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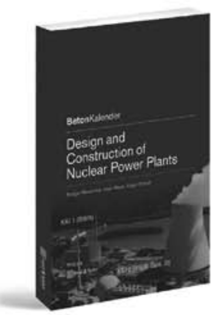

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Design and Construction of Nuclear Power Plants

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Despite all the efforts being put into expanding renewable energy sources, large-scale power stations will be essential as part of a reliable energy supply strategy for a longer period. Given that they are low on CO_2 emissions, many countries are moving into or expanding nuclear energy to cover their baseload supply.

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Design and Construction of Nuclear Power Plants

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Contents

Editorial

The "Concrete Yearbook" is a very important source of information for engineers involved in design, analysis, planning and production of concrete structures. It is published on a yearly basis and offers chapters devoted to various subjects with high actuality. Any chapter gives extended information based on the latest state of the art, written by renowned experts in the areas considered. The subjects change every year and may return in later years for an updated treatment. This publication strategy guarantees, that not only the most recent knowledge is involved in the presentation of topics, but that the choice of the topics itself meets the demand of actuality as well.

For decades already the themes chosen are treated in such a way, that on the one hand the reader is informed about the backgrounds and on the other hand gets acquainted with practical experience, methods and rules to bring this knowledge into practice. For practicing engineers, this is an optimum combination. Engineering practice requires knowledge of rules and recommendations, as well as understanding of the theories or assumptions behind them, in order to find adequate solutions for the wide scope of problems of daily or special nature.

During the history of the "Concrete Yearbook" an interesting development was noted. In the early editions themes of interest were chosen on an incidental basis. Meanwhile, however, the building industry has gone through a remarkable development. Where in the past predominantly matters concerning structural safety and serviceability were in the centre of attention, nowadays an increasing awareness develops due to our responsibility with regard to society in a broader sense. This is reflected e.g. by the wish to avoid problems related to limited durability of structures. Expensive repair of structures has been, and unfortunately still is, necessary because of insufficient awareness of deterioration processes of concrete and reinforcing steel in the past. Therefore structural design should focus now on realizing structures with sufficient reliability and serviceability for a specified period of time, without substantial maintenance costs. Moreover we are confronted with a heritage of older structures that should be assessed with regard to their suitability to safely carry the often increased loads applied to them today. Here several aspects of structural engineering have to be considered in an interrelated way, like risk, functionality, serviceability, deterioration processes, strengthening techniques, monitoring, dismantlement, adaptability and recycling of structures and structural materials, and the introduction of modern high performance materials. Also the significance of sustainability is recognized. This added to the awareness that design should not focus only on individual structures and their service life, but as well on their function in a wider context, with regard to harmony with their environment, acceptance by society, the responsible use of resources, low energy consumption and economy. Moreover the construction processes should become cleaner, with less environmental nuisance and pollution.

The editors of the "Concrete Yearbook" have clearly recognized those and other trends and offer now a selection of coherent subjects which resort under a common "umbrella" of a broader societal development of high relevance. In order to be able to cope with the corresponding challenges the reader is informed about progress in technology, theoretical methods, new findings of research, new ideas on design and execution, development in production, assessment and conservation strategies. By the actual selection of topics and the way those are treated, the "Concrete Yearbook" offers a splendid opportunity to get and stay aware of the development of technical knowledge, practical experience and concepts in the field of design of concrete structures on an international level.

Prof. Dr. Ir. Dr.-Ing. h.c. *Joost Walraven*, TU Delft
Honorary president of the international concrete federation *fib*

Preface

Despite all the efforts being put into expanding renewable energy sources, large-scale power plants will be essential as part of a reliable energy supply strategy for as long as we can see. Given that nuclear power is low on CO_2 emissions and has no competitors when it comes to being operated cheaply, many countries are moving into or expanding nuclear energy to cover their baseload supply. Germany will need its existing nuclear power plants to supply it with cost-effective, reliable energy for many years to come, and the financial power of German utility companies like E.ON and RWE and German design and construction knowhow is helping realise new building projects in neighbouring countries. At home, there are many challenges to be met when it comes to continuously updating existing plant. The authors are extensively involved in designing, operating and inspecting existing plant, designing newbuilds, doing retrofits and conversions and updating specific nuclear power rules.

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1 Introduction

1.1 The demand for energy

As the world's population grows, the demand for primary energy, and hence electrical energy, is growing massively with it. At the same time, the demand for individual electrical power is increasing, especially as the so-called emerging nations are seeing their energy demand soar as they strive to become industrialised. The International Energy Agency (IEA) estimates that businesses and private households will need around 60% more energy by 2030 than they do today. With their massive populations, China and India will account for two-thirds of the increasing demand forecast.

The result, 'More and more people needing more and more energy', is shown in Figure 1.1. Between 2000 and 2020, world population is set to increase from six to eight billion (33%), but the demand for energy is forecast to rise at nearly twice the rate, by around 62%. This increase will mean major challenges in terms of a sustainable energy supply, based on the three-pillar concept of balancing economics, ecology and society, as set at the world summit in Rio de Janeiro in 1992.

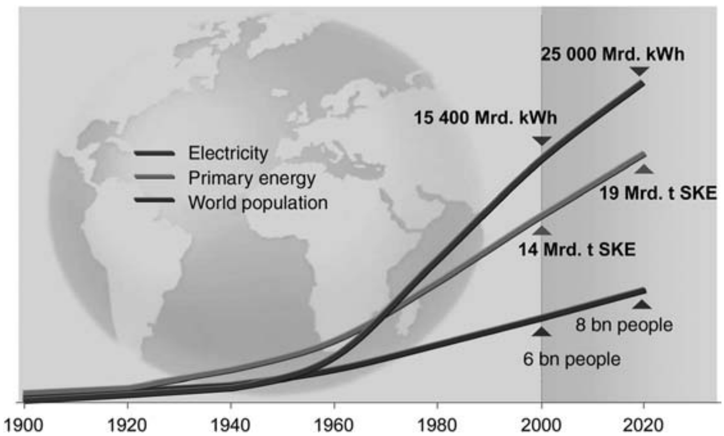

Fig. 1.1 Growth in global energy demand

1.2 Electricity generation

A number of options are available to cover this energy demand: they can basically be divided into two groups, thermal power plants and power plants running on renewable energy sources. Thermal power plants break down into oil-fired, gas-fired, lignite-fired, hard-coal-fired and nuclear power plants. Apart from hydroelectric power, the main renewable energy sources are wind power, solar energy, biomass and geothermal energy.

Different electricity production options are rated differently in environmental and economic terms, but the difference in generating power (a 1000 MW coal-fired power

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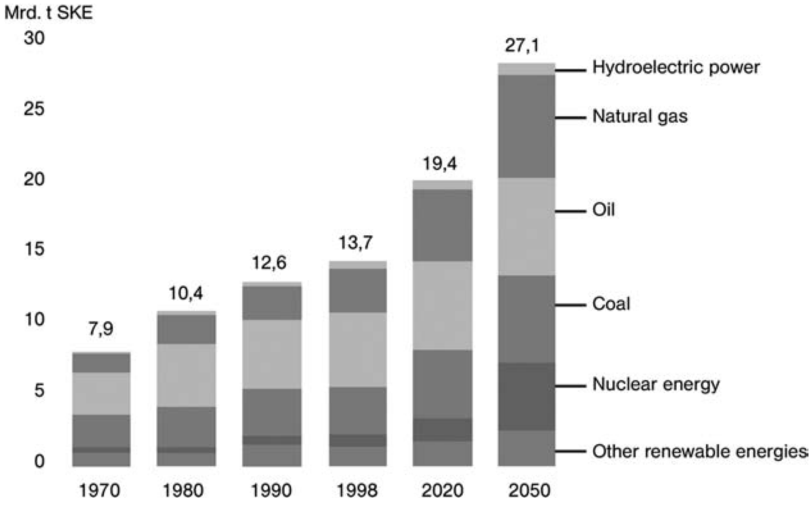

Fig. 1.2 World energy demand and coverage (WEC reference scenario)

plant is equivalent to 200 offshore or 400 onshore wind farms, for example) makes it clear that, essentially, the only way that rising world energy demand will be met is by using powerful cogeneration plants.

As Figure 1.2 shows, even though renewable energies are set to expand enormously in the coming decades, the capacity they actually generate will only grow slightly, so that their percentage share of total electricity generated will actually fall. The forecasts by World Energy Council (WEC) and IEA also say that even after 2020, more than 70% of energy will be obtained from coal, oil and gas, and the share of nuclear power will increase considerably. As well as output, having a reliable electricity supply is also extremely important, which also shows that thermal power plants are essential. If we look at a typical day load curve as in Figure 1.3 – broken down into baseload, average load and peak load – it is clear that thermal power plants are needed for baseload particularly.

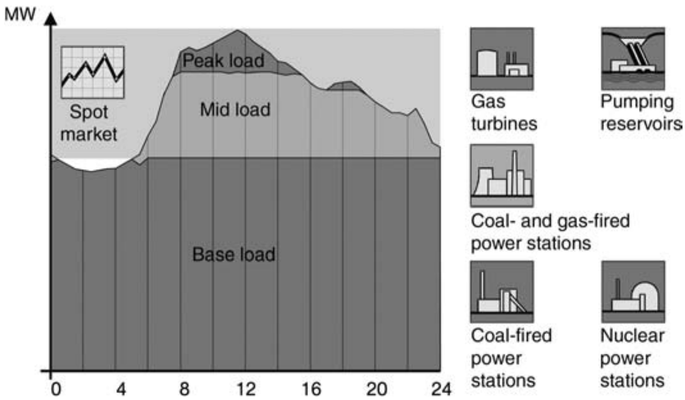

Fig. 1.3 Energy supply load curve

1.3 Importance of nuclear energy

In contributing towards covering world energy demand over a forecast period up to 2050 (Figure 1.2), nuclear energy plays a key role in generating electricity, which will mean a large number of newbuild projects worldwide. As the overview in Figure 1.4 shows, as at autumn 2009, as well as the 437 nuclear power plants already in operation, another 53 new nuclear power blocks were under construction, and another 76 new blocks were planned. The new blocks currently being built or planned mostly have (electricity) outputs from 1000 to 1600 MW. (Please note: the figures given in MW below indicate electrical energy, as opposed to thermal energy, which is stated in MW_{th}.)

Nuclear energy now provides around 15% of the electricity generated worldwide. It avoids around 2.5 bn tonnes of CO_2 emissions, so it makes a major contribution towards a sustainable electricity supply which achieves the goals in terms of economics, capability and the environment to a large extent.

For Germany, which uses wind energy relatively intensively, direct comparison shows that theoretically more wind energy was installed than nuclear in 2008 (23,300 MW as against 21,497 MW), but nuclear generated much more energy than wind, at 148.8 TWh as against 40.2 TWh. In other words, nuclear energy generates nearly 50% of baseload electricity in Germany.

Just how important nuclear energy is can also be seen from how economical it is in generating electricity. Building new nuclear power plants is relatively expensive in terms of capital costs, but the fuel costs involved (uranium), including disposal, are so low that the total cost (including disposal and end stage planning) of generating electricity is around 3–4 Euro cents per kWh [2]. This means that nuclear power is not affected by volatile fuel prices and guarantees a reliable supply, as the uranium deposits

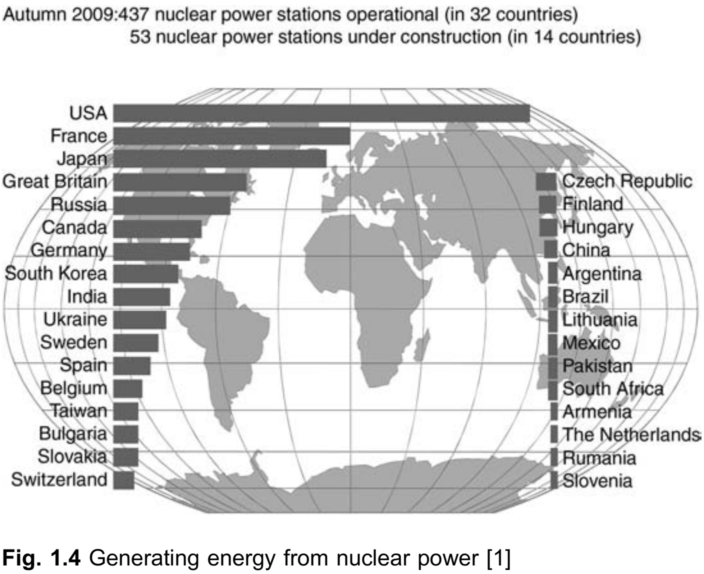

Fig. 1.4 Generating energy from nuclear power [1]

that are worth extracting at today's prices will be enough for more than 200 years, are spread across the world and the countries they originate in are politically stable.

The world, and Europe in particular, has recognised how important nuclear energy is when it comes to generating electricity, as the many newbuild projects show. A number of European countries, including Finland, France and Britain, have actually been building new nuclear power plants or planning them since 2005. These newbuild projects impose different requirements on structural engineering, not just in building them, but in interim and final storage and restoration work. We will look at these tasks, with their specific safety requirements, below.

2 Nuclear energy

2.1 Generating electricity by nuclear power plants

Basically, nuclear power plants work in the same way as coal- and gas-fired plants, converting heat to electricity. Whereas fossil-fuel-fired power plants run on energy media such as oil, lignite or hard coal, nuclear power plants use the heat given off when atomic nuclei split.

Figure 2.1 shows how a nuclear power plant works (in this case, a pressurised water reactor, cf. Section 2.4.2) and shows the whole energy conversion process. Nuclear fission inside the reactor pressure vessel generates heat, which heats water until it vaporises, turning thermal energy into latent energy in steam. This steam, which is under high pressure, then drives the turbines (converting to mechanical energy), which turn the generators connected to them, generating electrical energy, like a bicycle dynamo. Condensing the steam required to drive the turbines is done either by direct flow or seawater cooling or via a cooling system using a cooling tower.

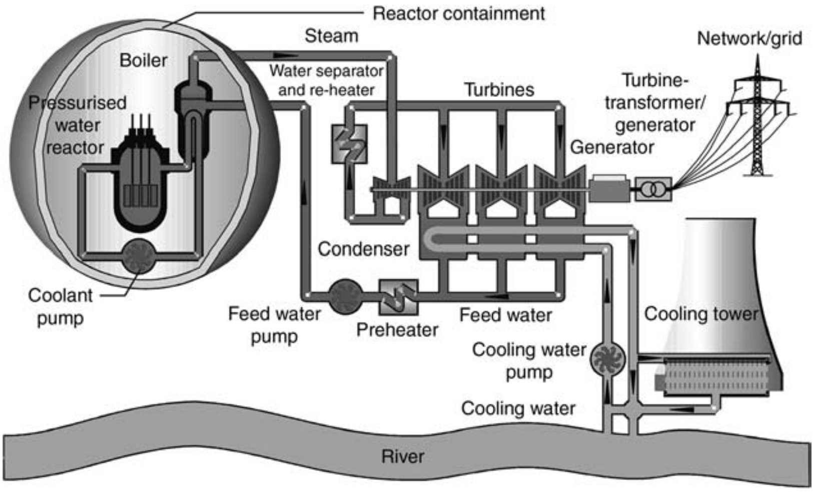

Fig. 2.1 How a nuclear power plant works (pressurised water reactor model)

2.2 Nuclear fission

Most elements on Earth are stable, and the structure of their atomic nuclei is constant. A few of them decompose radioactively, however: that is to say, their atomic nuclei turn into those of other elements by emitting radiation or particles.

In a nuclear reactor, or a reactor at a nuclear power plant, nuclear fission is induced deliberately and the resulting radioactive decay used. Atomic nuclei are split by bombarding them with neutrons.

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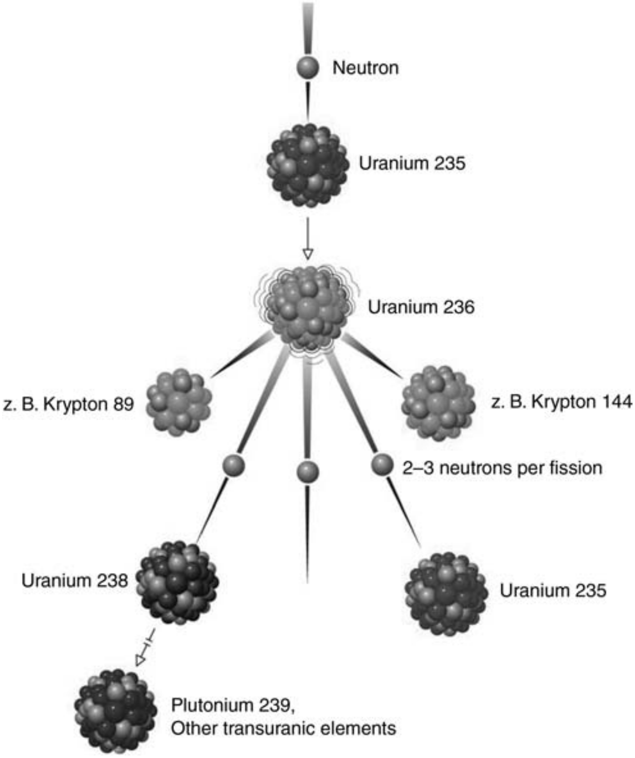

Fig. 2.2 The nuclear fission process

The process of nuclear fission is shown in Figure 2.2. In the reactor, uranium U-235 nuclei are bombarded with neutrons, causing them to fission and emit radiation, known as ‘nuclear radiation’ (cf. Section 2.3). The products of decay are usually two fission products, such as krypton or barium, and two or three neutrons. The neutrons that are emitted can in turn split other atomic nuclei, setting off a chain reaction in which energy is released.

The fission products that arise when atomic nuclei split are unstable: they give off radioactive radiation, turning into stable end products, releasing more energy in the process. This post-decay heat keeps on being generated even after a nuclear reactor has been shut down, and requires special post-cooling systems (Figure 2.3).

A constant steady chain reaction needs a certain minimum mass of fissionable material, also known as the ‘critical mass’. Critical mass exists if the number of secondary fissions (second generation neutrons) is equal to the number of primary fissions (first generation neutrons).

Uranium U-235 is the only element occurring in nature that can maintain fission via a chain reaction. U-235 accounts for just 0.72% of the total mass of uranium occurring

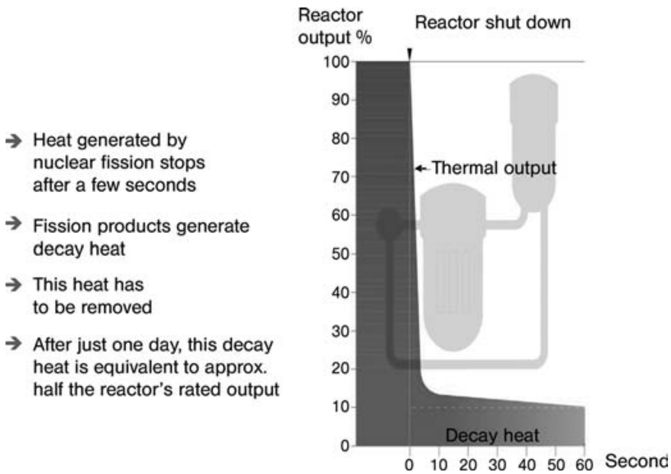

Fig. 2.3 Thermal output from a reactor once it is shut down

naturally, so it does not provide the critical mass required: this has to be increased, i.e. the uranium has to be enriched. This can be done using diffusion, gas centrifuges or separation nozzles.

The critical mass of U-235 required is less if the neutrons that are released when its nuclei split can be slowed down to lower, thermal speeds (moderated). This can be done using what is known as a moderator. Apart from carbon in graphite form and heavy water (deuterium oxide, or D_2O), this is best done using light water, or H_2O. The water molecules slow the neutrons down very effectively, thus maintaining the chain reaction; and the water absorbs the energy from nuclear fission, which heats it up considerably, making it ideal for generating electricity. When using H_2O as moderator, the natural uranium has to be enriched to around 3.5% U-235.

2.3 Radioactivity

Radioactivity can be defined as when atomic nuclei of one element turn into nuclei of another element, emitting radiation or particles in the process. Radioactive processes can be divided into decays of different kinds. The most important decay and radiation processes involved with uranium ore are as follows (Figure 2.4):

- **Alpha radiation**
  Has little penetration strength, and can be blocked by just a sheet of paper (1) (discovered by *Becquerel* 1896).
- **Beta radiation**
  More penetrating than alpha radiation, but can be blocked by thin plate or a few mm of aluminium (2) (discovered by *Rutherford* 1896).
- **Gamma radiation**
  High-energy short-wave electromagnetic radiation, often created during alpha or beta decay. Can be shielded by plates of varying thickness, depending on how much

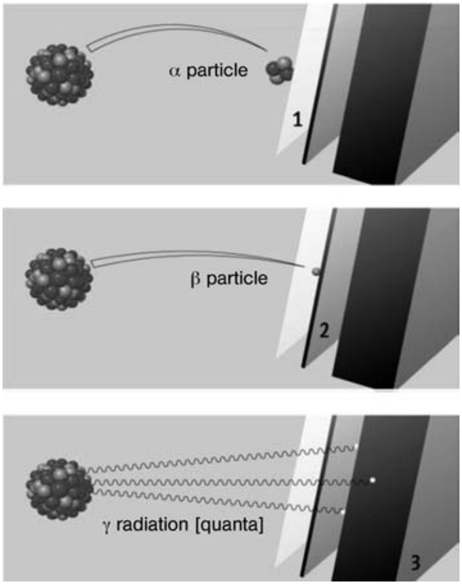

Fig. 2.4 Radioactivity and shielding

energy it contains (3). Nearly all atomic nuclei emit gamma rays (discovered by *Villard* 1900).

What effects radiation has depends on what kind of radiation it is, what the dosage levels are over time and how sensitive the material being radiated is. Radiation absorbed by the human body is abbreviated to Rad for short (radiation absorbed dose).

At a given energy dose D, the biological effects may vary considerably, depending on the type of radiation involved: so a weighted radiation dose (equivalent dose) is used as the biologically effective dose. This equivalent dose H is expressed in sieverts (Sv), generally quoted as mSv or μSv, and is calculated from the energy dose D and an assessment factor q which reflects the characteristics of the radiation. This radiation dose over time then gives the radiation load as a dosage level. A number of natural and man-made radiation sources, with their radiation loads, are compared in Figure 2.5.

The radiation load from nuclear power plants is controlled by law, so the limits as stated in the radiation protection regulations must not be exceeded, even where the effects are worst. Nuclear power plants also have retention systems to prevent radioactive substances getting into the environment.

