzeń, określenia poziomu bezpieczeństwa uszkodzonej konstrukcji lub zaprojektowania wzmocnienia.

W rozdziale czwartym zestawiono najczęstsze przyczyny awarii konstrukcji budowlanych wraz z opisem i ilustracją graficzną uszkodzeń powodowanych przez te przyczyny. W kolejnych podrozdziałach opisano:

- uszkodzenia budynków wynikające z warunków gruntowych,
- uszkodzenia konstrukcji żelbetowych wynikające z przeciążenia, skurczu i obciążeń termicznych oraz oddziaływań korozyjnych środowiska, w szczególny sposób zajęto się uszkodzeniami zbiorników na ciecze i silosów,
- uszkodzenia konstrukcji murowych, a w szczególności uszkodzenia ścian oraz łuków i sklepień, a także uszkodzenia wynikające z oddziaływań korozyjnych środowiska,
- uszkodzenia konstrukcji drewnianych, będących elementami budynków wykonywanych w technologii tradycyjnej.