These retention systems include:

– Ventilation systems working at a partial vacuum to ensure that air always flows from less active to more active areas

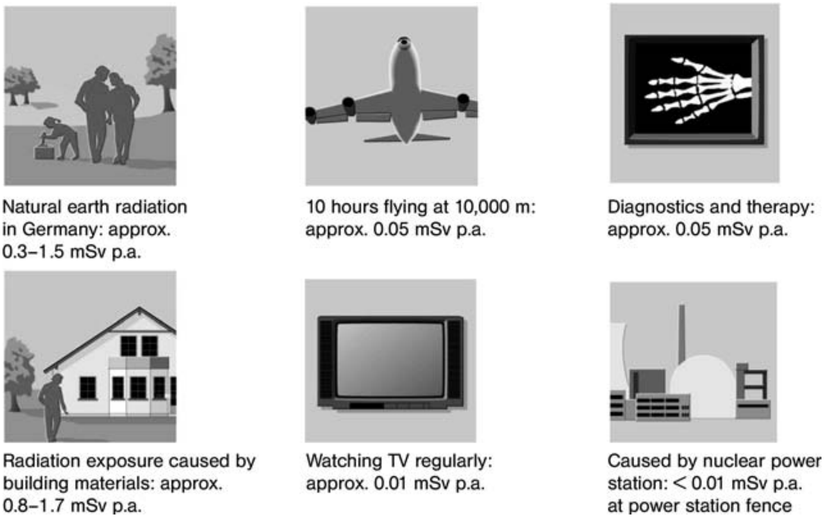

Fig. 2.5 Natural and man-made radiation sources

– Exhaust systems (microfilters etc.)
– Systems for treating radioactive contaminated water to achieve a high decontamination factor (relative energy levels before and after treatment) and minimise waste.

If we look at the radioactive waste from nuclear power plants more closely, we find that, once it has been used in the nuclear reactor, the high-energy nuclear fuel consists of 95% uranium, 4% fission products and 1% plutonium. This spent nuclear fuel can be reprocessed, recycling its useful component, but the current nuclear consensus in Germany has ruled out reprocessing, so spent fuel elements must be kept in intermediate storage until they are put into final storage at the nuclear power plant sites (see also Section 4.4). As well as this highly active waste, nuclear power plants also produce moderate- and low-activity waste. Putting this more clearly: a 1300 MW pressurised water reactor produces around 510 m^3 of radioactive waste a year in total, of which 1% is highly active and around 92% is low-activity waste (Figure 2.6).

2.4 Reactor designs

2.4.1 Overview

Many kinds of nuclear reactors have been developed since the discovery of uranium's nuclear decay in 1938. These can be divided into generations, in the order in which they were developed, as follows:

– **Generation I**
  The initial prototypes built between 1957 and 1963.

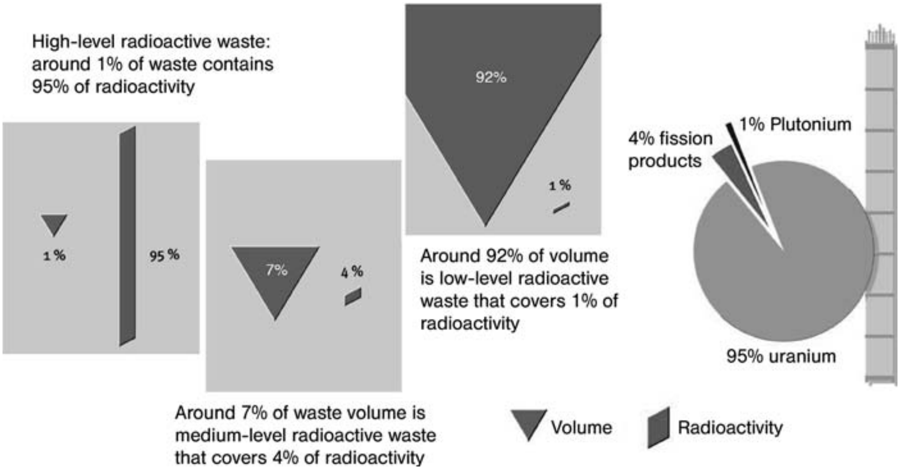

Fig. 2.6 Radioactive waste from a 1300 MW pressurised water reactor (approx. 510 m^3 p.a.)

- **Generation II**
  Commercially viable reactors built from the mid 1960s onwards.
- **Generation III**
  Advanced reactors, generating much more power and with much more concern about safety, built since the early 1980s.
- **Generation III+**
  The next generation of reactors, with structural safeguards against meltdown and/or passive safety features.
- **Generation IV**
  The reactors of the future, highly efficient, with advanced safety features and producing little spent nuclear fuel, but not expected to come on stream until 2030 at the earliest.

(Remark: At the international level, Generation III is often classed as part of Generation II, so Generation III+ is referred to as Generation III.)

Of the types of nuclear reactor that have been developed, there are only a few that can be used in commercial operation. The different types can be broken down by the following aspects:

- **Fuel**
  e.g. natural uranium, enriched uranium, plutonium, thorium; whether they use clad or unclad solid fuels (cladding materials are zirconium, aluminium, magnesium or magnesium oxide – Magnox); fuel elements may be rods, plates, tubes or pellets
- **Neutron energy**
  thermal reactors (moderated neutrons, using moderators such as graphite, light water H_2O or heavy water D_2O) and fast reactors (without moderating the neutrons)
- **Coolant**
  light water H_2O, heavy water D_2O, gas (air, but mainly carbon dioxide and helium).

Table 2.1 Different types of reactor (different combinations of moderator and cooling)

Moderator	Coolant	Reactor type
Light water (H_2O)	Light water (H_2O)	PWR – pressurised water reactor
Boiling light water (H_2O)	Boiling light water (H_2O)	BWR – boiling water reactor
Heavy water (D_2O)	Light water (H_2O)	Advanced CANDU
Heavy water (D_2O)	Heavy water (D_2O)	CANDU – Canadian deuterium uranium reactor
Graphite	Helium (He)	HTGR – high temperature gas-cooled reactor
Graphite	Carbon dioxide (CO_2)	AGR – advanced gas-cooled reactor
Graphite	Light water (H_2O)	RBMk – graphite moderated pressure tube reactor

The first basic distinction here is between thermal and fast reactors. Fast reactors are better known as fast breeders, because when they are operating they ‘breed’ more fissionable plutonium from the uranium than they use, which means that they can get around twice as much energy out of the uranium. Fast breeders have failed to establish themselves, however, for a number of reasons (political reasons in Germany).

Amongst the thermal reactors, there are a number of combinations of moderators and coolants which have been developed successfully for commercial use (Table 2.1). The two main families involved here are gas-cooled reactors (Magnox reactors), advanced gas-cooled reactors and high-temperature reactors and water reactors (light and heavy water reactors).

The most important of these are the light water reactors, as they are also operated in Germany at present. They have proved themselves worldwide, and are the reactors of choice not least because of their safety aspects. Apart from a few exceptions, light water reactors are the only ones that have been designed and built worldwide for some years now.

2.4.2 Light water reactors

The water which is typically used as coolant in light water reactors can be used both in a single-circuit system or – to prevent contamination – in a multiple-circuit system via heat exchangers. Light water reactors are known as pressurised water reactors (PWRs) or boiling water reactors (BWRs), depending on whether the water in them is pressurised or boiling.

In a pressurised water reactor, the water in the reactor pressure vessels is at extremely high pressure, around 150 bar, so the water does not boil, even at the design operating

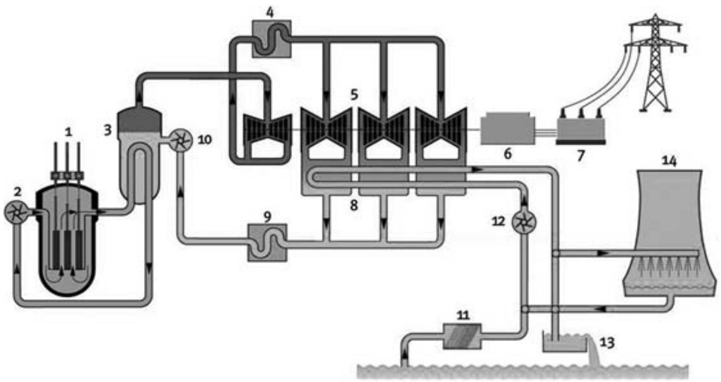

Fig. 2.7 Nuclear power plant with pressurised water reactor (PWR)

temperature of 300 °C. This prevents steam bubbles forming, which would complicate the heat transfer process.

As Figure 2.7 shows, a PWR has two coolant circuits: the primary and secondary circuits (water-steam circuit). In the primary circuit, the coolant water flows round the fuel elements directly: the water which is heated in the reactor core of the reactor pressure vessel is then fed to the boiler and back to the reactor core via circulating pumps. The steam generator then transmits the heat to the secondary circuit, producing steam which drives the turbine and consequently the generator, so the steam passing to the turbine is not radioactive. At the end of the secondary circuit, the steam which was depressurised and condensed in the condenser (coolant water circuit) is pumped back to the steam generator via heat exchangers (preheater unit).

Unlike a pressurised water reactor, in a boiling water reactor (Figure 2.8), the water in the reactor core of the reactor pressure vessel is heated to boiling point: so comparatively little pressure is required at the proposed operating temperature of 300 °C. A pressure of 70 bar is sufficient. Nor is a boiler required, so only one coolant circuit (direct circuit) is necessary. The live steam is fed directly from the reactor pressure vessel to the turbine, which means that the turbine becomes radioactively contaminated to a limited extent. Unlike with the PWR, in which the reactor is controlled and can be crash shut down by control rods from above, with the BWR, control rods are inserted into the reactor core from below. (Please note: control rods are used to control and shut down nuclear reactors.)

If a loss of coolant accident (LOCA) occurs (Section 2.5) in a BWR the pressure is reduced by condensing the steam released in a condensation chamber, so the safety

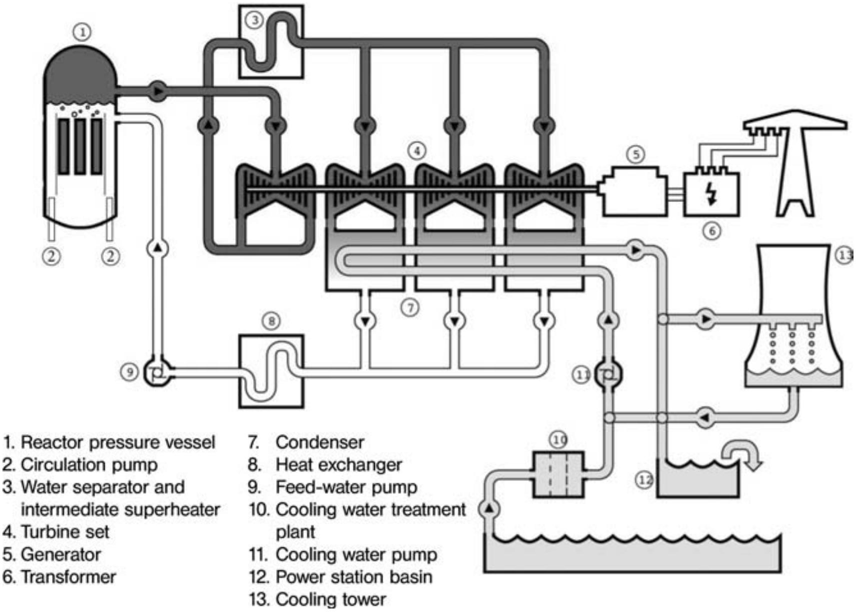

Fig. 2.8 Nuclear power plant with boiling water reactor (BWR)

vessel containing the reactor pressure vessel in the reactor building is much smaller than for a PWR of comparable output.

Of the eleven PWRs and six BWRs operating in Germany, the three PWRs of the Convoy model (Siemens KWU) with an output of approx. 1400 MW are the most advanced. One of these Convoy plants, which may be classified as Generation III, operates at the Isar site (near Landshut) together with a BWR unit (Figure 2.9).

Fig. 2.9 Isar nuclear power plant, Germany: KKI 1 (BWR) and KKI 2 (PWR)

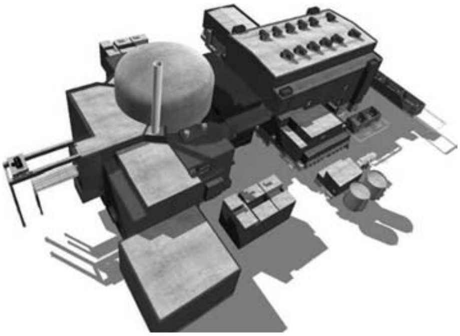

Fig. 2.10 Overall view European pressurised water reactor EPR (AREVA, 3D visualisation)

In the course of the further development of the Convoy nuclear power plant design, German nuclear power plant operating companies decided to join forces with the French state company EDF to develop the EPR (European Pressurised Water Reactor) in 1991. This EPR, a Generation III+ model generating 1600 MW, is currently being built in Finland and France (Figure 2.10). It is being supplied by French plant supplier AREVA, which acquired the former Siemens KWU some years ago.

As well as EPR, AREVA with German involvement (E.ON Kernkraft) is also developing the boiling water reactor KERENA (formerly designated SWR 1000) with an output of 1250 MW (Figure 2.11).

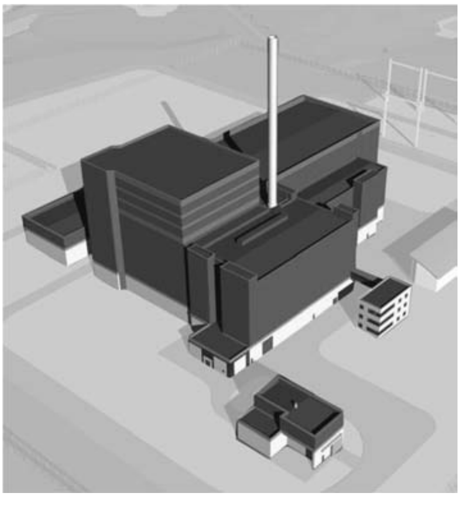

Fig. 2.11 Overall view boiling water reactor KERENA (AREVA, 3D visualisation)

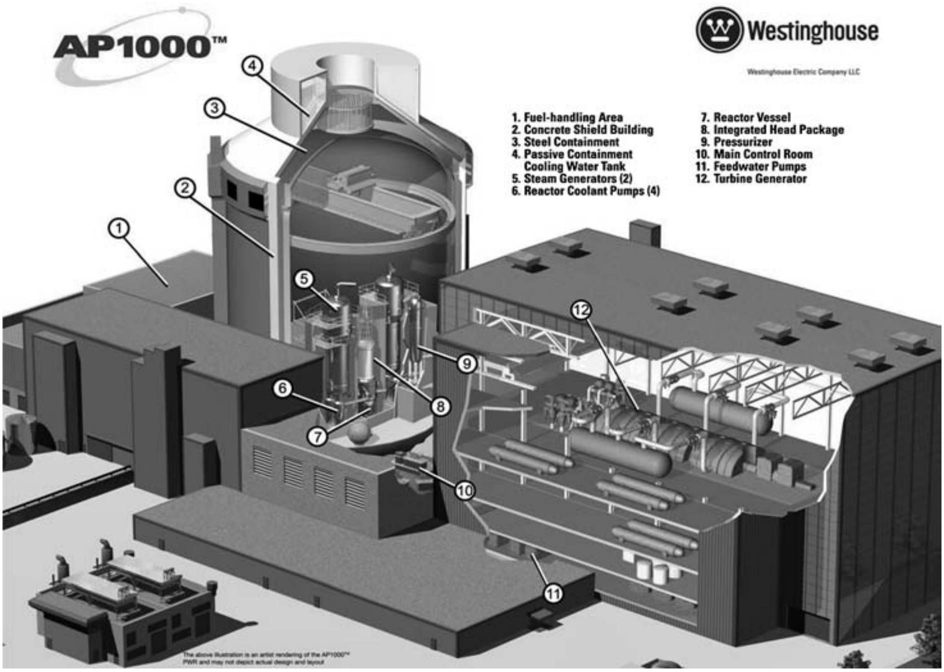

Fig. 2.12 AP 1000 pressurised water reactor (Westinghouse)

Further new developments in Generation III+, which are now being offered and preferred as large-scale power plants with outputs of well over 1000 MW each, are the boiling and pressurised water reactors as listed below (Figures 2.12 and 2.13):

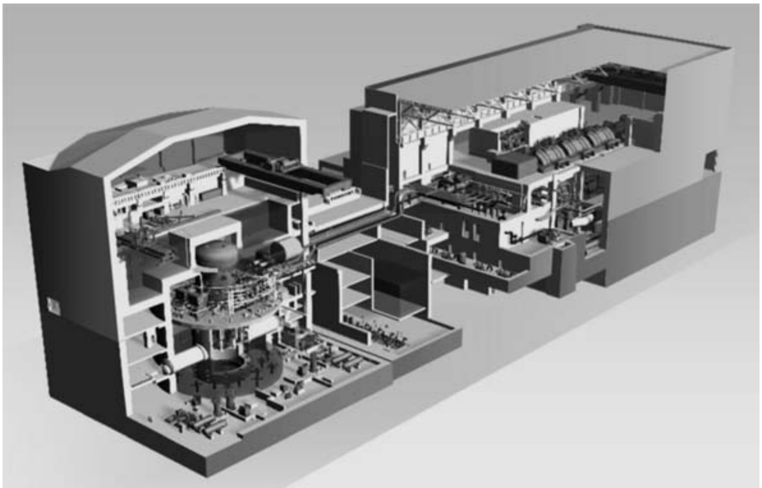

Fig. 2.13 ABWR boiling water reactor (Westinghouse/Toshiba) [3]

- ABWR: BWR – 1350 MW;
  Supplied by: Westinghouse (USA)/Toshiba (Japan),
- AP1000: PWR – 1000 MW;
  Supplied by: Westinghouse (USA),
- AES 92: PWR – 1000 MW;
  Supplied by: ASE (Russia),
- APR1400: PWR – 1400 MW;
- Supplied by: KOPEC (South Korea).

2.5 Safety philosophy

When making safety assessments, we need to make a basic distinction between the risk potential, as the maximum possible damage a risk source can cause, and the risk, which involves considering both the potential extent of the damage and how likely that damage is to occur. Nuclear power plants and nuclear installations generally have a high risk potential, so safety is absolutely vital when designing, building, operating and shutting down such plants to minimise the risks involved (damage prevention).

In nuclear power plants, these requirements mean protective goals such as controlling reactivity, cooling fuel assemblies, confining radioactive substances and limiting radiation exposure must be adhered to. The components and building structures required to meet these requirements are safety-related and are therefore referred to as safety-related components and building structures.

To meet these safety goals, we basically use the safety barriers as shown in Figure 2.14 and the safety systems, which may be designed as either active or passive safety systems.

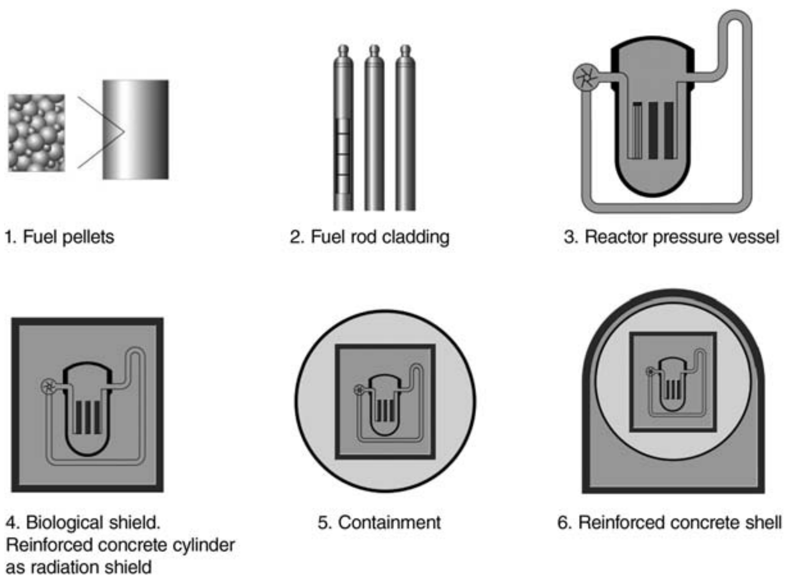

Fig. 2.14 Passive safety barriers

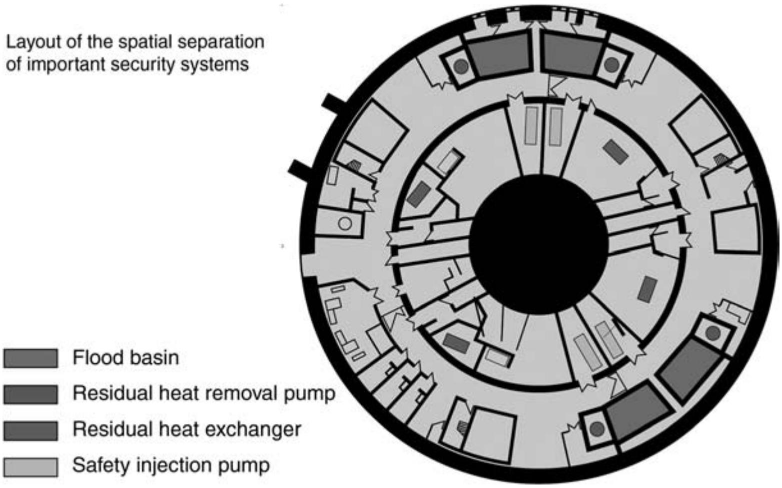

Fig. 2.15 Section through reactor building

These safety systems are highly reliable, thanks to the following:

- **Redundancy**
  Main safety system components are multiplicated so that, should one of these modules fail, another identical module can take over.
- **Diversity**
  Major components are made to different designs, so that not all the modules of the same type needed for these safety systems are likely to fail at the same time. This also reduces the risks of failing for the same reasons (common cause failure).
- **Spatial separation**
  Major components of redundant safety systems are located away from one another, so that if an incident occurs that has limited local events that cause one module to fail, an identical module somewhere else which is not affected by that incident can take over the safety function (Figure 2.15).

Safety systems are needed to manage incidents, and must therefore be designed for both rare internal incidents (internal actions) and rare external ones (external actions). Internal design basis accidents include loss of coolant accident and internal flooding. A loss of coolant accident, and how it is handled, is shown in Figure 2.16. Significant external actions include earthquakes and floods. So plants must be designed to withstand an earthquake with the greatest seismic effects foreseeable where they are located.

These rare design basis accidents can be distinguished from the system status conditions in nominal use (normal operations as regular condition and anomalous operations as frequent condition) and the extremely rare events resulting from an accident (Table 2.2). Under the banded safety concept used in Germany, these system

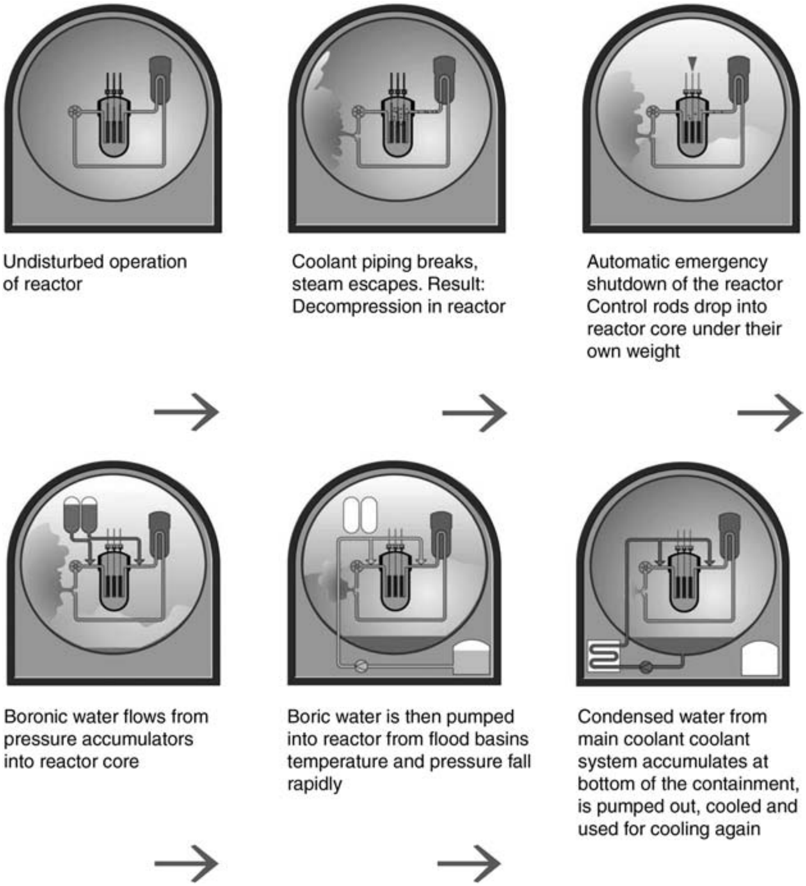

Fig. 2.16 Controlling loss of coolant accidents

status conditions and events are assigned to four safety levels, as shown in Table 2.2, plus the extremely rare events which as accidents count as so-called ‘residual risks’ and which call for disaster prevention and environmental protection measures accordingly.

In the scenarios in Table 2.2, which must be considered as part of a safety philosophy, the focus is on preventing damage. One major contribution to this damage prevention is made by the International Atomic Energy Agency (IAEA), the international atomic energy organisation which sets the standards for erecting and operating nuclear plants. Another contribution comes from reviewing preventive measures and how effective the safety systems of each nuclear power plants are in the light of past events at nuclear plants which have occurred worldwide.

Following the reactor accident at Chernobyl in 1986, the IAEA launched the INES scale in 1991 (Table 2.3) for recording incidents and events at nuclear facilities.

Table 2.2 Safety strategies for nuclear power plants

Safety Level	System Status/Events		Notes, Explanations
1	Operating nominally	Normal operation	Including repairs/maintenance
2		Anomalous operation	Operating processes involved if plant components or systems malfunction
3	Design basis accidents		Rare events to be considered, such as loss of coolant accident, earthquake, flood
4	Beyond design basis accidents	Specific rare events	Extremely rare events to be considered, such as aircraft impact, shock wave from explosion
		System status outside design criteria	
Residual risks		Damage with relevant effects on environment	Action: disaster and environmental protection

This ranges from anomalies (levels 0–1), incident (levels 2–3) and accidents (levels 4–7). Depending on how they are rated, events must be assessed at the nuclear power plant concerned and rated with the controlling government organisations to show that the safety procedures in place are effective and to improve them if required.

On the INES scale, the Chernobyl incident must be classified as a major accident at the highest level 7. By way of comparison: the equally notorious event (partial meltdown) at Harrisburg ('Three Mile Island' in 1979, where the effects of the meltdown were limited to the plant itself, without damaging the health of the population, were classified as a level 5 event. In Germany, there have been 74 events since 1991 which were rated as level 1 on the INES scale and just three that were rated at level 2.

In assessing the risks that nuclear power plants present, we also include extremely rare events as 'hypothetical' accidents classified as residual risks. These include a meltdown due to serious core problems. Compared with the units now operating (up to Generation III), in which the effects of a meltdown are studied as part of a safety analysis, the new Generation III+ plans are designed such that they have structural

Table 2.3 INES (International Nuclear Event Scale)

Level	Description in brief	Aspect 1: Radiological effects outside plant	Aspect 2: Radiological effects inside plant	Aspect 3: Effects on safety precautions
7	Major accident	Acute emissions, affecting health and environment within a large radius		
6	Serious accident	Major emissions, full disaster measures		
5	Accident with wider consequences	Limited emissions, some disaster measures taken	Serious damage to reactor core and radiological barriers	
4	Accident with local consequences	Minor emissions, radiation levels amongst population approx. on a par with natural exposure	Limited damage to reactor core and radiological barriers; staff exposed to radiation, resulting in deaths	
3	Serious incident	Very low emissions, radiation levels amongst population a fraction of natural exposure	Major contamination, staff exposed to radiation with acute health damage	Banded safety precautions mostly fail
2	Incident		Significant contamination, radiation levels amongst staff over permitted limits	Limited failure of banded safety precautions
1	Anomaly			Levels out of range for plant to operate safely
0	Below scale			Not significant in safety terms

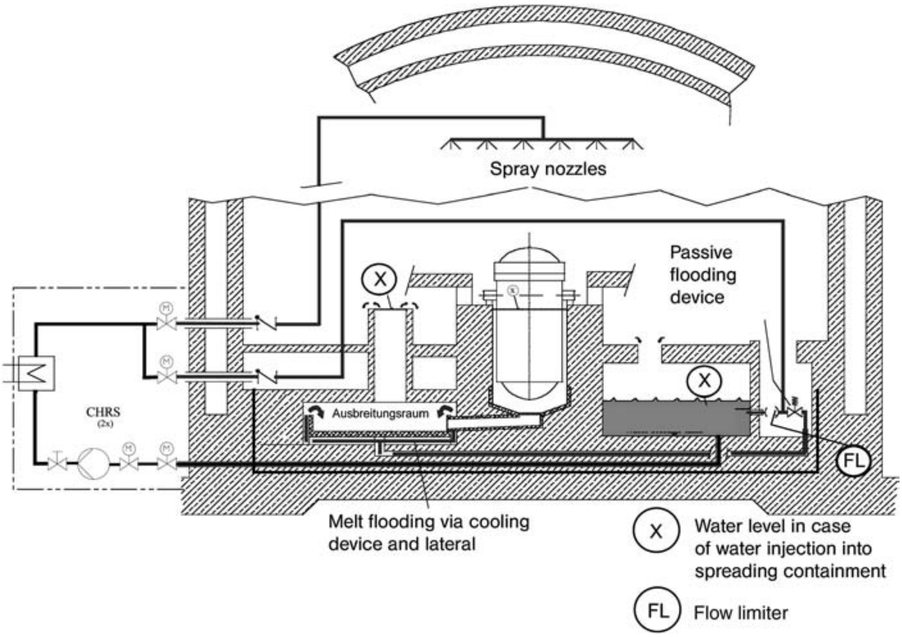

Fig. 2.17 EPR, core catcher (heat removal system)

safeguards and/or passive safety features to protect against a meltdown. For the EPR, for example, a meltdown prevention system was designed with a 'core catcher'. As Figure 2.17 shows, should the reactor's pressure vessel burn through, the molten core can be collected in a space below the reactor pressure vessel and cooled to avoid it burning through the foundations and to prevent pressure in the reactor pressure vessel increasing any further. This covers not only short-term but also long-term safety functions, without any further serious disaster prevention measures being required in the area around the plant.

3 Approval aspects

3.1 Atomic energy and construction law

Building, operating and making major changes to nuclear installations must be approved under atomic energy law. The government approval process used depends on the national law requirements involved in each case, so varies from one country to another.

Before they can be allowed to build and operate nuclear installations, there are a number of basic requirements that applicants and operators have to meet:

– Applicants and operating managers must be reliable and expert.
– Those working at the company must know how to run it safely, what the potential risks are and the safety precautions required.
– Know what precautions are required under the state of the art of science and technology to prevent erection and operating the plant causing damage.
– Know what protection is required against anomalies and other effects by third parties.
– Keep water, air and soil clean in the public interest.

In Germany, the legal framework for using nuclear energy for peaceful purposes is provided by the Atomic Energy Act and regulations issued pursuant to it. The safety goals and measures are laid down in Section 2.5 of the Atomic Energy Act [4]. These are clarified specifically by Atomic Energy Act regulations and internal administrative rules and guidelines, such as the guidelines for PWRs [5] or design basis accidents guidelines [6]. These guidelines cover the design basis accidents to be considered and managed; they also specify the requirements of the radiological protection ordinance [7].

The building structures required to meet the safety targets, which are therefore classified as safety-related, have to meet not only the requirements of construction law but also those of the Atomic Energy Act. That means both planning permission and permission under the Atomic Energy Act are required in Germany. To be approved under Atomic Energy Act, constructors/operators must show that they have taken the necessary precautions against damage in accordance with the state of the art of science and technology. Safety-related building structures must therefore meet not just the conventional requirements of construction law but also additional safety requirements in line with the state of the art of science and technology.

3.2 Interface between plant and structural engineering

The inspection required to be approved under the Atomic Energy Act includes a holistic examination of the safety precautions of the building structures. This involves defining the interface between the building structures (structural engineering) and plant components (plant engineering) and hence the distinction between construction and atomic energy law. Generally speaking, plant components such as pipes and containers are part of the building, so that the fastenings in each case (anchor plate) constitute the

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interface. In exceptional cases, this interface will have to be defined in the official planning process.

Each interface must have an interface document which, amongst other things, specifies the loads calculated from the plant technology and the structural member or structure in each case. Such documents are generally called structural design requirements. They are first considered as part of the atomic energy law terms, covering systems engineering aspects, and then used as the basis for the construction assessment.

3.3 Periodical safety reviews

As part of the approval process to give the go-ahead to construct and operate a nuclear plant, it must be shown that the necessary safety precautions have been taken, in accordance with the current state of the art of science and technology. The evidence required must be considered deterministically, in the light of a reasonable safety strategy, as laid down by the banded safety strategy in Table 2.2, for example.

While plants are operating, the competent atomic supervisory authority monitors the state of their systems and how they are being operated, to verify that these comply with the conditions of the approval order. In addition to these checks, in the operating phase, regular safety status presentations must be made considering whether new safety findings from operating experience, safety studies and research and development should be incorporated.

In Germany, safety status is monitored by periodical safety reviews, or PSÜs in German. These must be held every ten years, and cover:

- Deterministic safety status analysis (DSA) in the shape of a safety target oriented review of a plant's safety status including how it is managed operationally and analysing its operating experience
- Probabilistic safety analysis (PSA) [8]
- Plant safety strategy review.

3.4 Planning and design requirements

3.4.1 IAEA Rules

Given the potential threat of nuclear weapons, but particularly in expectations that atomic energy would be used peacefully, the International Atomic Energy Agency (IAEA) was set up at the initiative of the United Nations (UN) in 1957. The IAEA has its headquarters in Vienna, and is an independent international organisation with close links to the UN. It sees its task as making it possible to use nuclear energy, subject to the necessary safety requirements, and ensuring technology transfer.

The IAEA lays down safety requirements for building and operating nuclear installations, assisted by international experts, and is constantly updating these requirements. Once agreed with the countries that operate nuclear installations, these are published as IAEA Safety Standards (see [9–12] for example). Individual countries can then use them as they stand or as the basis for further-reaching national rules, such as German nuclear safety standards (see Section 3.4.3).

One particular area that the rules of the IAEA focus on is earthquakes respectively seismic risks. It has published a number of standards, including methods for determining seismic load assumptions, earthquake-proof design and the earthquake safety of existing nuclear installations. The IAEA also provides advice in cases where nuclear power plants are hit by earthquakes. Calling in experts as required, it assesses damage, considers whether plants can continue to operate and in some cases makes new findings on earthquake risks. To improve this work, the International Seismic Safety Centre (ISSC) was founded in Kashiwazaki, Japan, in 2007.

3.4.2 European catalogue of requirements

Many new nuclear power plants are currently being built or are at the planning stage worldwide, especially in China, Japan and the USA; however, there are also numerous newbuild projects at the planning or construction stage in Europe, such as in England, France and Finland. For these European projects, the European nuclear power plant operators have drawn up a catalogue of requirements in the shape of the European Utility Requirements (EUR) [13].

The EUR relates to nuclear power plants as a whole, and includes details of specific nuclear power plant actions to be taken into account when designing them, such as earthquake design spectra for minimum earthquake design requirements and specific load/time functions to protect against aircraft impact. The EUR also includes basic design criteria. In terms of construction design, these criteria assume basically that Eurocode standards will be observed.

The EUR is intended to ensure that the various nuclear power plant providers can rate and offer their products based on them. Corresponding qualifications have already been drafted for system strategies for Europe, such as EPR or AP1000.

3.4.3 Safety standards of nuclear safety commission

Nuclear installations must meet stringent safety requirements, and so require design strategies accordingly, but which are not covered by conventional plant and construction rules. In 1972, therefore, the Federal German Ministry of Education and Research (BMBW, now the BMU or Federal Ministry for the Environment, Nature Conservation and Nuclear Safety) set up the Nuclear Safety Standards Commission (KTA), on the model of the German steam boiler committee. The Nuclear Safety Standards Commission has assumed responsibility for drawing up safety rules in nuclear systems and promoting their use, through bringing about consistent opinions amongst specialists from those who build, install and operate nuclear power plants, inspectors and the authorities.

Safety Standards of the Nuclear Safety Commission (KTA safety standard) [14] lay down safety requirements, compliance with which provides the precautions required when building and operating plants in accordance with the state of the art of science and technology. These precautions required under the Atomic Energy Act are necessary to achieve the safety targets as laid down in the Atomic Energy Act and radiological protection ordinance and in more detail in the safety criteria for nuclear power plants and design basis accidents guidelines.

The KTA safety standards cover more than 100 fields which include all the issues relevant to nuclear technology and relevant disciplines. They are reviewed regularly (every five years) to see if they need revising: that means more than 50 proposed rules are currently being considered.

KTA safety standards must be regarded as mandatory overall standards in Germany which must always be complied with. They can be varied, in theory, but that means those involved (nuclear regulators and inspectors) would have to reach a consensus viable in law. KTA safety standards are publicly available, and are not used in Germany alone: there are many countries, especially in Europe, that accept these codes or would even like them to be used for their nuclear installations, so that KTA safety standards are largely available in English also.

3.4.4 DIN Codes

The codes of DIN, the German standards institute, are generally accepted as codes of the art which are reflected in the KTA safety standards. These standards do not normally apply to nuclear facilities, or are even expressly excluded from applying to them; so the DIN has set up a nuclear technology standards committee, now standards committee materials testing (NMP) specialist area 7 – nuclear technology, which is responsible for producing and updating specifically nuclear standards.

As far as construction technology is concerned, there are two of these codes whose status means that they are also applied internationally: DIN 25449 [15] and DIN 25459 [16]. DIN 25449 covers designing reinforced concrete and pre-stressed concrete components to allow for the rare effects from inside (EVI) and outside (EVA) as safety levels 3 and 4 (see Section 2.5). DIN 25459 also lays down rules for designing safety containments using reinforced and pre-stressed concrete. Due to the integrity requirements involved, such containments also require additional claddings such as steel or plastic liners; this standard also deals with their design, including laminate effects with the reinforced or stressed concrete design. DIN 25459 only exists to date as a pre-standard, which is currently being revised and should be published as a fully fledged standard in the near future.

4 Building structures for nuclear plants

4.1 General notes

Nuclear plants are divided into generator reactors, research and training reactors and nuclear fuel supply and disposal systems. This includes, in particular, nuclear power plants for generating electricity, fuel element production installations, uranium enrichment plants, protective structures such as ponds and storage facilities for radioactive waste, which in turn are divided into interim and final storage facilities.

Building structures required for nuclear plants whose protective function means that they are classified as safety-related (cf. Section 2.5) have to meet particular construction requirements. These requirements, which are more stringent than those involved in conventional construction, must be observed not just when designing and constructing buildings but also when operating and dismantling nuclear plants.

4.2 Nuclear power plants

4.2.1 Building structure classification system

Large-scale power plants, whether conventional or nuclear, have many system components and structures which must be clearly marked and classified. This used to be done earlier using the plant coding system (in German: Anlagen-Kennzeichnungs-System, AKZ), which was replaced by the identification system for power plants (in German: Kraftwerk-Kennzeichensystem, KKS) in the 1980s.

The internationally used KKS system covers 17 digits in different blocks which can be used to designate whole systems, functions, aggregates and operating resources. Viewed as a whole, system components and structures which are universal to all power plants, whether coal-fired, hydroelectric or nuclear, are designated uniformly, such as cooling towers and turbine buildings.

Building structures can be clearly distinguished by three letters of the function code, with the initial letter U being used to designate building structures generally. Building structures for generating heat atomically, for example, are coded UJ, with the third letter such as building structure designation UJA for the inside of reactor buildings and UJB for the annular space of reactor buildings.

Table 4.1 shows some examples of building structures of nuclear power plants designated in accordance with the KKS and AKZ systems. Unlike the more recent Convoy plants (Emsland, Isar 2 and Neckarwestheim 2), older German nuclear power plants still use the AKZ from when they were built. As an example, the layout diagram of the Isar 2 nuclear power plant using the KKS designation system is shown in Figure 4.1.

Of the building structures shown in Figure 4.1, those which are essential in a nuclear power plant are as follows:

– **Reactor building**
  As well as the reactor itself, with a PWR installation, the reactor building also includes all the primary circuit components and, with a BWR one, essential parts of the live steam

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Table 4.1 Model list of building structures and designations (KKS: power plants identification system; AKZ: plant identification system German: KKS: Kraftwerk-Kennzeichensystem, AKZ: Anlagen-Kennzeichnungs-System)

Building structure code		Building structure
KKS	AKZ	
UJA/UJB	ZA/ZB	Reactor building – inner space/annular space
UKA	ZC	Auxiliary system building/conditioning system building
UBA	ZE	Switchgear building
UMA	ZF	Turbine building
UBP	ZK	Emergency backup diesel building (emergency generator building)
ULB	ZX	Emergency feed building
UKH	ZQ	Chimney (stack)
URA	ZP	Cooling tower
UPC	ZM1	Cooling water take-off building
URD	ZM2/4/5	Cooling water pump building
UFC	ZD	Interim fuel element store
UST	ZL0	Workshop building
UYC	ZY	Administration building (offices and staff facilities)
UYE	ZV	Porter’s lodge

system. The reactor building also includes the containment, which must prevent leaks under all prospective problems accidents.

- **Auxiliary system building**
  This building houses various storage, stock and wastewater containers, workshops, barrel stores and filter, ventilation and treatment plants.
- **Switchgear building**
  This building, which is relevant to control and guidance systems, houses all the switchgear and modules which supply the various systems involved with electric power.
- **Emergency backup diesel building (emergency generator building)**
  This building houses the emergency diesel generators that supply electricity to the power plant and hence the residual heat removal systems.
- **Emergency feed building**
  This building houses the emergency feed and residual cooling pumps and their associated systems and the switchgear room. This building also houses the emergency feed system, which in an emergency supplies the boilers with feed water to ablate the residual heat that the reactor generates.

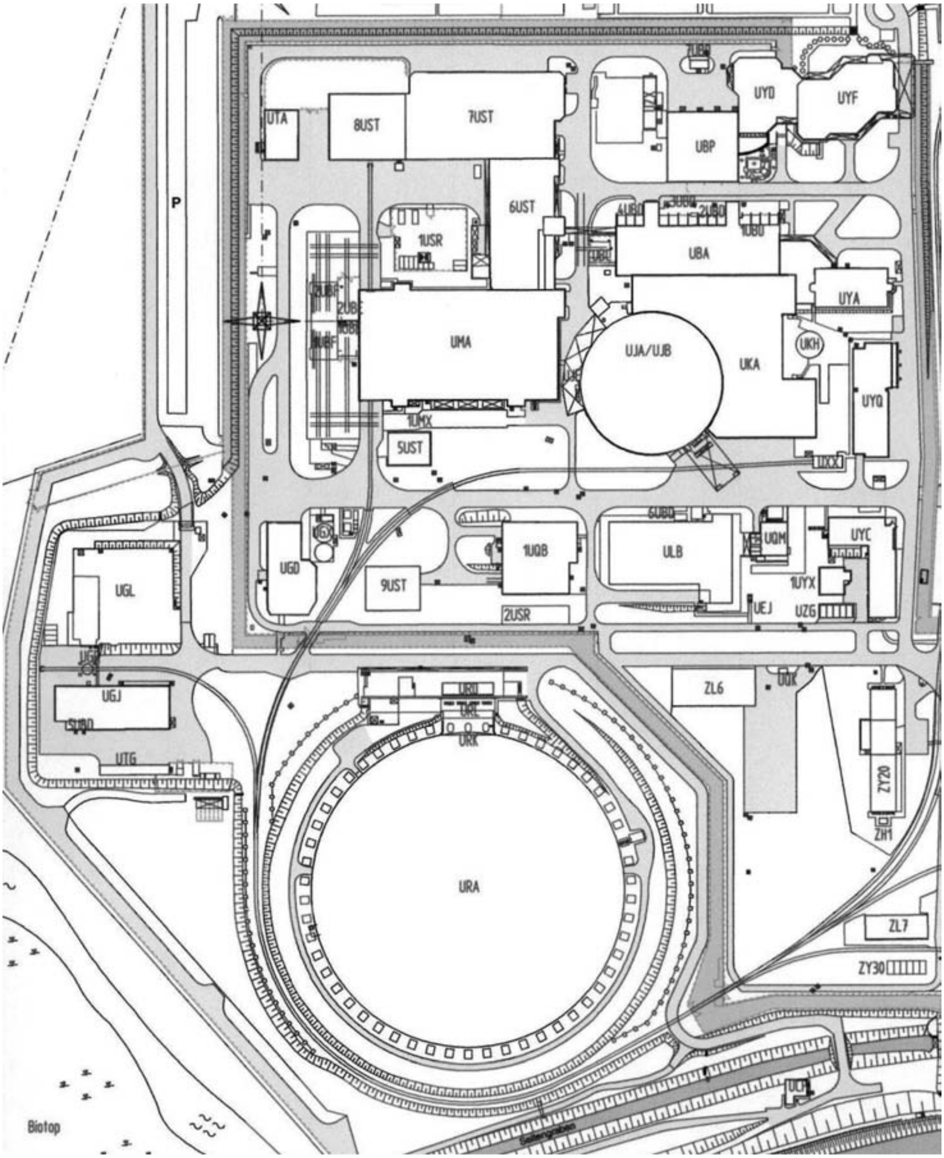

Fig. 4.1 Layout plan of Isar 2 nuclear power plant (cf. Table 4.1)

– **Vent stack**
This chimney releases at a great height the vent air that comes from ventilating buildings and systems. This vent air is monitored for radioactive substances.
– **Water supply building**
This includes the building works for extracting the cooling water, such as the cooling water extractor building or cooling water pumping station, and the building works for returning the cooling water, such as the outlet structure.

For the purposes of nuclear safety philosophy (cf. Section 2.5), building structures are divided into safety-related and non-safety-related.

Safety-related building structures include such things as the reactor building, auxiliary systems building, switchgear building, emergency backup diesel building or the vent stack as well. In a BWR, as opposed to a PWR, the turbine building is also classified as safety-related, as radioactive live steam is fed directly into the turbine in the turbine building (cf. Section 2). Non-safety-related building structures typically include administrative buildings, workshop buildings, gatehouse and cooling towers.

The layout plan in Figure 4.2 shows the main buildings of a nuclear power plant, using the example of Gundremmingen.

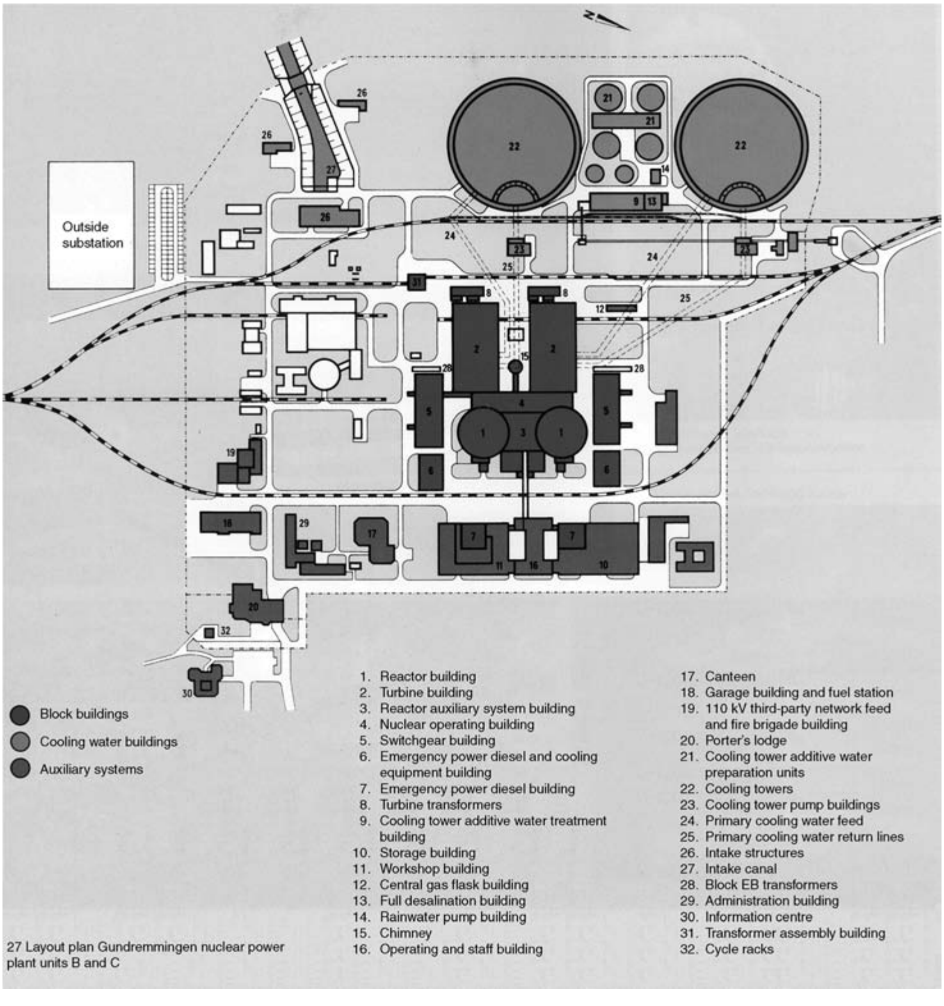

Fig. 4.2 Layout plan of Gundremmingen nuclear power plant [17]

In building design terms, how the buildings are laid out in relation to one another (plant layout) is governed mainly with a view to making safety-related buildings redundant, to protect against external effects (aircraft impact, earthquake, pressure waves etc.) and hence the safety strategy. In this context, the physical separation between redundant buildings and even gap widths between adjacent buildings are also a construction issue.

Redundant buildings, for example, should be spaced so that external events do not stop them being duly redundant. If this cannot be guaranteed, the consequential effects must be studied or the buildings concerned designed to withstand external events.

Another construction aspect of plant layout is the question of optimum building design and/or reducing construction times. With a compact layout, site crane movements tend to overlap, so there are areas which cannot be built at the same time, so the plant takes longer to build. This meant, the buildings before the Convoy stations, were spaced relatively far apart, which also meant that the sites themselves were larger.

This 'relaxed' approach has been reversed with more recent plants, like the EPR or KERENA (Figures 2.10 and 2.11); for example, short cable and pipe runs and protecting safety-related building sections under one roof and on a common foundation slab have proved to be more cost-effective.

4.2.2 Materials

4.2.2.1 General notes

Building structures in recent nuclear power plants are now expected to last for 60 years in operation, and even more than 80 years if we include commissioning and shutdown, so ensuring the materials characteristics required over such long periods makes choosing the right materials particularly important.

4.2.2.2 Concrete

Normal weight concrete

The concrete strength grades normally used in nuclear installations in Germany are C30/37, and in exceptional cases C35/45, as in site concrete in particular cases. Concrete is normally mixed on site.

It was initially thought to make financial sense to use concrete in strength classes C55/67, because of its high strength, but this has proved to be less robust than expected, for various reasons. High-strength concrete is less ductile: any cracks which occur develop straight through the aggregates, creating relatively smooth crack surfaces. This affects integrity and 'self-healing' considerably.

Radiation protection loaded concrete, heavyweight concrete

When assessing shielding levels, a distinction must usually be made between gamma radiation and neutron radiation. DIN 25413 [18] classifies shielding concretes by the proportion of elements they contain. How much shielding concrete provides against gamma rays depends directly on the bulk density of the concrete and the proportions of elements it contains by weight. In other words, the higher the weight of concrete, the more shielding it provides. With neutron radiation, how much shielding concrete provides depends on what chemical elements it contains. As well as using additives with crystal water content, the proportion of light elements, such as hydrogen, is particularly important, (or boron compounds are also used, such as boron carbide, borax frit, colemanite, boron calcite) which are particularly good at ‘trapping’ fast neutrons.

Raw density specifications are generally based on dry weight or dry raw density. DIN 25413 defines different compositions of concrete mixes and the main element proportions involved, such as O, C, Si, Ca and Al or it recommends a so-called average composition. For heavyweight concrete, this standard also specifies different kinds of concrete and characteristic proportions of elements, depending on the heavy aggregates used (haematite, magnetite, ilmenite, barytes, limonite and serpentine). Under DIN EN 206-1 [19] and/or DIN 1045-2 [20], heavyweight concrete has a dry specific gravity in excess of 2.6 t/m^3. However, DIN 25413 refers to an older definition of heavyweight concrete. Radiation-proof concrete made with heavyweight aggregates therefore generally has a raw density over 2.8 t/m^3.

Heavyweight concrete is considerably more expensive to bring in than standard concrete. Because of the largely angular aggregates and higher density involved, it does not pour nearly as well as standard concrete, and it requires more careful mixing to ensure that components of different density do not separate. For notes on this, and an overview of heavy aggregates, see the DBV code of practice for radiation protection concretes [21].

Heavyweight concrete as radiation protection concrete was already being used in the first nuclear facilities in Germany in the 1960s, at the Jülich research centre (research reactors DIDO and MERLIN). So-called ball scrap concrete (with cast-iron granulate), for example, with a specific gravity of 5.6 t/m^3, had been used.

In more recent plants, however, additives such as magnetite, serpentine, haematite or barytes or in some cases granulated iron additives had been used, as they are easier to work.

A cement content of 340 kg/m^3 (CEM III/B 32.5), a water content of 170 kg/m^3, with 1410 kg/m^3 of haematite 0/6, with 1680 kg/m^3 haematite 6/25 and 150 kg/m^3 sand 0/8 has been used to give a specific gravity of 3.6 t/m^3, for example. Using 890 kg/m^3 haematite 0/8 instead of sand 0/8 and 1920 kg/m^3 haematite and an extra 1350 kg/m^3 iron granulate can give a bulk density of approx. 4.5 t/m^3.

Heavyweight concrete as radiation-proof concrete is mostly required in the immediate environment of the reactor pressure vessel as part of the bioshield. In the support area of the reactor pressure vessel (known as the skirt area) of the bioshield in the containment at Gundremmingen a heavyweight concrete with specific bulk densities of 2.7–4.2 t/m^3 was used, for example.

What is particularly important in construction terms is the dry specific bulk density of a normal concrete to be designed for radiation protection purposes. Using normal quartz gravel as aggregate can only reliably give a dry specific gravity of 2.2 t/m^3. If shielding requires a higher specific bulk density, it must be borne in mind that special aggregates will be required. These may have to be brought by considerable distances, which could increase costs.

4.2.2.3 Reinforcing steel

One essential characteristic of reinforcing steel in nuclear installations is how ductile it is and hence how readily the internal forces and moments can be redistributed. This characteristic is an essential factor in deciding how robust structures are. These characteristics are particularly important when it comes to extreme extraordinary actions such as aircraft impact or earthquakes to dissipate the energy involved as desired. Bst 1100 (or aircraft steel) was widely used in the past, but its lack of ductility meant that it ceased to be able to meet these requirements; today, normal reinforcing steel B500B is used which meets the ductility requirements.

Outside structural sections to be used in areas to be designed to withstand aircraft impact (APC or airplane crash shells) generally use sleeve joints as it is assumed that the bond will be lost in the immediate vicinity of the impact.

4.2.2.4 Pre-stressing steel

Before the building of the pre-stressed concrete pressure vessel at Schmehausen and the pre-stressed concrete containment at Gundremmingen, pre-stressing was only used as a general rule in Germany in wide-spanned precast girders in the turbine buildings and other special support structures, such as the instrument room inside the reactor building at Krümmel.

4.2.3 Reactor building

In terms of structural particularities, it is the reactor building that poses the highest requirements. In what follows, we will limit ourselves to looking at reactor buildings in light water reactors, PWRs and BWRs. The different functional requirements involved here also mean that the shapes of the buildings themselves differ, rectangular buildings being preferred for BWRs and curved building structures with circular footprints (cylindrical or spherical) for PWRs.

A Convoy type reactor building is shown in Figure 4.3. This consists of the spherical reinforced concrete shell typical of many PWRs, with very thick walls ($h = 1.80$ m)

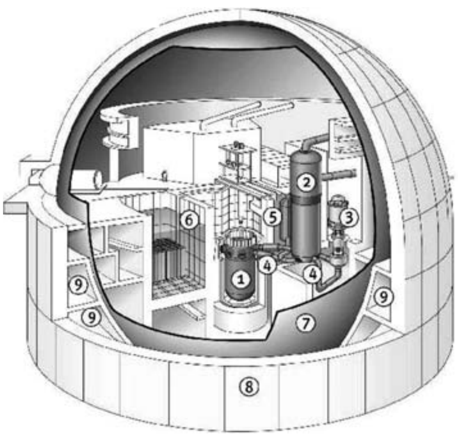

Fig. 4.3 PWR reactor building (Convoy type)

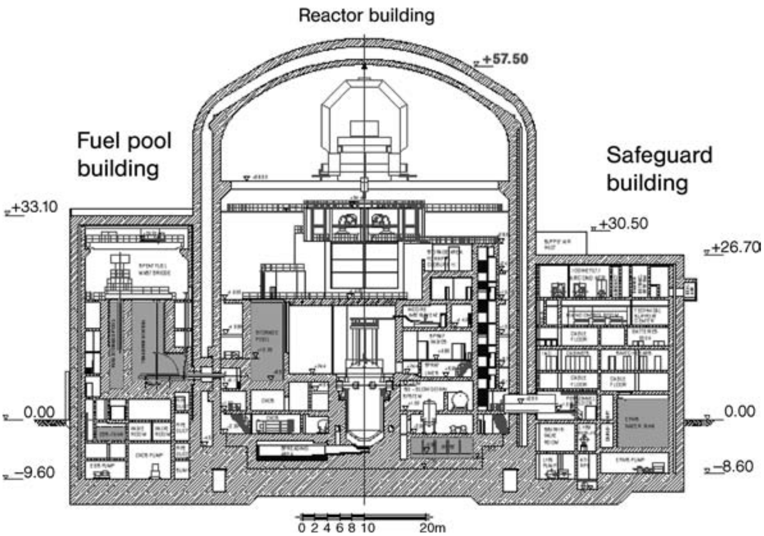

Fig. 4.4 Reactor building with fuel element storage and safety building of a PWR–EPR type [22]

designed to withstand an aircraft crash. This encloses a steel safety container as containment, which maintains integrity even in an anomaly.

The further development of the Convoy power plant model as part of the Franco-German partnership led to the EPR, a Generation III+ reactor, as is currently being built in Finland and France. The main features of the EPR reactor building are as follows (Figure 4.4):

– There is a clear structural separation between the building complexes of the nuclear island (reactor building, fuel element storage building, safety systems building etc.) and those of the conventional island (turbine building, etc., which is why it is also often called the 'turbine island').
– There is a common baseplate for the relevant buildings on the nuclear island, to make it easier in the event of an earthquake to manage the induced shocks acting on the building structures and mechanical components and avoid individual buildings shifting in relation to one another.
– The double-shelled outer wall structure of the reactor building consists of an outer reinforced concrete wall 1.80 m thick, an air gap of 1.30 m and an inner pre-stressed concrete wall 1.30 m thick. The inner wall is of pre-stressed concrete design, with an additional steel liner 6 mm thick on the inside to ensure that the containment does not lose its integrity even in an extreme accident (internal pressure approx. 0.5 MPa at temperatures of approx. 150 °C)

The so-called 'double containment concept' described above has established itself worldwide as far as the layout of the reactor building is concerned. What this means is that external influences, such as earthquakes, aircraft impact, pressure waves etc., can be absorbed by a reinforced concrete structure of a suitable thickness (APC shell). The integrity of the reactor building to contain radioactive substances is maintained by a separate integrity barrier, which constitutes the actual containment or safety enclosure of the reactor building. As well as maintaining integrity, however, the containment must also contain the internal pressures resulting from operations and accidents, plus high thermal stresses.

There are a number of containment concepts, depending on what kind of reactor is involved:

- Reactor containment of steel (e.g. Convoy PWR models)
- Pre-stressed concrete containments without liners (in French N4 reactors, for example, but note that this concept has not proved itself, as integrity requirements cannot be met long term)
- Pre-stressed concrete containment with steel liner (e.g. EPR, approx. 6 mm thick)
- Non-pre-stressed steel containment with steel liner (e.g. KERENA, approx. 10 mm thick)

This list in itself makes it clear that a combination of pre-stressed and reinforced concrete and its associated steel liner is extremely important as an integrity barrier.

Regulatory authorities worldwide are demanding increasingly that plant technology provides passive safety systems and robust design. Whether this should also apply to structural engineering, even using pre-stressing with composite construction, raises ‘argumentation problems’ as far as this robustness requirement is concerned. The pre-stressing, which is usually extremely high, must be maintained over the very long period of more than 80 years. Monitoring pre-stressing with composite construction is difficult, and pre-stressed members cannot be replaced in practice.

The very high pre-stressing also has other drawbacks, as it devolves creep and shrinkage onto the steel liner and other steel components, such as pipe mountings and locks.

More recent containment developments, like AREVA’s KERENA containment, thus omit the pre-stressing, preferring instead to use thicker steel liners and suitable composite construction elements as structural elements in a composite construction with the concrete.

In terms of building construction, making the reactor building roof structure is particularly important, as it has to be extremely thick to withstand the impact of an aircraft. Once the reactor pressure vessel is installed, the interior work begins soon afterwards, so that supporting the shell inwardly is no longer possible in most cases. So the hemispherical roof structure with cylindrical reactor buildings, which can be precast separately, was developed – this was done also because curved structures are much better at withstanding the stresses of an aircraft impact once the membrane strength cuts in than flat surfaces.

On the other hand, doubly curved load-bearing structures are more expensive and take longer to construct, which is why the roof structures of Gundremmingen B and C reactor buildings were made with precast, wedge-shaped laid precast segments with locally cast concrete added (Figure 4.5).

Fig. 4.5 Gundremmingen B and C reactor building roof structure, with precast wedge-shaped laid precast segments with cast concrete added locally [17]

4.2.4 Turbine building

With coal-fired power plants, the conveyor belts which carry the coal normally end at a great height in what is known as the intermediate structure of the turbine building, which for this reason must be considerably higher than the remainder of the turbine building area.

Nuclear power plant turbine buildings do not need such a high intermediate structure, so a continuous turbine floor level and hence a level turbine building roof can be made.

In structural engineering terms, this means that the turbine building roof must span more than 40 m, as the machinery crane must be able to cover the full width of the turbine building.

Most turbine buildings for nuclear power plants therefore use pre-stressed concrete precast girders (Figure 4.6), which are usually installed using the turbine building crane which is already in place.

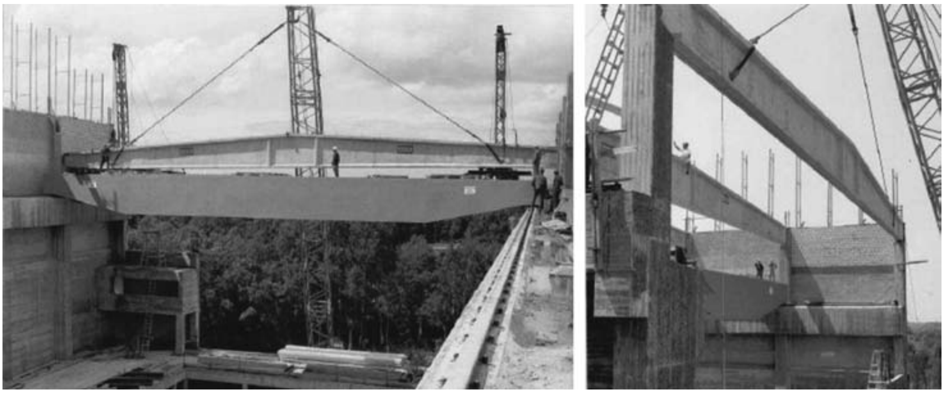

Fig. 4.6 Laying pre-stressed concrete precast girders for the turbine building at the Gundremmingen site, with the turbine building crane already installed in the front left of the picture

The roof structure in earlier turbine buildings at nuclear power plants was generally made of hollow pre-stressed concrete slabs laid directly on pre-stressed concrete girders; however, as designing for earthquakes became increasingly necessary, this solution proved to have many problems because there was no enclosed roof segment to make the structure rigid.

More recent turbine buildings therefore seek to use semi-precast component solutions, with the cast in site concrete topping being added as continuous shear slab.

There is another particular feature with designing the turbine building with boiling water reactors such as Gundremmingen. With this reactor type, the slightly radioactive primary steam is fed directly to the turbine. To protect against radiation, a thicker and therefore also heavier roof construction is required, which in turn imposes particular requirements on the design of the pre-stressed concrete ties and designing to withstand earthquakes.

The global bracing systems in the lateral and axial directions of turbine buildings vary considerably, depending on the plant context as a whole.

While only relatively soft framework systems are available laterally, the building is rigidified mainly by a shear wall in the longitudinal direction. This bracing design, which is different in the two directions of the building, means 3D modelling is often required when it comes to dynamic earthquake analysis of turbine buildings.

This is where the highly rigid spring-mounted turbine table comes in, which absorbs the high levels of static and dynamic loads from the turbine and generator and transmits it to the framework structure. The spring bodies are still sufficiently rigid in horizontal terms that in dynamic studies of how the building as a whole would behave in the event of an earthquake, the relatively soft lateral framework is 'connected elastically' via the turbine base.

4.2.5 Cooling water supply

The cooling water supply structures form part of the safety-related structures of nuclear power plants, as they still have to ablate residual heat even when a station is not in operation or experiencing any incidents.

The cooling water intake structures, usually right on the bank of a river or on the coastline, in particular, are subject to high requirements in terms of protecting property and being designed to withstand external actions. Being designed to cope with explosions internally, or external aircraft impact, leads to large buildings with extremely thick walls. Alternatively, if they are not so designed, there are discussions as to whether an adequate flow of water can still be guaranteed in the event of a partial collapse (after an aircraft impact, for example). In more recent plants, this has led such buildings to be arranged redundantly, spaced at minimum distances apart, so that they do not need to be designed to withstand an aircraft impact.

4.2.6 Flood protection structures

All plant components that assume safety functions to meet the safety goals as described in Section 2.5 must be protected in such a way that they continue to perform their safety

Fig. 4.7 Sea dyke as flood protection structure

functions even in the event of extreme floods. This calls for a specific plant protection strategy which requires flood protection measures including structural protection measures.

Structural protection measures with their flood protection structures must in principle provide permanent flood protection against design water levels. Alternatively, temporary flood protection measures may be included in the safety strategy if there is sufficient advance warning time. KTA 2207 [23] defines the design water level as the highest water level that can be expected with a probability in excess of 10^{-4} p.a., in front of the protective structure or plant component to be protected.

The main protective measures, especially in coastal areas, include dykes enclosing plant components to be protected against floods. These can be divided into inland and sea dykes, depending on the differing design water levels involved with inland and coastal locations (see also Section 5.3).

Sea dykes are particularly important, as the flood risks involved may be assumed to be relatively high (Figure 4.7) (cf. [24]). For the governing storm flood water level, which consists of the storm flood water level plus wave impact, dykes must be designed to demonstrate a sufficient dyke height, allowing for possible minor wave overflows, and sufficient stability. These proofs are influenced significantly by dyke structure (material) and cross-section with its internal and external slope angle (geometry).

As well as dyke design verification proofs, there must also be a monitoring programme to review settlements at regular intervals, for example annually, which must be assumed as the settlement forecasts with the proofs. If settlements exceed permitted levels, repairs must be made, and as the storm flood risks are greater in autumn and winter, these can only be carried out during the summer months.

4.2.7 Foundations

4.2.7.1 Raft foundations

As a general rule, as with conventional power plants, nuclear power plant structures are laid on raft foundations, but the demands on the subsoil are extremely high, especially under the reactor building, with its high permanent loads combined with the exceptional actions of aircraft impact or earthquakes. Soil compression from permanent loads alone often reaches levels of approx. 500 kN/m^2. If an earthquake occurs, or an aircraft hits a station, levels could exceed 1000 kN/m^2.

This high level of soil compression often also calls for additional theoretical studies looking for weak points, such as cavities, in the soil; these have a major influence when designing the slab on ground. Such non-constant soil conditions must also be taken into account when considering the soil–structure interaction when designing to resist earthquakes.

4.2.7.2 Pile foundations

The monolithic raft footings used in building nuclear power plants largely use large bored piles up to 1.50 m or so in diameter, with piles approx. 4–6 m apart. One particular feature arises if bituminous sealing is used. In that case, the slab on ground is often divided into two separate slabs: the lower pile head slab and the upper building seal slab, between which the sealant is then applied.

Bored piles are often used in foundation work in existing structures, as they are largely vibration-free.

When performing foundation structures in existing buildings, work often has to be done close to, or even over, safety-related underfloor structures. There is usually not much space for large drilled piles in such areas, so that micro-piles or continuous flight auger piles are used, which can be used virtually vibration-free even in the vicinity of safety-related pipes.

Particularly in the vicinity of safety-related pipes in the subsoil, particular attention must be paid to the horizontal loads transmitted by wind, earthquake etc. to avoid putting any additional stresses on the pipes. This can be avoided, for example, by ‘insulating’ piles in the vicinity of pipes or using angled piles if there is enough space.

4.2.8 Physical protection requirements of building structures

As the basis for assessing the structural and other technical, personal and administrative/organisational safety measures in nuclear power plants that operators are required to prove, the component Federal and Federal State authorities issued the ‘Guidelines for the protection of nuclear power plants with pressurised water reactors against impacts and other third-party effects’ [25] on 24.11.1987 and analogous guidelines for nuclear power plants with boiling water reactors on 01.12.1994 [26]; these define the protection goals, the buildings and other plant components to be protected and safety measures required.

These guidelines are unpublished, as they are classified.

Based on the scope of these guidelines, specific safety goals are defined with a wide range of system conditions for pressurised water reactors (PWRs) and boiling water reactors (BWRs).

The structural safety measures of the buildings to be protected relate to their outside walls and penetrations (doors, gates and gratings). Different safety areas must be set up to meet safety requirements.

The structural barriers for the widest possible range of safety areas are defined in terms of wall thicknesses and their reinforcement content; the number of access points must be kept to a minimum.

There are specifications defining doors, gates and gratings for different classes of barriers. These are also classified and unpublished.

4.3 Disposal structures

4.3.1 Disposal requirements

In Germany, radioactive waste is divided into two kinds:

– radioactive waste producing substantial heat [hot waste]
– radioactive waste producing negligible heat [cool waste]

The latter – minimal thermal radiation waste – can be compared with low radioactive waste, and to some extent with moderately radioactive waste. Radioactive waste producing substantial heat comprises highly radioactive and to some extent moderately radioactive waste.

Waste comes from decommissioning and operating nuclear power plants, from the nuclear industry, nuclear research, and, in very small quantities, from medicine and from the *Bundeswehr* (German armed forces), and includes contaminated tools, protective clothing, sludges and/or suspensions.

Radioactive waste producing negligible heat accounts for more than 90% of the total volume of waste, but just 0.1% of the total radioactivity of waste to be put into final storage in Germany.

4.3.2 Interim storage

Under the agreement between the Federal Government of the Federal Republic of Germany and the utility companies of June 2000 and the subsequent amendments to the Atomic Energy Act in April 2002, so-called on-site (decentralised) interim storage facilities were built at nuclear power plant sites between 2004 and 2007.

Decentralised interim storage facilities are those in which burned-out fuel elements are kept under controlled conditions at nuclear power plant sites for relatively long periods before being moved to final storage.

Interim storage facilities can be divided into two basic types:

– **WTI design**
  Lightweight double-bay hall structures, walls approx. 70 cm thick, roof slabs approx. 55 cm thick, double-bay buildings consisting of two halls separated by a partition wall.

  This model is based on the interim storage facilities at Gorleben, Ahaus and Lubmin/-Greifswald (northern interim storage facility).

  Integrated operating areas with two cranes, stored in double rows (Figures 4.8 and 4.9)
– **STEAG design**
  Solid single-aisle hall design with walls approx. 1.20 m thick, roof slabs approx. 1.30 m thick with separate operating building, one crane, compact storage (Figures 4.10 and 4.11).

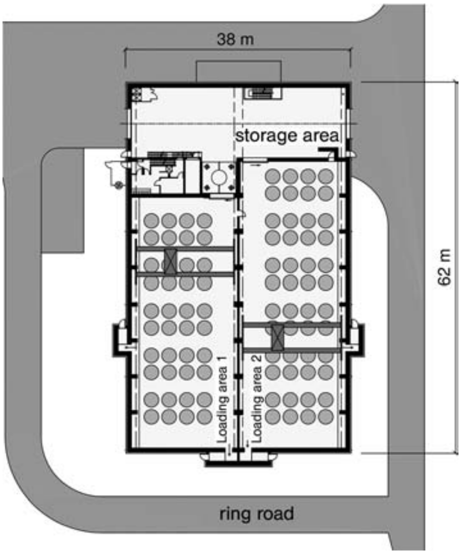

Fig. 4.8 Ground plan, WTI design [27]

The STEAG design was developed in view of using more cost-effective containment models in the future.

In accordance with the multiple barrier principle in nuclear technology the strengthened building structure and future containment generation are designed to serve as additional barriers.

Both models share the same basic features: single-storey reinforced concrete halls with wall and roof slab openings for natural cooling. Inside the halls, a partition wall separates the reception/trans-shipment area from the storage area. Both storage designs have 140 t crane systems; the WTI halls need two of these because of their two-bay structure. For the building design of interim storage facilities see Section 4.3.2.3.

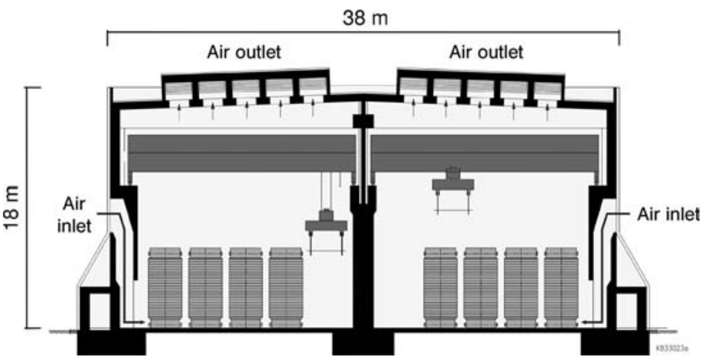

Fig. 4.9 Cross-section, WTI design [27]

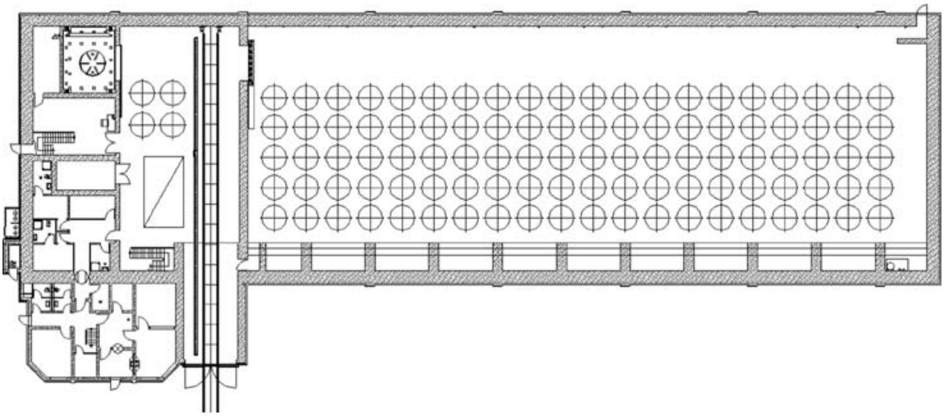

Fig. 4.10 Ground plan, STEAG design [27]

Once taken into store, the containers, which essentially contain irradiated fuel rods, can be described approximately as follows: height 6.50 m, diameter 2.80 m and a dead weight of 125 t. The container walls in the cylindrical and floor areas are approx. 420 mm thick.

The containers are sealed tightly using a cover system, using mainly CASTOR V/19 (Castor: cask of storage and transport of radioactive material) containers to date. These containers can hold up to 19 fuel elements (Figure 4.12).

The top of the container body is stepped to take the cover. At the head and foot of the container body are two overlapping carrying frames to which the storage hall crane lifting gear can be attached.

Containers are transported by rail or road exclusively and delivered to the interim store. Storage containers are transported horizontally to be stored in the interim store.

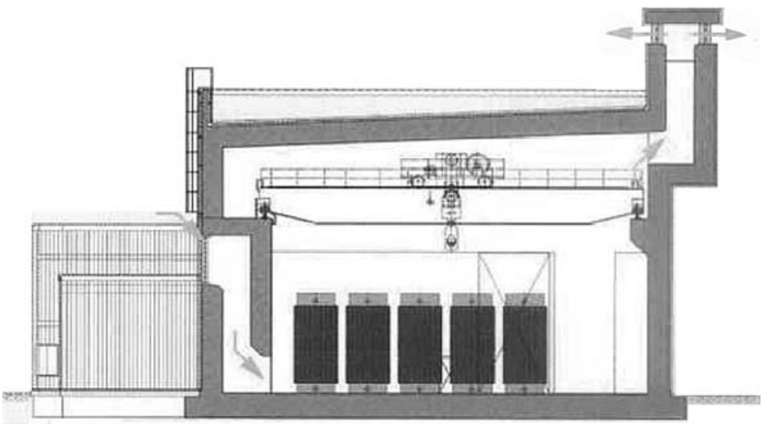

Fig. 4.11 Cross-section, STEAG design [27]

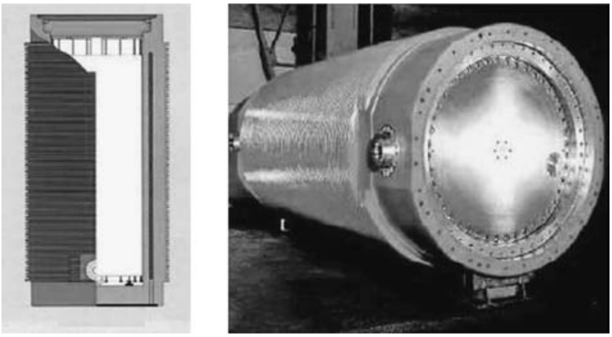

Fig. 4.12 CASTOR V/19 transport container [28]

To unload containers, the storage hall crane attaches to them via the carrying frames provided, and the transporter vehicle takes them. Containers are then driven to their preset storage positions, set down upright and connected to a container monitoring system.

4.3.2.1 Safety requirements

Safety requirements for structural systems can be deduced overall from the statutory requirement to prevent damage and from the safety goals to be complied with.

Specific requirements here are laid down in the nuclear rules, accident rules and KTA safety standards.

The storage building is required mainly to:

– provide shielding
– remove heat
– be designed for operating and exceptional loads
– provide protection against fire and lightning strike
– protect against the weather
– protect against third parties (sabotage).

4.3.2.2 Design criteria

Design criteria are governed by:

– **Shielding**
  Most of the ionising radiation that fuel elements emit is shielded by their containers. The reinforced concrete building structure provides further shielding, keeping radiation levels within the limits laid down by the radiation protection regulations and protecting staff and the environment.
– **Heat removal**
  The interim storage facility design is designed to remove the heat that the fuel elements give off as they decay, by way of natural convection. The air inlets and outlets required must be arranged and dimensioned to remove heat reliably.

- **Building settlement**
  Building settlement due to the container loads involved must not compromise the structure or the operation of the cranes etc. Settlement is estimated technically at the planning phase, allowing for subsequent partial occupation levels, and is monitored in operation via recurrent settlement testing.
- **Structural integrity**
  As with conventional structures, this requirement can be met via the rules of building design on the design of the roof and sealing the building externally, if groundwater conditions allow.
- **Floor structure and decontaminatable coatings**
  The slab and ground in the storage area must have sufficient compression strength and wear resistance to take the containers put into storage. This is achieved by using a mechanically smoothed concrete surface with hardening agents mixed in. In the reception and maintenance area, the floor is given a decontaminatable coating as a precaution. In the loading and unloading zone in the reception area a shock-absorbent layer of so-called damper concrete can be included in the floor slab to protect containers and floor slab if a container is dropped from a height of 3 m, which cannot be ruled out.
- **Durability**
  Interim storage facilities are designed to be permanent in accordance with conventional standards. If they are built properly of tried and tested reinforced concrete designs, they should last for their full working lives.

4.3.2.3 Building design

As we saw in Section 4.3.2, building structures in Germany fall into one of two different designs: WTI and STEAG. These designs differ from one another in particular in terms of their structural design.

WTI Design

The building is designed to withstand exceptional effects from outside, such as earthquakes and blast waves from explosions. They do not need to be designed to absorb aircraft impact, as the containers themselves are designed for this external event.

Exceptional events from inside are containers falling from a height of 0.25 m in the hall area and 3.00 m in the loading area. In the trans-shipment hall, so-called damper concrete is used in areas in which containers could fall, to absorb the energy released, enabling the loads involved to be transmitted without additional strengthening the floor slab at this point.

When floor slabs are occupied by CASTOR containers in blocks of eight, this gives a floor slab loading of 200 kN/m^2.

What is not typical, compared with similar lightweight hall constructions, however, is the roof construction; this has to be 55 cm of normal concrete to be radiation-proof. This high permanent load component means that this design has to have relatively high roof girder constructions.

STEAG Design

The greater roof slab and wall thicknesses of the solid STEAG design will at least protect against penetration from aircraft impact. Unlike the WTI design, such halls can also hold containers designed for a debris load of 2 t at least, should roof sections fall in.

Temperature effects

The relatively high room temperatures of approx. 80 °C mean that the outer walls and roofs must be reinforced accordingly, to guard against a correspondingly high crack width, which must be demonstrated in many areas for centric forces as finally built (with the concrete at its full tensile strength).

The floor slabs are designed not merely for a high load per surface area of up to 200 kN/m^2, but also for hot spot temperature effects of approx. 120 °C immediately below the containers. This makes an additional consideration of the upper reinforcement of the floor slabs necessary. The hot areas cause concentric inherent stresses leading to cracking.

However, non-linear studies of floor slabs made at interim storage facilities have shown that no additional reinforcement is required because of the hot spot effect.

4.3.3 Final storage

The Federal Government of Germany has decided to store radioactive waste in final storage facilities in ‘deep geological formations’ to keep them out of the biological cycle for as long as possible. This decision was taken because of Germany’s population density, climatic conditions and the fact that Germany has geological formations that are suitable for this purpose.

Both hot and relatively cool waste will be stored finally in deep geological formations for safety reasons.

Hot radioactive waste (from spent fuel elements) is more active, so the temperatures that radioactive waste generates are correspondingly higher. The Germans are still looking for the most suitable deep geological formations in which to store them finally.

Studies to date have shown that, highly radioactive hot waste can be safely stored finally in deep geological formations even with today’s state of the art science and technology.

Germany has approved the Konrad shaft as the final storage facility for radioactive waste producing negligible heat.

The salt stock Gorleben site is currently the most studied site for a possible final storage facility for radioactive waste producing substantial heat.

Any further investigations have been interrupted by a politically motivated moratorium since 1 January 2000, and have not resumed to date.

The radioactive waste producing substantial heat obtained at present, such as spent fuel rods, is put into storage at the nuclear power plant sites themselves in interim site storage facilities in CASTOR containers.

Other radioactive waste producing substantial heat is prepared and put into storage in glass moulds at the final storage facility in Gorleben, which is also where the waste returned from the reprocessing plants in France and Great Britain is stored.

For radioactive waste whose thermal radiation is negligible, overground interim storage facilities have been set up as collection and buffer stores and as storage facilities, as no final storage facilities are available.

From 1967 to 1978, radioactive waste producing negligible heat – then called low and moderately active waste – was stored at Salzbergwerk Asse II (experimental final storage facility) under the strategy at that time.

Before it was used as a final storage facility, Asse II worked as a salt mine for more than fifty years; the waste is stored in the chambers excavated in the course of extracting the salt. The prevailing geological conditions led to the salt formations moving and loosening, so water penetrated into the mine, which means that Asse does not fully meet the integrity and stability requirements for a final storage facility. The German Federal Office for Radiation Protection, which operated the Asse final storage facility at that time, believes that the site safety conditions at the time only exist to a limited extent today.

As far back as 2007, the Federal Ministry for the Environment, Nature Conservation and Nuclear Safety instructed the Federal Office for Radiation Protection to refit the former Konrad shaft facility at Salzgitter as a final storage facility for radioactive waste producing negligible heat. The Konrad final storage facility has natural barriers which contain the radioactive waste permanently. Above the final storage facility, there is a covering layer of clay up to 400 m thick which prevents surface water penetrating. The storage areas are between the 800 m and 850 m strata.

Nine storage areas have been approved to allow for the storage space originally applied for of 605,000 m^3. As matters currently stand, two storage areas capable of holding 280,000 m^3 of waste should suffice, as new conservation procedures have reduced the volume of waste involved.

Under the planning approval of 2002, the Konrad final storage facility can hold up to 303,000 m^3 of radioactive waste producing negligible heat. By way of comparison: a single CASTOR container with thermally radiant waste contains more radioactivity than the entire 303,000 m^3 of radioactive waste Konrad is allowed to hold.

By the time all the nuclear power plants in Germany have reached the end of their working lives, it is estimated that a total of approx. 17,000 t of heavy metals will have accumulated as spent fuel elements, that is thermally radioactive waste producing substantial heat.

Final storage facilities for highly radioactive waste could be any geologically stable ground formations such as salt stocks or rock formations. The structural engineering challenges which final storage facilities present are in particular how to build the tunnels required to access the actual storage facilities and designing the storage facilities at great depths.

This model is the most advanced in the world, and was developed at the nuclear power plant site at Olkiluoto, Finland, where the deepest point achieved in the rock formations is approx. 420 m. The highly active waste stored at this depth is fused into glass, and will be enclosed completely in concrete once it has cooled down to some extent.

4.4 Building execution

This section deals with aspects of building design execution which are specific to nuclear power plants, first looking back at the building of more recent nuclear power plants in Germany, which were built in the 1980s. We will also look at experience and current developments in constructing the Olkiluoto 3 (OL3) power plant in Finland.

4.4.1 Site installations

Construction sites for nuclear power plants are some of the largest construction sites there are, employing several thousand people.

A section from the site installation plan for KRB II Gundremmingen can be seen in Figure 4.13. Apart from the site management and workshop buildings, the infrastructure is particularly important: barracks, a canteen to cater for the workers, utility and disposal lines and parking places must be designed and installed.

With the OL3 project, building the nuclear islands took 13 tower cranes at times, a stationary Demag PC 9600 crane with a capacity of 1000 t to install the steel components of the safety containment, plus mobile cranes to lift in the equipment

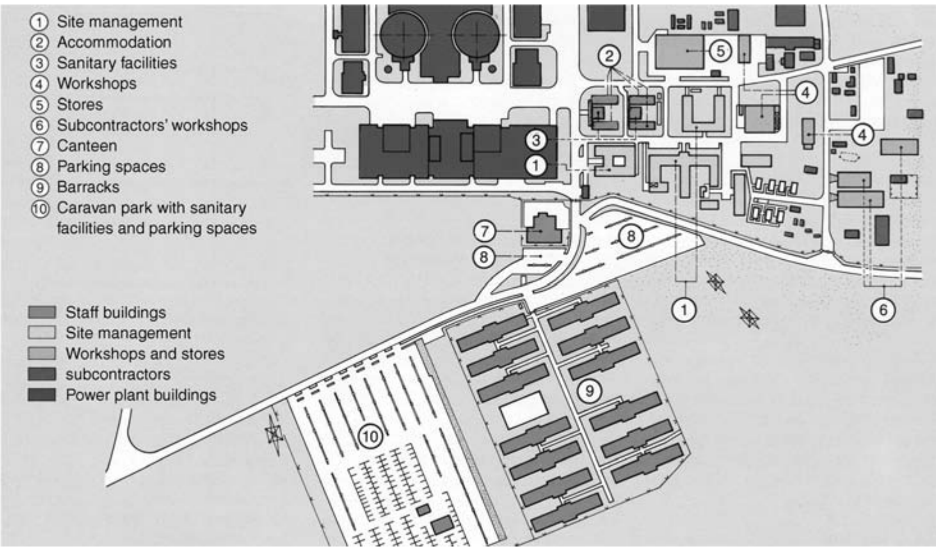

Fig. 4.13 Section of site installations plan KRB II Gundremmingen [17]

Fig. 4.14 OL3, cranes used on nuclear island – reactor building and auxiliary buildings (left) and conventional island (right) [22]

to be used. Three of the tower cranes, two of them inside the reactor building, could not be supplied directly, but had to be served by other cranes (Figure 4.14).

An anti-crane collision system was used at OL3 which analysed where crabs, outriggers and counterweights were and, if need be, restricted adjacent crane movements to prevent them colliding.

The crane layout selected allows all cranes to rotate freely with the crabs run in, at times when they were not in use, such as on rest days or in strong winds.

4.4.2 Project organisation

Organising who is responsible for what and how things should run is of decisive importance when creating a major project. Clients, authorities, inspectors, contractors and designers must be involved in the project in such a way as to ensure that work proceeds perfectly and in an orderly fashion and that quality goals are achieved. Any project organisation is based on contractual foundations, which lay down the rights and obligations of those involved.

The overall project organisation chart of the general contractor in charge of building the OL3 power plant as a whole, the consortium of AREVA NP and Siemens PG, is shown in Figure 4.15.

By way of example, some of the governing tasks in these areas are listed below:

Quality and environment

- Checking subcontractors' quality documents
- Auditing subcontractors' staff and own staff
- Monitoring the work of the other departments, to check that they comply with the quality assurance plan

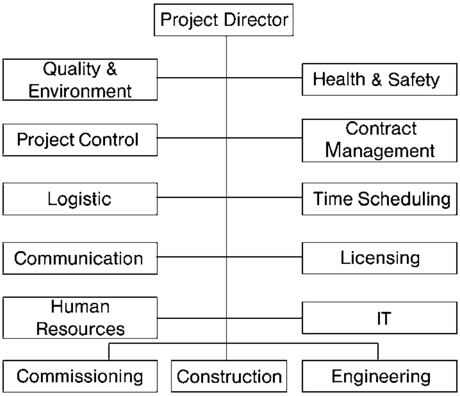

Fig. 4.15 OL3, general contractor's overall organisation chart (as at 08/2009)

- Managing and leading the quality assurance teams for the individual trades
- Training site staff in quality assurance
- Assisting the construction and engineering teams in coordinating specifically with the client and the authorities

Project control

- Bookkeeping
- Contract management for subcontractors' contracts.
- Assisting subcontractors commercially
- Monitoring the subcontractors budget
- Invoicing (to client)

Logistics

- Organising delivery of plant components to site

Communications

- Marketing/public relations
- Organising site inspections
- Producing presentation documents
- Producing and approving site photos

Human resources

- Dealing with staff employed on site

Commissioning

- Managing and coordinating system commissionings

Construction

- Coordinating construction and installation
- Organising and coordinating building construction sections, installing components

Engineering

– Coordinating schedules
– Producing working documents
– Checking subcontractors’ working, concreting and installation drawings
– Producing amendments to drawings

Health and safety

– Producing safety at work instructions for use on site
– Checking safety at work on site
– Reporting involved

Contract management

– Drawing up subcontractors’ contracts
– Negotiating subcontractors’ contracts
– Administrating client’s contract

Time scheduling

– Verifying that subcontractors’ timetables match project timetables
– Assisting construction department with producing specific coordination timetables

The tasks and communications paths between building management, client and subcontractors are shown in Figure 4.16.

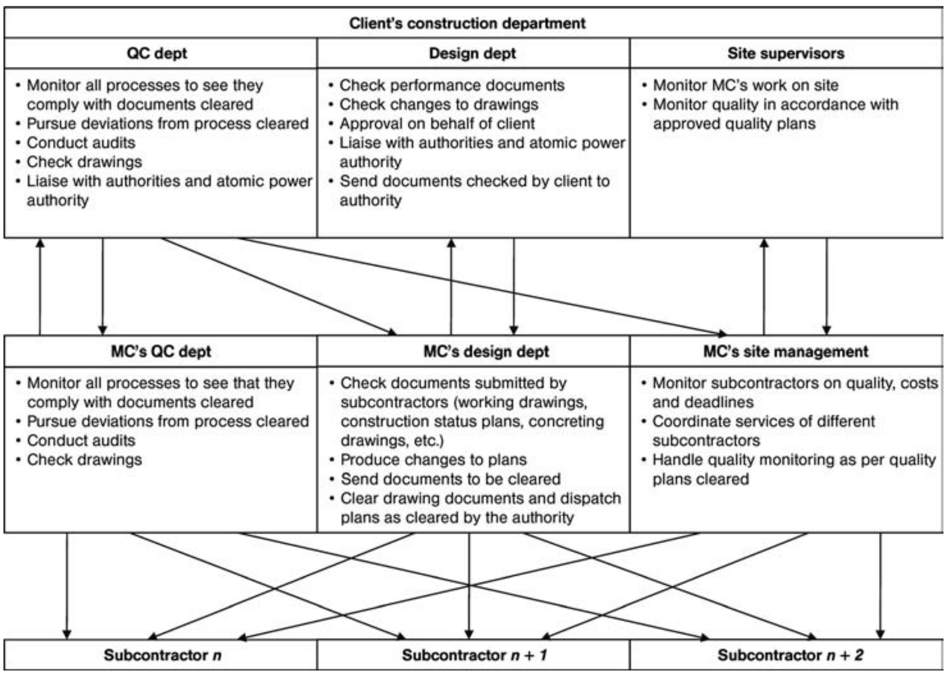

Fig. 4.16 Tasks and communications paths in project management

4.4.3 Quality assurance

Work on nuclear power plant projects is subject to strict quality assurance requirements.

With the OL3 project, as well as the usual design documents, the designers also produced governing documents on quality assurance which were checked by the client and authorities:

- Work specifications, such as defining specific works, indicating training required
- Quality control plan – checklists defining what checks are to be conducted and how, and stating who is responsible in each case

Based on these documents, the contractors involved produce work plans, which essentially extrapolate the designers' quality control plans with specific construction aspects (materials, work rates, etc.).

Work plans are produced based on the overall quality assurance system, referencing the underlying performance documents. In substantive terms, they include the working resources and procedures required to perform tasks, the project organisation stating who is in charge of performing work and what to do in the event of problems. They also include risk assessments on individual relevant issues.

4.4.4 Formwork and scaffolding

The average power plant block involves erecting formwork for approx. 500,000 m^2 of concrete surface [29].

Precisely in terms of time and costs, it is essential to plan the use of formwork and scaffolding beforehand, as this may affect the performance schedules that the designers produce, such as producing evidence of specific building conditions and additional reinforcement resulting. With complex construction projects, contractors specialising in planning, constructing and providing formwork are involved at an early stage.

The aim in principle is to use formwork and scaffolding elements which are as large as possible and can be used frequently, even if building power plants often involves constructing irregular shapes with variable slab thicknesses and formwork heights.

Making the cupola of a reactor building in a pressurised water reactor presents particular demands. At the Philippsburg 2 nuclear power plant, the safety enclosure of steel plate under the cupola could not withstand any major stresses, so the concreting load had to be borne by projecting formwork construction (Figure 4.17).

When building nuclear power plants, slipforming can be used not only for box- and annular-shaped sections such as chimneys, but also in building large freestanding walls, making consoles without further ado or 'slipping in' cutouts. The slipforming method was adopted when making the bioshield at the Krümmel nuclear power plant. Using heavy concrete and the many cutouts involved had to be included in considerations.

The OL3 construction project used climbing formwork and/or self-climbing formwork for the more standard building structures such as safety containment and aircraft impact structures (Figure 4.18).

Fig. 4.17 Cupola formwork, Philippsburg nuclear power plant [17] (left), section through cupola formwork (right)

The compact layout of the nuclear island components calls for using a special single-headed formwork. The walls enclosing the UKA, UFA, UKS, UJH and UKE construction modules are separated in some cases by as little as 30–40 cm. These narrow spaces must be kept clear at all times.

The confined working space involved rules out double-headed formwork for whichever module comes later in time. Tying and releasing ties on double-headed formwork on the outside of the enclosing wall would be impossible, as the working space required is not available.

The solution adopted in this case was therefore as follows: on the inside of the enclosing wall of the following module, the ‘Trio’ framework formwork section system by the Peri company is used, which transmits its load via Peri SB framework sections to the

Fig. 4.18 OL3, UFA building, using large-format wall girder formwork sections [22]

lower ceiling or wall–ceiling node point of the lower level. The load is led into the concrete via cast-in tension bars.

The external formwork was made via a special steel formwork section pre-stressed against the outer wall of the preceding building.

The formwork is installed, fixed and removed from the top.

Formwork can be removed once concreting is complete without leaving parts in the join.

4.4.5 Other particular construction features

In what follows, we will present some other particular features of construction which are particularly characteristic of building nuclear power plants.

4.4.5.1 Reactor building – containment

The pre-stressed concrete containment in the OL3 reactor building is made of K60 concrete to Finnish standard BY50 (comparable with C50/60) with a steel liner. The cylindrical section has walls 1.3 m thick. The inner steel liner, made of S355J2SN steel, is 6 mm thick. The pressure vessel has an internal radius of 23.40 m and an outer radius of 24.70 m.

To make the cylindrical section of the steel liner, 90° sections 6 m high were delivered to the site. These sections were then assembled to form rings 12 m high and were lifted into place (Figures 4.19–4.21).

Before being lifted into place, segments were coated with epoxy resin based triple-layer paintwork (basecoat, intermediate coat and topcoat).

Once each liner segment was lifted into place, it was welded to the segment below it. Once it was welded, the reinforcement and tendon sleeve tubes were installed. Tensioning blocks for the horizontal tendons of the containment were spaced 120° apart.

Fig. 4.19 OL3, Assembling the steel liner on site [22]

Fig. 4.20 OL3, lifting in the liner ring [22]

Vertical reinforcement joints were made using overlapping or Lenton screwed sockets. The horizontal reinforcement joints were made mostly with overlaps. The connecting reinforcement for the anchor plates integrated in the steel liner (such as polar crane consoles) was made with back closed stirrups, tying the anchor plates to the stirrups with position sockets.

Fig. 4.21 OL3, lifting in the liner dome [30]

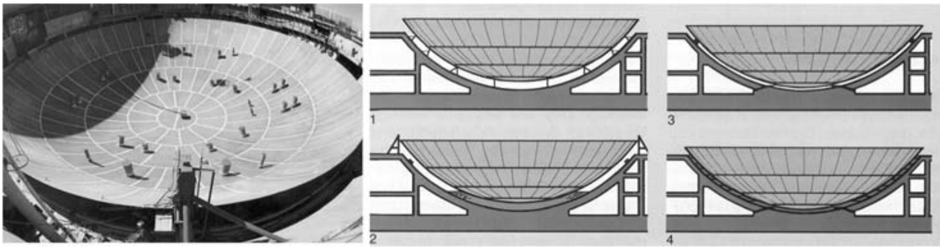

Fig. 4.22 Lower embedded section of safety containment (spherical steel segment) (left), floating on and underfilling the steel shell (right) [17]

The cylindrical section of the safety containment was shuttered using single-headed self-climbing formwork. The formwork was supported against the structure of the preceding outer containment (APC shell).

One particular structural engineering feature of pressurised water reactors made in Germany is installing the lower steel cap of the steel containment.

The lower section of the spherical steel containment was first made supported on trestles in the spherical concrete segment, so it could be welded on both sides. It was then lowered floating into its final position defined by spacers. Lastly, the cavity remaining was then carefully filled with injection mortar (Figure 4.22).

4.4.5.2 Embedded parts

The large number embedded parts involved (a nuclear power plant block may have more than 100,000 anchor plates) calls for a particular feature of planning, such as recognising collisions in good time and avoiding them, and particular preparations on site to ensure that they can be finished on time in parallel with the formwork and reinforcement work. As well as the anchor plates just mentioned, fitted components include such items as pipes, foundation frames and the frames for Omega water stops, known as Omega frames. These are attached to the formwork or to special support structures, and this must be carried out in such a way as to maintain the tight tolerances in terms of precision location once concreting is complete.

With the OL3 construction project, the anchor plates used to fix components later on are made largely of ferritic steel anchored by headed studs (Figure 4.23).

These anchor plates arrive on site coated with rust-protection base coat, and are painted in the finishing phase. The plates are painted once again once the load-bearing structure is in place.

Pipe lead-throughs of ferritic or austenitic steel are installed in the first- or second-cast concrete. Fitting them at the second-cast concrete stage means an extra work process before handing over to the mechanical trades, which could delay the latter starting; but the installation quality is generally higher in terms of precision.

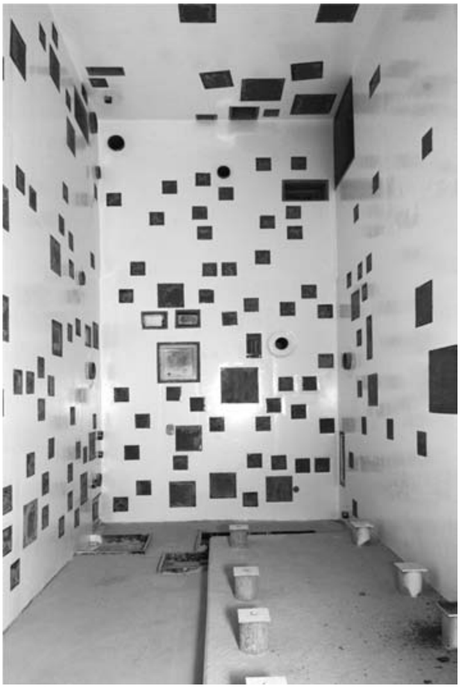

Fig. 4.23 OL3, Wall view with embedded parts in the UFA building [22]

4.5 Dismantling

Dismantling nuclear power plants represents a major part of nuclear engineering in Germany today.

Dismantling principles

We need to distinguish here between the systems and structures inside the control area – which could be contaminated or live – and those outside the control area, which are not.

Dismantling the control area, with its contaminated and live sections, breaks down into stages, as follows:

- Shutting the plant down, residual operation, deconstructing the contaminated systems not required for residual operations and making changes to systems and building sections as necessary as takedown proceeds
- Taking down the contaminated, active primary components and the concrete structures which are live from being irradiated for years, and the bioshield in particular
- Demolishing the remaining systems, decontaminating buildings, conducting clearing surveys on buildings and external areas

This is aimed at decommissioning the plant as a whole of supervision according to Atomic Energy Act terms.

This is followed by conventional demolition of both the now cleared former control area and the systems and buildings outside the control area.

Weights and costs

For a typical PWR, the demolition and disposal weights estimated in tonnes [t] (internal estimate by HOCHTIEF [31]) are as follows:

- Total power plant: 500,000 t
- Control area: 156,500 t, of which:
  - structural components: 143,000 t
  - system components: 13,500 t

The remaining radioactive waste for final storage is estimated at 4,000 t.

Dismantling costs, excluding residual operation, materials handling and packaging costs – may be estimated roughly at €350 m per power plant block [32].

4.5.1 Legal foundations and rules

Under the Atomic Energy Act [33] (AtG) §7 – approving plant – para. 3, consent is required to decommission plant and safely contain the ultimately decommissioned plant or demolish plant or sections of plant.

Apart from the Atomic Energy Act, there are other statutory foundations and nuclear regulations to be considered:

- Radiation protection regulations (in German: StrSchV)
- Atomic Energy Act procedural regulations (in German: AtVfV)
- Law on environmental compatibility testing (in German: UVPG)

The ‘decommissioning guidelines’ (guidelines for decommissioning, safe containment and demolition of plant or plant components under §7 of the Atomic Energy Act) [34] are designed to bring the relevant aspects of approval and regulation together. It is also intended to create a common understanding between the Federal Government of Germany and Federal States on proper performance and harmonising existing views and methods.

There are also BMU guidelines, reactor safety committee (RSK) recommendations, Nuclear Safety Standards Committee rules (KTA), radiation protection committee (SSK) rules and relevant conventional rules to be taken into account when planning and implementing dismantling.

4.5.2 Decommissioning strategies

There are basically two decommissioning options to choose from when dismantling nuclear power plants:

- Dismantle immediately, as soon as the rundown phase has been completed
- Safe confinement: after the rundown phase, put the nuclear power plant into ‘safe confinement’ for around 30 years before starting to demolish it

Immediate dismantling

- Demolish all contaminated and active building sections, systems and components immediately
- Prepare and pulverise all radioactive waste for interim or final storage
- Decontaminate and release other remains
- Decontaminate and release building, demolish conventionally

Safe confinement

- Demolish all contaminated structures, systems and components outside the containment area immediately
- Reduce control area and prepare and pack radioactive waste involved for interim or final storage.
- Decontaminate and clear other residuals involved
- Clear media (press, TV etc.) if possible
- Leave active structural components (nuclear installations, pressure vessel, bioshield) as installed, seal system interfaces appropriately
- Continue to operate essential systems during safe confinement (ventilation, pressurisation, monitoring systems)
- Confine safely for 25–30 years
- Apply for dismantling permit during safe confinement phase (around five years before safe confinement ends)
- Create new infrastructure facilities
- Demolish and clear plant as with immediate demolition
- Timescale:
  Establish safe confinement: 5–8 years
  Operate in safe confinement mode: 25–30 years
  Demolish completely: 8–10 years

In Germany, the only nuclear power plants that have been put into safe confinement are Lingen (KWL) and Hamm-Uentrop (THTR). In the light of experience gathered with dismantling projects to date, the prevailing view today is that starting dismantling as soon as the rundown phase is complete is preferable. The advantages include: the nominal costs of direct dismantling are less than those of safe confinement, plant personnel are still on hand, personnel can continue to be employed and the site can be available to be reused sooner if required. The considerations in favour of safe confinement include reducing potential activity in the plant from radioactive decay, possibly using technical innovations and developments and reducing immediate costs.

4.5.3 Dismantling phases

The process of decommissioning a nuclear power plant until when it is deregistered under the Atomic Energy Act, including the rundown phase in direct dismantling, takes around 12 years.

We will now describe the individual phases of the process, taking the Stade nuclear power plant (KKS) as our example (Figure 4.24).

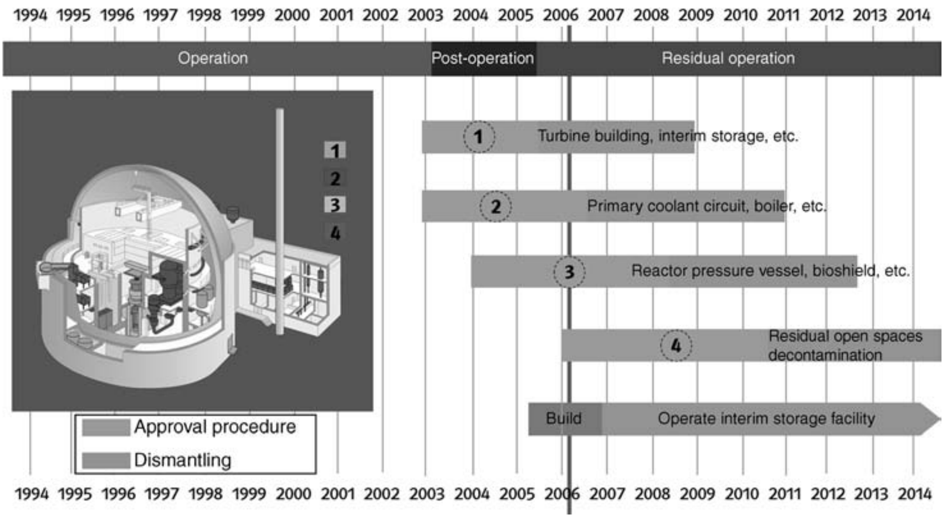

Fig. 4.24 Stade, dismantling timetable [32]

Dismantling phases, using KKS as example:

Phase 1:

- Take down various systems inside and outside control area
- Put different buildings to different uses
- Set up dismantling infrastructure

Phase 2:

- Remove main coolant lines and pumps
- Remove boiler and other large components
- Remove other contaminated system components

Phase 3:

- Make preparations to remove and pack activated building sections and components
- Remove and treat reactor pressure vessel fittings
- Remove reactor pressure vessel
- Remove bioshield

Phase 4:

- Remove remaining system components within control area
- Decontaminate and clear standing structures
- Withdraw from decontaminated areas in stages and seal against recontamination
- Clear site
- Release complete site from Atomic Energy Act monitoring

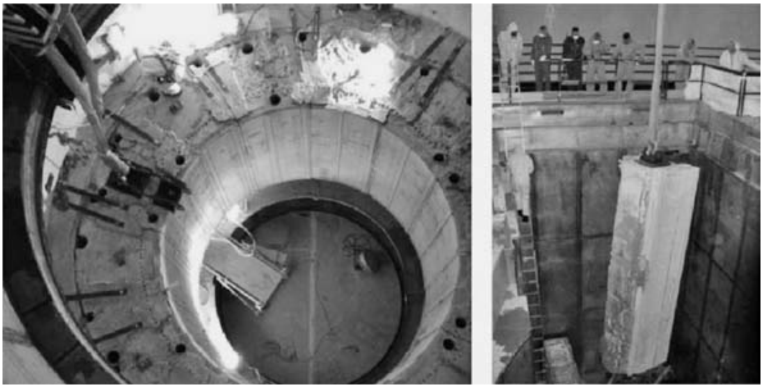

Fig. 4.25 Removing bioshield using reinforced concrete blocks previously obtained by using wire saws using HOCHTIEF LAP 60 heavy load anchor as retrofitted load attachment point

4.5.4 Individual structural measures involved in dismantling

We list some individual building services below which those involved in dismantling will normally have to carry out [35]:

- Reconstruction measures: fit extra shields, create openings, close openings, take fire precautions
- Strengthen building components to take increased loads due to temporary interim conditions, for example
- Create routes for transport logistics within control area while removing contaminated and activated system components and logistics work retrofitting lifting gear, including creating access roads required
- Removing contaminated concrete structures: establishing depth and extent of contamination levels, cutting off and removing contaminated building surfaces and packing in transport containers
- Removing activated concrete structures (bioshield)
- Measures to prevent building work problems: making safe against falling loads
- Setting up temporary buildings for treating and storing radioactive waste, clearance measurement of system components released completely and unconditionally if classified accordingly
- General service functions such as scaffolding or client providing site electricity supply

4.5.5 Structural demolition technologies

There are a number of criteria to consider when selecting the right demolition procedure:

- Technical criteria: component materials, geometry and accessibility
- Radiation protection criteria: minimising aerosol release, primary and secondary waste, avoiding spreading decontamination, ease of decontamination, high level of recyclability of installations used
- Financial criteria: setup costs, equipment costs, operating costs, cutting services

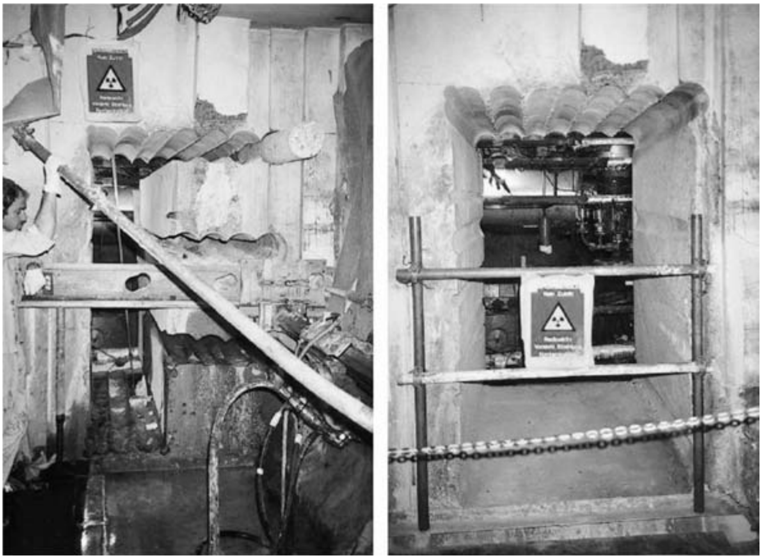

Fig. 4.26 Making a cut-out in the control area using wire saws and overlapping core drillings

– Strategic criteria: site location, demolition strategy, disposal routes, constraints of materials handling strategy

Demolishing concrete and reinforced concrete structures is often done using wire saws. These have the advantage that large-format blocks can be obtained which can be readily carried away; the drawbacks are that cutting and cooling water may be required and drill holes have to be made to guide the wires in first (Figure 4.26).

Contaminated concrete layers can be demolished as shown below, depending on the nature and depth of the contamination involved (Table 4.2, Figure 4.27).

Table 4.2 Methods for decontaminating reinforced concrete sections

Nature and Depth of Contamination	Method
Loose contamination on surface of concrete which can be wiped off	Vacuum, brush off, wipe off, wash or spray, apply chemicals
Subsurface contamination (has penetrated and attached itself)	Grind off, mill over large areas (Figure 4.27) flame hammer and chisel
Deeper contamination into concrete	Chisel off conventionally, mill over large areas, combine milling and chiselling

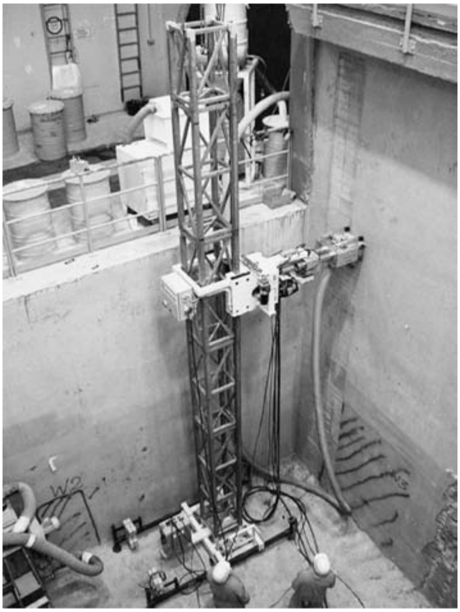

Fig. 4.27 HOCHTIEF Decon surface milling system in use [36]

5 Extraordinary actions involved when designing nuclear installations

5.1 Overview

Compared with designing conventional structures, designing system components and structural systems for nuclear power plants is subject to the maximum safety requirements, which means safety systems for managing incidents must be designed to withstand extraordinary actions at safety levels 3 and 4 (cf. Section 2, Table 2.2 and DIN 25 449 [15]). These rare and extremely rare actions are divided into internal and external actions.

A summary of internal and external actions appears in Table 5.1. Typically, internal factors are induced by:

– leaks or fractures in pressurised pipes (e.g. jet loads and differential pressures)
– problems and incidents while handling fuel elements (e.g. dropped load scenarios)
– internal plant events such as fire, explosion or flood (e.g. temperature or pressure differences).

External actions break down into:

– natural actions which occur extremely rarely, such as 1 in 100,000 year earthquakes which occur according to KTA 2201.1 [37] and 1 in 10,000 year flood effects to KTA 2207 [23]
– man-made actions due to specified airplane crash and explosion pressure wave.

5.2 Internal factors

5.2.1 Leaks and ruptures of pipes

The impact of leaking/broken pipes must be taken into account in accordance with the underlying safety strategy for a plant. German RSK guidelines [5], for example, require a leak of 0.1 A (where A is the open cross-sectional area of the pipe in question) to be assumed in relevant pipes, such as main coolant pipes, for example, leading to jet loads and differential pressures in combination with increasing temperatures.

Jet loads are caused by the impact of the oncoming medium, and act as concentrated loads on the structural member involved. They are expressed as load–time functions or as static equivalent load, stating the impact area, load distribution and impact angle. Figure 5.1 shows the idealised function of jet load over time.

Leaks or ruptures in pressurised pipes induce pressures in the spaces affected which act as loads per unit area over time on the structural members and pressure differentials. What has to be taken into account here is how the differential pressures behave over time, as Figure 5.1 shows in idealised form.

Design and Construction of Nuclear Power Plants. First Edition.
Rüdiger Meiswinkel, Julian Meyer, Jürgen Schnell.

Table 5.1 Extraordinary actions (internal/external)

Internal/External Events			Consequences
Design incidents (safety level 3)	Internal actions	Pressurised components leaking or broken	Jet loads, differential pressures, support and retention forces, whipping pipes, debris loads, temperatures, water pressure (static)
		Problems and incidents while handling fuel elements	Falling loads
		Fire or explosion inside plant	Pressure and temperature differentials
		Flooding internally	Water pressure (static)
	External actions	Earthquake	Mass forces due to self weight of structural components and fittings (components), debris loads, displacements, blast waves due to bursting pressure vessels with high energy content which are not designed against earthquake.
		Flood	Water pressure (static)
Beyond design events (safety level 4a)	External actions	Airplane crash	Direct to the surface area hit and induced vibration, secondary impact of falling debris
		Explosion pressure wave	Pressure load affecting the whole building structure, with pre-specified time sequence and induced vibration

Combined with the jet loads and pressure forces involved, leaking or broken pipes can increase room temperature and hence structural member temperature. The temperatures in the structural components affected increase subject to a time delay, so that the temperature curves in those structural members must be recorded to obtain a realistic overlap of the jet loads or differential pressures with their associated temperature effects over time.

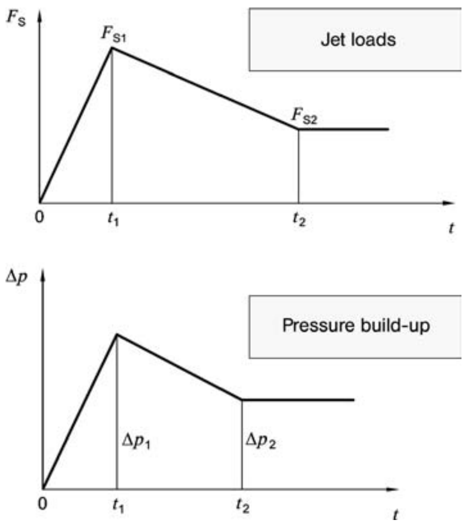

Fig. 5.1 Internal factors (EVI), jet loads and differential pressures

5.2.2 Other internal installation events

Potential problems and incidents when carrying fuel elements must be considered in the course of the fuel handling process. This mainly involves the consequences of dropping a load, which could happen while handling fuel elements in the fuel element storage pool, including loading fuel element containers or moving them around in the reactor building or interim fuel element storage. In an interim fuel element storage, for example, the possible effects of fuel element containers being dropped must be covered which could occur in the delivery area when lifting fuel element containers off carrier vehicles or in the storage area itself when moving fuel elements by crane.

Other internal installation events could include fires, explosions and flooding which could also occur. This calls for specific plant studies to show where such events could occur and what might be the impact of those events in terms of differential temperatures, pressures and flooding heights.

5.3 External actions

5.3.1 Earthquakes

5.3.1.1 General notes

For any nuclear installation, the risk of earthquakes at the location concerned must be assessed in principle and it must be designed to deal with seismic effects. Details here can be found in the relevant IAEA Safety Standards (cf. Section 3.4.1) and corresponding national rules and regulations, such as the German KTA 2201.1 [37], which many other countries also use.

Earthquakes can be defined as shocks to solid rock emanating from an underground source (hypocentre) attributable to natural causes. Earthquakes can be divided into a number of types, depending on what causes them:

- **Collapse earthquakes**
  When underground cavities suddenly collapse
- **Volcanic earthquakes**
  Incandescent molten rock rises to the surface from inside the Earth under high pressure
- **Tectonic earthquakes**
  Sudden violent shifts of rock strata along geological fault lines or faults; with faults, there are three basic kinds of movement: gravity faults, upthrusts and horizontal faults.

In what follows, we will concentrate on tectonic earthquakes, as they account for more than 90% of all earthquakes (cf. [38]). The effects of such tectonic earthquakes, which induce seismic effects, manifest themselves in considerable amounts of energy being released, due to the rock strata shifting. From the earthquake hypocentre, shock waves spread out at different speeds and amplitudes, referred to as compression or primary waves (P waves) and shear or secondary waves (S waves). These shock waves can also be recognised in recorded acceleration time displacements (Figure 5.2). The earthquakes themselves which trigger these waves can be defined and/or quantified either by their magnitude or their intensity. Magnitude, which is normally used as local or close earthquake magnitude (M_L), measures the energy released at the hypocentre of the earthquake underground. This scale was introduced by C. F. Richter in 1935, and is therefore often referred to as the Richter magnitude, or magnitude on the Richter scale. This magnitude is obtained as the logarithm of the maximum deflection of recorded seismograms, allowing for the distance to the hypocentre (Figure 5.3). That means each additional unit of magnitude increases the energy released by around approximately 30 times. One of the greatest earthquakes recorded to date occurred in Alaska in 1964, and reached a magnitude of around 8.8.

Intensity can be defined as the impact of an earthquake at a given location on the surface of the Earth (normally a land surface) as a function of its magnitude at a given hypocentre depth. Intensity is a measure of the impact of seismic waves and dislocations at the surface of the Earth on people, objects and building structures. The strength of these effects is classified in qualitative terms based on the effects observed in a limited area. Intensity is divided into 12 degrees, which are defined as macro-seismic scales, such as the MSK scale (Medvedev–Sponheuer–Karnik; cf. Table 5.2) or the EMS scale 1998 (European Macroseismic Scale). Comparing two earthquakes of the same magnitude but whose hypocentres are at different depths (shallow and deep hypocentres) shows that earthquakes are more intensive the closer their hypocentre is to the surface.

The level of earthquake governing earthquake design, or design basis earthquake, is given generally by the intensity to be expected for the site. In line with this site-specific intensity, with its associated ground movements (accelerations, velocities, displacements), a ground response spectrum must be defined as the basis for the further design of building structures or components. Such a response spectrum, in the form of an acceleration spectrum, represents the maximum acceleration amplitudes of the

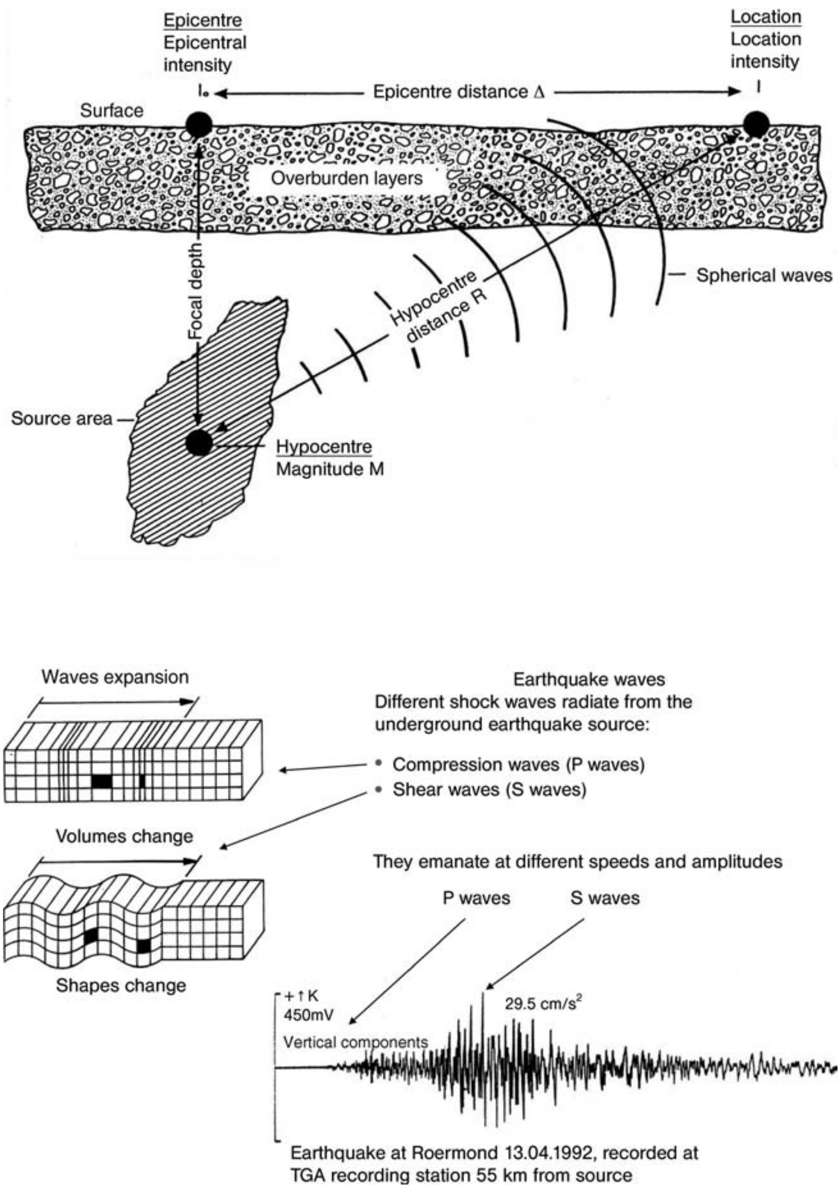

Fig. 5.2 Earthquake waves spreading out

vibration of single mass oscillators with different eigenfrequencies and damping in response to a non-stationary excitation (Figure 5.4).

5.3.1.2 Defining seismic actions

When designing conventional building structures for seismic design, DIN 4149 [39] or DIN EN 1998 [40] gives ground response spectra as a function of rigid body

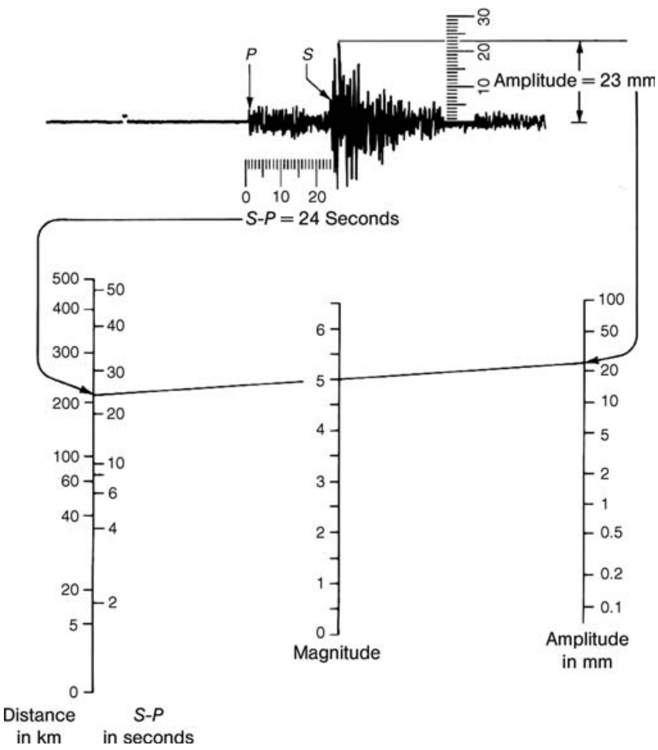

Fig. 5.3 Classifying earthquakes on the Richter magnitude scale [38]

acceleration and the nature of the subsoil. The rigid body acceleration is defined based on the specific German earthquake zone map and represents the intensity at a given location at an exceedance probability of 1/475 a $\approx 2 \cdot 10^{-3}$/a. Other European countries have their own national earthquake zone maps.

Reference earthquake standards for nuclear installations are necessarily more stringent. As opposed to DIN 4149 [39] or DIN EN 1998, KTA 2201.1 [37] requires a reference earthquake intensity for an exceedance probability of $1 \cdot 10^{-5}$/a to be used. Establishing this calls for highly detailed studies as part of a seismological expertise.

KTA 2201.1 requires the design basis earthquake to be defined based on deterministic and probabilistic analyses. The outcome of these analyses, the requirements for which are defined in KTA 2201.1 is a ground response spectrum for both horizontal axes and one for the vertical component. These spectra are taken as free field response spectra for a reference horizon normally defined as the top of the ground.

5.3.1.3 Structural analysis

For earthquake design purposes, KTA 2201.1 [37] divides components and building structures into three classes, as follows:

Table 5.2 Macroseismic intensity scale MSK 1964

Intensity	Observations
I	Detectable by earthquake recording instruments only
II	Felt by a few people at rest only
III	Felt by a few people only
IV	Widely felt; cutlery and windows shake
V	Hanging objects swing back and forth; many sleepers wake up.
VI	Slight damage to buildings, fine cracks in plaster
VII	Plaster cracks, walls and chimneys split
VIII	Major cracks in masonry, gables and roof cornices collapse
IX	Some building walls and roofs collapse; ground tremors
X	Many buildings collapse; cracks open in ground up to 1 m wide
XI	Widespread cracks in ground, avalanches
XII	Major changes to the surface of the Earth

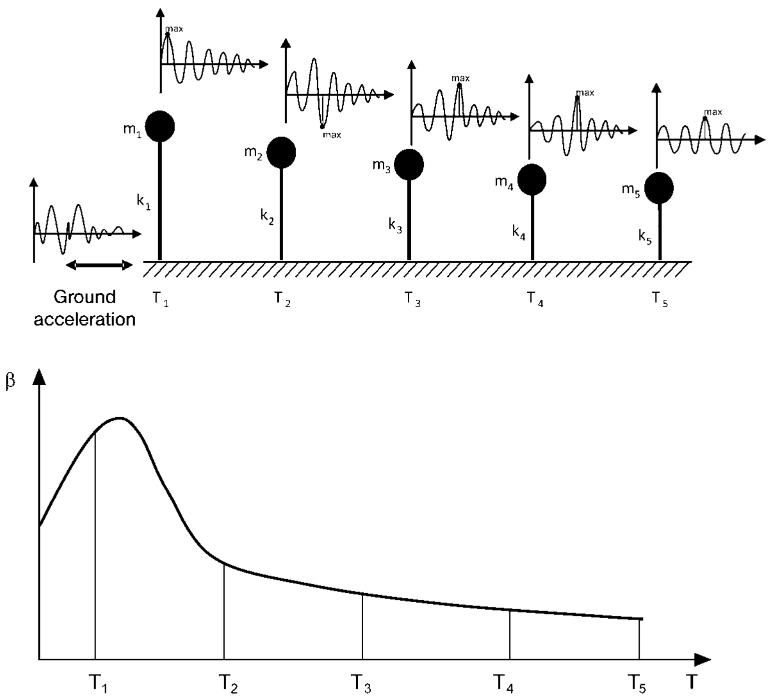

Fig. 5.4 Response spectrum

- **Class I**
  Components and building structures that are required to fulfil the protective goals (control radioactivity, cool fuel elements and contain radioactive substances) and limiting radiation exposure (safety-related system components and building structures)
- **Class IIa**
  Components and building structures that do not belong to Class I, but which, due to their own damage and the sequential effects, possibly caused by an earthquake, could detrimentally affect the safety-related functions of Class I components and building structures
- **Class IIb**
  All other components and building structures

The only components and building structures for which seismic safety is required are those in Classes I and IIa. Components and building structures of Class I must be verified in terms of load-carrying capacity, integrity and functional capability, i.e. deformation or crack widths in reinforced concrete must be limited in some cases. For components and building structures of Class IIa, generally verification of load-carrying capacity will be sufficient.

To verify earthquake safety, structural analyses are required reflecting the design basis earthquake and its possible consequences. Possible consequences could include the failure of high-energy containers, not designed to withstand earthquakes, such as feed water tanks in the turbine building of a PWR plant. Combined effects of earthquakes and other extraordinary actions are not generally taken into account as they are extremely rare.

For structural analysis purposes, earthquake effects are to be set as the ground response spectra for the reference earthquake or compatible recorded acceleration over time curves in each case, recording the simultaneous excitation in both horizontal and the vertical direction. The subsequent superposition of parallel stress variables can be taken either as the root of the sum of the quadratics or the superposition rules as in DIN 4149 [39] or DIN EN 1998 [40].

Structural modelling is subject to particular requirements, due to the dynamic effects and to the influence of the subsoil at the site in particular. Precise details of structural modelling, including details of structural damping and subsoil modelling can be found in KTA 2201.1 [37], KTA 2201.2 [41], KTA 2201.3 [42] and KTA 2201.4 [43].

In principle, the structural models to be used for the building structure, including the subsoil for the plant components with their support structures are those which record how the structures behave in the governing frequency range of an earthquake. Depending on the purpose of verification involved, it must be decided whether structural modelling requires a level beam model or a spatial beam model or even a spatial surface structure model, allowing for possible decouplings between the building structure as a whole and part structures or decoupling criteria between the building structure as a whole and components.

As far as the dynamic behaviour of the structure is concerned, the influence of the interaction between structure and subsoil (subsoil–structure interaction) must be taken into account, varying the soil characteristics to give a lower, medium and upper subsoil strength. The results of the calculations at different subsoil strengths must then be included.

The structural analyses can be carried out using the usual dynamic calculation methods, including in particular the response spectrum method, frequency range method, time history method and the quasi-static method as a simplified method. These are generally used as linear methods. Non-linear methods such as non-linear time history methods are also used in exceptional cases.

The result of the dynamic structural analyses, as well as eigenfrequencies, is to give the internal forces and deformation variables required to assess the strength and deformation behaviour of the structure studied. Response spectra can also be calculated at the intersections with other building structures or components to use these to analyse the building structures or components meeting at these nodes. The resulting method to be used in conducting structural analyses of building structures and components with a view to using response spectra is therefore as follows (cf. Figure 5.5):

– Specify the site excitation as ground response spectra or time history (primary response/primary spectra)
– Calculate the response over time or response spectra of the structure (secondary response/secondary spectra)
– Calculate the response over time or response spectra for system components (tertiary response/spectra)

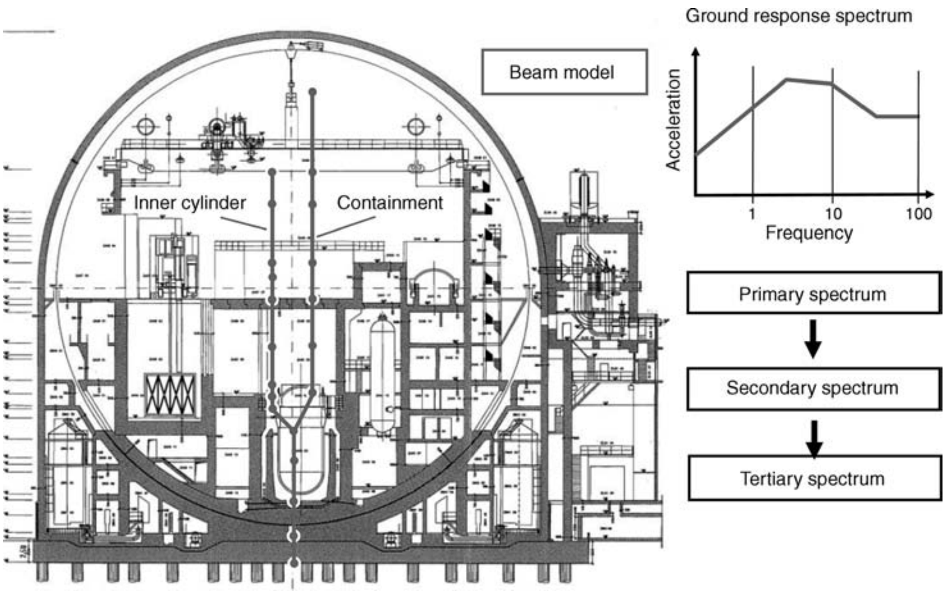

Fig. 5.5 Response spectrum method (building structure/components)

5.3.2 Floods

5.3.2.1 General notes

As we saw in section 4.2.6, protecting nuclear power plants against floods as in KTA 2207 [23] involves allowing for a reference flood level with an exceedance probability of 10^{-4}/a, often also referred to as a one in 10,000 year flood. By way of comparison: normal flood protection is based on a flood occurring at a frequency of 10^{-2}/a (100-year flood); one in 10,000 year floods are only considered for high risk potential systems, such as dams.

The methods used in calculating the reference flood level with an exceedance probability of 10^{-4}/a for inland and coastal sites, including sites on tidal flows (such as the upper Elbe or Weser rivers) are different. For coastal sites, the reference water levels can be determined directly using storm tidewater levels. For inland sites, on the other hand, we need to calculate the flood runoff from which we can then obtain the design basis water levels using suitable methods. KTA 2207 describes methods both for determining the design basis flood runoff at inland sites and to determine storm tide water levels.

5.3.2.2 Inland sites

For inland waterways, KTA 2207 [23] assumes a flood runoff with an exceedance probability of 10^{-4}/a. This flood runoff can be determined either purely on a basis of probabilities or by extrapolating from statistics available. KTA 2207 uses this extrapolation, which is based on the Kleeberg and Schumann method [44]. This extrapolates from a peak level water runoff with an exceedance probability of 10^{-2}/a to a peak level water runoff with an exceedance probability of 10^{-4}/a.

This flood runoff value obtained, finally, gives the design basis water level from a corresponding water level runoff relationship for the location concerned.

5.3.2.3 Coastal sites

KTA 2207 [23] defines the reference water level for coastal sites and sites on tidal flows as a storm tide water level with an exceedance probability of 10^{-4}/a. This storm tide water level $\mathrm{SFWH}_{(10^{-4})}$ can be obtained using suitable but highly laborious probabilistic methods, which can also be used to determine flood runoffs (cf. [45]). Alternatively, according to the annexe to KTA 2207, a probabilistic based extrapolation method can be used, taking the storm tide water level $\mathrm{SFWH}_{(10^{-4})}$ as the total of a base value $\mathrm{BHW}_{(10^{-2})}$ and an extrapolation difference ED as follows:

$$\mathrm{SFWH}_{(10^{-4})} = \mathrm{BHW}_{(10^{-2})} + \mathrm{ED}.$$

The design basis water level $\mathrm{BHW}_{(10^{-2})}$ with an exceedance probability of 10^{-2}/a is calculated here based on a quantitative statistical extreme value analysis. The spread of the results with the usually long, good-quality time series of water levels on the coasts and in tidal flows is relatively low.

Determining the extrapolation difference ED calls for detailed studies of the coastal or estuary levels of the tide flows concerned. At the water gauge sites of Cuxhaven and

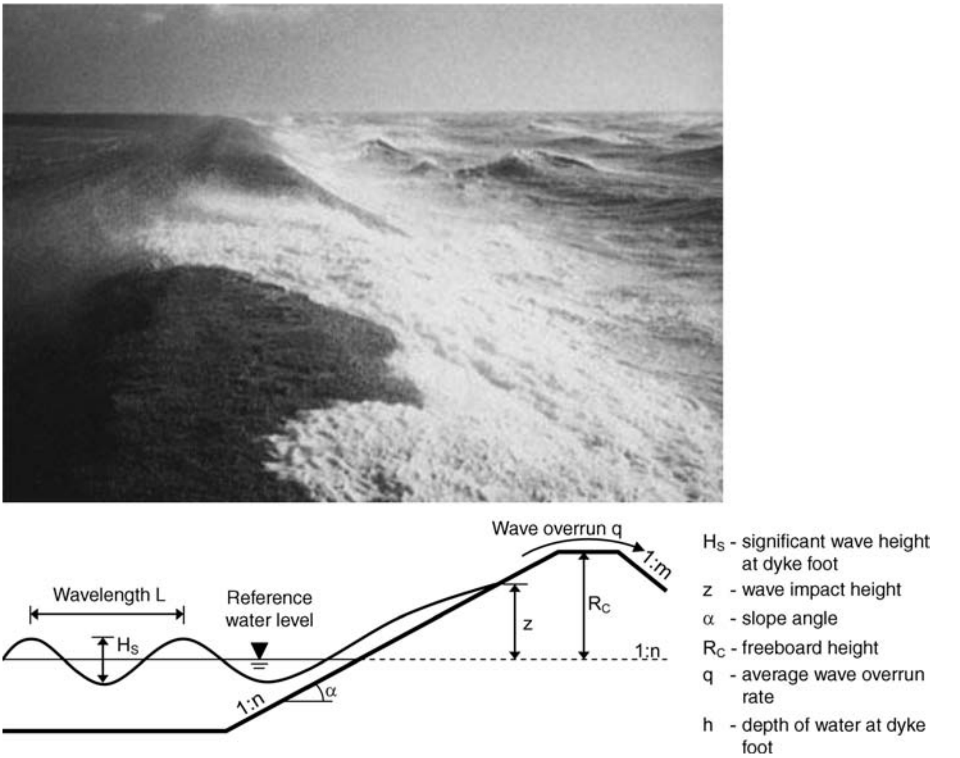

Fig. 5.6 Sea dyke as flood protection for nuclear power plants [46]

Brokdorf on the river Elbe, for example, this gives an extrapolation difference ED of the order of 100–150 cm.

With dykes, as well as the storm tide water level $SFWH_{(10^{-4})}$ the wave run-up must also be taken into account (Figure 5.6) and, having superposed these two variables, the dykes must be designed without waves breaching them or a possible breaching wave putting the stability of the dyke at risk. The wave run-up height at the dyke depends not only on the wave height and wave period, but also on the characteristics of the dyke itself, such as its slope or surface area. When calculating wave heights, it should be borne in mind that these are particularly subject to local wind speed and direction and to the topography of the foreshore.

5.3.3 Airplane crash

5.3.3.1 General notes

Airplane crash must be considered as an exceptional, extremely rare event which, unlike earthquakes or floods, is not rated as an anomaly at safety level 3, but as a beyond design system status condition at safety level 4 (cf. Section 2.5). An airplane hitting a building has dynamic effects on that building which can be defined as a load over time function. It is appropriate here to distinguish between the different dimensions of military aircraft (small compact) and commercial ones (large).

Crashing fast-flying military aircraft was included as a fundamental design event when building new nuclear power plants in Germany, particularly after military aircraft (mainly Starfighter) crashes piled up in the 1970s. In the first instance, therefore, a load over time function was developed for a Starfighter crash and used as the basis for design. Even while designing the Convoy plants and their immediate predecessors, known as pre-Convoy plants, it had been decided to use a more robust design based on a Phantom F-4 crashing at a speed of 215 m/s. The requirements involved, including the load over time function, can be found in the RSK guidelines, and became the design standard for German nuclear power plants since the Convoy and pre-Convoy models.

Unlike Germany, other countries – with a few exceptions – did not allow for the impact of a fast flying military aircraft when designing and building nuclear power plants. That was evidently because such a scenario was highly unlikely, and the additional construction costs were high.

When terrorists flew aircraft into the World Trade Center on 11 September 2001, however, ideas about using airplane crash as a basic design principle changed. Many countries, especially in Europe and the USA, now take airplane crash into account when building new nuclear power plants. It may be assumed that Europeans require new installations to be designed to withstand the impact of both military and commercial aircraft. When designing for airplane crash, it should be borne in mind that redundantly proposed building which are physically separate need not be designed expressly for aircraft impact, as the redundancy means that a aircraft impact can only destroy one of those buildings.

5.3.3.2 Load over time functions

The dynamic effects of an airplane crash give rise to load over time functions which depend on the type of aircraft involved (weight, geometry, impact area) and how fast it is travelling when it hits the building (impact velocity). The load over time function must show in each case that the building affected can withstand the loads, both locally (punch-through) and globally (stability, load bearing to foundations) and that the shock induced by the impact does not damage structural members or components inside the building.

We can derive the load over time function by using the RIERA model [47,48]. This assumes a 'soft impact', that is a rigid wall and the impacting body then deforming. This assumption can be justified by the fact that the buildings concerned are made of solid reinforced concrete with very thick walls (generally ≥ 1.50 m) and the aircraft body may be taken to be very yielding compared with the building. On a soft impact basis, the reaction force as the ordinate of the load over time function consists of two components: a bursting load component and a component as the product of the aircraft weight and the square of its velocity. The quadratic component shows how important the velocity assumption is.

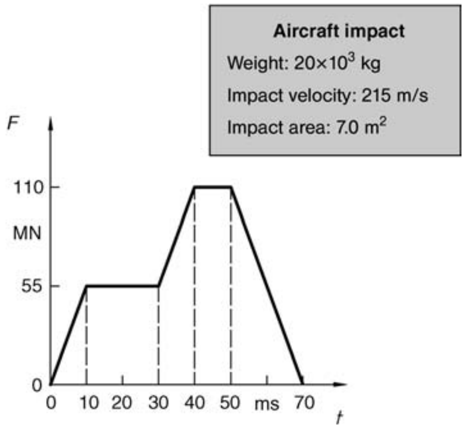

Fig. 5.7 Aircraft impact, load over time function of a military aircraft (Phantom F-4)

Figure 5.7 shows the load over time function obtained using the RIERA model for a Phantom F-4 hitting at 215 m/s as mentioned above. The tests conducted on this in Sandia confirmed this theoretical function: it matches the function specified in the RSK guidelines [5], and is often used as a design principle when building new nuclear power plants in Europe.

The RIERA model can also be used to derive load over time functions for commercial aircraft impacts. Compared with a military aircraft impact, the load over time functions obtained for larger commercial aircraft flying at 100–150 m/s give rise to much higher maximum loads and greater pulses accordingly. As a commercial aircraft would have a much larger impact area, on the other hand, the local surface area loads are much less than those of a military aircraft, so that where a military aircraft hits would be much more decisive than a commercial aircraft when conducting the punch-through proof required. It has also been found that the much larger pulse of commercial aircraft in general induces much greater induced vibrations in a building than a military aircraft.

5.3.4 Explosion pressure wave (chemical explosion)

Like an airplane crash, an explosion pressure wave is rated as an extremely rare event (safety level 4), and thus qualifies as beyond design system status. An explosion pressure wave is a chemical explosion in the form of a deflagration (pressure rising relatively quickly, building up reflected pressure). It may be caused by using explosives or if a high-energy container bursts, so that an explosion pressure wave must be accepted as a design basis when carrying hazardous cargos by rail, water or road and when storing containers with high energy content.

A chemical explosion causes pressures on the building concerned and induced vibrations in that building. The external explosive loads due to air pressure waves give an explosion pressure which can be expressed in time and place terms

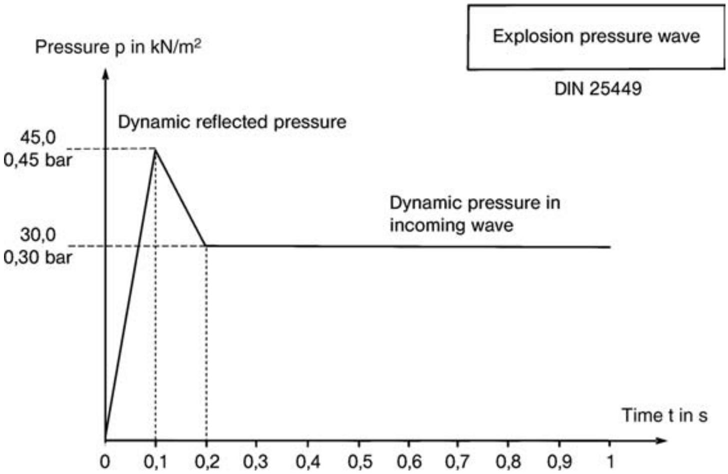

Fig. 5.8 Explosion pressure wave to BMI guidelines

as follows:

$$p = p_s + c \cdot q$$

where

p_s is the compression pressure, including reflected increase
q is the velocity pressure (dynamic pressure)
c is a coefficient of form

With box-shaped buildings (non-slender structures), the $c \cdot q$ component may be ignored; with slender structural sections, the explosion pressure can be treated as a static wind load $c \cdot q$ as defined in DIN 1055-4 [49]. For more details of using this function for explosion pressure see DIN 25 449 [15].

As a general rule, if no more precise local studies are available, possible explosion pressure waves can be established using the pressure wave in the BMI guidelines [50]. This function, as shown in Figure 5.8, is specified in the RSK guidelines for PWRs, and represents a conservative assessment of potential explosion pressure waves. This approach assumes that the pressure wave can come from any given direction and that there is a level pressure front.

6 Safety concept and design

6.1 Underlying standards

When designing structural systems for nuclear power plants in Europe, the verifications of stability and serviceability for purpose must be conducted based on Eurocode standards with the semi-probabilistic partial safety concept. Eurocode standard principles are therefore followed in the specific rules for nuclear power plants, consisting of the KTA rules [14] and DIN standards. KTA status report KTA-GS-78 [51] and the corresponding new DIN 25449 [15] are of fundamental importance here.

KTA status report KTA-GS-78 provides recommendations to be used in the partial safety concept required for designing building structures in Europe. These recommendations, which focus particularly on categories of requirements specific to nuclear power plants relate to both reinforced and pre-stressed concrete and to steelwork. The new DIN 25449, which is guided by the recommendations of KTA status report KTA-GS-78, includes specific definitions of specific extraordinary actions involved in nuclear power plants and details of proofs and design concepts for concrete, reinforced and pre-stressed concrete structural members.

Apart from KTA status report KTA-GS-78 and DIN 25449 with their fundamental design requirements, KTA 2201.3 [42] and DIN 25459 [16] should also be mentioned. KTA 2201.3 gives details on designing nuclear power plant structures to withstand earthquake effects and DIN 25459 governs designing containments, covering possible containment variants – reinforced concrete and pre-stressed containments with liners.

6.2 Partial safety concept

6.2.1 General notes

When considering the partial safety concept, the various ultimate limit states (ULS) and serviceability limit states (SLS) must be verified: that is to say, the actions E_d must not exceed the structural resistance R_d to be considered in each case. For these limit conditions, following DIN 1055-100 [52], we distinguish between a number of effects:

- independent constant actions G_k
- independent actions of pre-stressing P_k
- dominant independent variable actions Q_{k1}
- other independent variable actions Q_{ki} $(i > 1)$
- extraordinary actions A_d
- effects due to earthquakes A_{Ed}.

When designing nuclear power plants, the effects designated as external or internal actions (cf. Section 5) can be assigned to the group of ‘extraordinary actions’ or ‘earthquake actions’. These are pre-given as design values (see Section 5), so that implicitly a partial safety factor is given a value of 1.0 and the design basis earthquake to KTA 2201.1 [37] is given a weighting factor γ_1 to DIN 1055-100 and the importance factor to DIN 4149 [39] is taken as 1.00. All other effects are to be stated as characteristic values.

Design and Construction of Nuclear Power Plants. First Edition.
Rüdiger Meiswinkel, Julian Meyer, Jürgen Schnell.

To obtain the reference values for these actions, we need to examine different combinations to DIN 1055-100. We distinguish between design situations, as follows:

Ultimate limit state

– Permanent and temporary design situation

$$E_d = E\left[\gamma_G \cdot G_k \oplus \gamma_P \cdot P_k \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i>1} \left(\gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}\right)\right] \tag{6.1}$$

– Extraordinary design situation

$$E_{dA} = E\left[G_k \oplus P_k \oplus A_d \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i>1} \left(\psi_{2,i} \cdot Q_{k,i}\right)\right] \tag{6.2}$$

– Design situation due to earthquake

$$E_{dAE} = E\left[G_k \oplus P_k \oplus \gamma_1 \cdot A_{Ed} \oplus \sum_{i\geq 1} \left(\psi_{2,i} \cdot Q_{k,i}\right)\right] \tag{6.3}$$

Serviceability limit states

– Rare (characteristic) combination:

$$E_{d,rare} = E\left[G_k \oplus P_k \oplus Q_{k,1} \oplus \sum_{i>1} \left(\psi_{0,i} \cdot Q_{k,i}\right)\right] \tag{6.4}$$

– Frequent combination:

$$E_{d,frequ} = E\left[G_k \oplus P_k \oplus \psi_{1,1} \cdot Q_{k,1} \oplus \sum_{i>1} \left(\psi_{2,i} \cdot Q_{k,i}\right)\right] \tag{6.5}$$

– Quasi-permanent combination:

$$E_{d,perm} = E\left[G_k \oplus P_k \oplus \sum_{i\geq 1} \left(\psi_{2,i} \cdot Q_{k,i}\right)\right] \tag{6.6}$$

6.2.2 Partial safety factors and combination factors for actions

Partial safety factors γ of effects may be assumed to DIN 1045-1 [21]. Recommended coefficients of combinations factors ψ to KTA-GS-78 [51] and DIN 25449 [15] are stated in Table 6.1.

6.2.3 Partial safety factors for structural resistance

For verifications of the serviceability limit states, the partial safety factors for the resistance are generally to be taken as 1.00.

Table 6.1 Reference values for partial safety factors and combined coefficients

Effects		Partial Safety Factor	Combined Factor		
		γ_G, γ_Q	ψ_0	ψ_1	ψ_2
G	Dead load	1.35[a)]	—	—	—
Variable actions Q	Quasi-permanent service loads	1.50[b)]	1.0	1.0	1.0
	Variable service loads	1.50[b)]	0.9	0.8	0.8
	Crane loads	1.35	1.0	0.9[d)]	0
	Indirect actions due to settlements	1.50[c)]	1.0	1.0	1.0

[a)]1.00 at favourable effects
[b)]1.35 if effect variable can be determined very precisely
[c)]1.00 if a linear calculation is used and the rigidity in the structure can be reduced (by cracks forming or relaxation, for example) (see Section 7.2)
[d)]With category A3 requirements (cf. Section 6.3) the crane load can be ignored as a variable action, i.e. $\psi_1 = 0$

The partial safety factors to determine the structural resistance of the ultimate limit states depend on the design situation (permanent and temporary, extraordinary) of the building materials used (concrete, concrete steel, pre-stressed steel, construction steel) and the demands on the structure or structural member in question.

Safety-related structural members are subject to different requirements under these effects. Factors to be taken into account here include:

- Chances of their occurring during working life
- Repair options available
- Limiting the extent of the damage, such that the structural members remain fit for use and system components remain intact and operational.

With these aspects in mind, requirements when designing structural components of nuclear installations are divided into three requirement categories, A1, A2 and A3. These are defined irrespective of the building materials involved as shown in Table 6.1.

Requirement category A1

Those combinations of physical effects corresponding to the permanent and temporary design situations in accordance with DIN 1055-100 will be assigned to requirement category A1. The partial safety factors specified in DIN 1045-1 for the load-bearing capacity regarding permanent and temporary design situations will be assigned to these combinations.

Requirement category A2

Following the method described in DIN 1055-100, those combinations of physical effects that comprise extreme design situations, which must be assumed to occur several times during service life, are assigned to requirement category A2. It must be ensured that the building elements designed accordingly are continuously useable after occurrence of these combinations. In regard to the stability or functional safety of plant components, additional requirements may have to be specified for individual locations (e.g. limit values for deformations and crack widths).

Requirement category A3

Combinations of physical effects comprising extreme design situations with a low probability of occurrence (internal or external events, $\leq 10^{-4}$ per year) which must be assumed to occur once during service life will be assigned to requirement category A3. The forming of large cracks and permanent deformations is permitted, provided, these are not prohibited for safety-related reasons. In regard to the stability or functional safety of plant components, additional requirements may have to be specified for individual locations (e.g. limit values for deformations and crack widths) that go beyond the minimum requirements with regard to the load-bearing capacity.

Partial safety factors of structural strength for structural components of concrete, reinforced concrete and pre-stressed concrete in requirement categories A1, A2 and A3 to KTA-GS-78 [51] and DIN 25449 [15] are shown in Table 6.2. Table 6.3

Table 6.2 Partial safety factors for structural members of concrete, reinforced and pre-stressed concrete (ULS)

Reinforced and Pre-Stressed Concrete Structures		Requirement Category		
		A1	A2	A3
Partial safety factors	Concrete γ_c	1.50	1.30	1.00
	Concrete steel/ pre-stressing steel γ_s	1.15	1.00	1.00
Non-linear procedures	System resistance γ_R	1.30	1.10	1.00
	Concrete compression strength f_{cR} a)	$0.85 \cdot \alpha \cdot f_{ck}$	$0.85 \cdot \alpha \cdot f_{ck}$	$1.00 \cdot \alpha \cdot f_{ck}$
	Yield strength concrete steel f_{yR} b)	$1.1 \cdot f_{yk}$	$1.1 \cdot f_{y,k}$	$1.0 \cdot f_{y,k}$
	0.1% proof stress pre-stressing steel $f_{p01,R}$ b)	$1.1 \cdot f_{pk}$	$1.1 \cdot f_{pk}$	$1.0 \cdot f_{pk}$

a) Reduction value α in DIN 1045-1:2001-07, 8.1

b) Tensile strength concrete steel: $f_{tr} = 1.08 \cdot f_{pk}$

Table 6.3 Partial safety factors for steel members (ULS)

Steelwork: Req. Category	A1/A2/A3	Notes
DIN 18800-1 γ_M	1.0/1.1[a)]	Cf. DIN 18800-1 Section 7.3.1
DIN EN 1993-1-1		
γ_{M0}	1.0	
γ_{M1}	1.0/1.1[a)]	Major deformations due to yielding are acceptable for capacities of action effects that depend on the yield stress (for stability failures etc.)
γ_{M2}	1.25	For capacities of action effects which depend on tensile strength (net cross-section failures under tension or bolt or weld failures etc.)

[a)] Needs to be established on a case by case basis

contains the partial safety factors for structural members of steel as recommended in KTA-GS-78.

6.3 Design instructions for concrete, reinforced and pre-stressed concrete structures

6.3.1 Strength parameters

In principle, the strength parameters for concrete, reinforced and pre-stressed concrete, including limits of ultimate limit strains to be observed, must be taken as per DIN 1045-1 [54]. Further details specific to nuclear power plants can be found in DIN 25449 [15].

To determine the design values, the characteristic strength parameters must be divided by the partial safety factor γ_M in each case. Concrete compression strength f_c must be divided by γ_c and the strength of concrete steel and pre-stressing steel (concrete steel: yield stress f_{yk} and tensile strength f_{tk}, pre-stressing steel: yield point $f_{p0.1k}$ and tensile strength f_{pk}) must be divided by γ_S, using the partial safety factors as shown in Table 6.2 for the proofs in ULS depending on the category of requirements concerned.

For concrete compression strength, the influence of long-term effects and the influences on design-relevant concrete characteristics, such as the effects of load duration, curing and loading speed, must be taken into account. In certain justified cases, variations from the design-relevant characteristics of concrete as a construction material from the characteristics on which DIN 1045-1 is based may be used (deviations from design values). This applies in particular to the concrete getting stronger as it cures in long-standing reinforced concrete structures and the increase in strength of concrete stressed in multiple axes or high expansion rates and the influence on the ultimate limit strains of concrete.

6.3.2 Shear force

Verifications of shear resistance of reinforced or pre-stressed concrete members must be conducted to DIN 25449. The verifications are based on the method in accordance with DIN 1045-1, having regard to the different requirement categories, A1, A2 and A3. Shear force reinforcement will be necessary if, in a cross-section, the design value of the acting shear force V_{Ed} is greater than the design value of the shear force $V_{Rd,ct}$ a structural member can withstand without shear force reinforcement, i.e. if:

$$V_{Ed} > V_{Rd,ct} \tag{6.7}$$

The design value $V_{Rd,ct}$ considers the different requirement categories A1, A2 and A3 by a factor c_d, which is to be taken as 1.0 for A1, 1.15 for A2 and 1.50 for A3. This makes the reference value:

$$V_{Rd,ct} = \left[c_d \cdot 0,10 \cdot \kappa \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} - 0,12 \cdot \sigma_{cd}\right] \cdot b_w \cdot d \tag{6.8}$$

Where

$$\kappa = 1 + \sqrt{\frac{200}{d}} \leq 2,0 \tag{6.9}$$

Where

- b_w is the smallest cross-sectional width within the tension area of the cross-section [mm]
- c_d is the prefactor reflecting the requirement category
- d is the static effective depth of the flexural reinforcement in the cross-section [mm]
- f_{ck} is the characteristic value of the concrete compression strength [N/mm²]
- ρ_l is the longitudinal reinforcement ratio in the tension area
- σ_{cd} is the design value of the axial concrete stress at the height of the centre of gravity of the cross-section where $\sigma_{cd} = \frac{N_{Ed}}{A_c} < f_{ctk;0,05}$

Shear force design of structural members that are subject to bending stresses must be carried out based on a truss system in which the angle of the strut of the truss must be limited and the shear force reinforcement must be proven as $V_{Ed} \leq V_{Rd,sy}$ and $V_{Ed} \leq V_{Rd,max}$. The maximum angle of the strut and the design value of the shear force which can be absorbed, limited by the strength of the shear force reinforcement $V_{Rd,sy}$ and the design value of the maximum shear force $V_{RD,max}$ that can be absorbed to DIN 1045-1 must be complied with in accordance with requirement categories A1, A2 and A3.

6.3.3 Punching shear

As with the verifications for shear resistance, punching shear verifications for reinforced and pre-stressed concrete structures in nuclear installations must be conducted to DIN 25449 [15]. Under these verifications, which are also based on DIN 1045-1 [54], punching shear reinforcement is required if, along the critical circular section to DIN 1045-1, the shear force v_{Ed} to be absorbed per unit of length is greater than the shear resistance v_{Rd}, i.e. if:

$$v_{Ed} > v_{Rd,ct} \tag{6.10}$$

Shear resistance $v_{Rd,ct}$ is obtained from DIN 1045-1. Like $V_{Rd,ct}$, $v_{Rd,ct}$ reflects the different requirement categories A1, A2 and A3 by a factor (cf. Section 6.3.2: 1.0 for A1, 1.15 for A2 and 1.50 for A3).

To find the punching shear reinforcement required, we distinguish between

– structural members subject to **indirect effects of action**, as covered in DIN1045-1 (e.g. supports in slabs or foundations), and
– structural members subject to **direct** effects of actions, as they occur in nuclear engineering construction as structural members subject to extraordinary actions in requirement categories A2 or A3, such as airplane crash or jet forces.

The reinforcement of structural members subject to indirect effects of actions must be obtained to DIN 1045-1. Should no more precise calculation and design procedure be used, the reinforcement required for structural members subject to direct effects of actions may be calculated in accordance with DIN 25449, which is based on DIN 1045-1 and additional experimental studies. The upper bound $v_{Rd,max}$ to DIN 1045-1, which is intended to prevent the concrete cover palling at column faces, is irrelevant here, although the concrete strut resistance must be verified to ensure that the stirrup reinforcement is activated, i.e.

$$V_{Ed} \leq V_{Rd,max} = 0.25 \cdot f_{cd} \cdot u_{load} \cdot d \tag{6.11}$$

Where

f_{cd} is the design value of concrete compression strength [N/mm^2]; $f_{cd} = f_{ck}/\gamma_c$ (γ_c in Table 6.2)
u_{load} is the circumference of the load area A_{load} (equivalent circle with radius R_{load}; see Figure 6.1)
d is the static effective depth of the flexural reinforcement of the side facing away from the load in the cross-section considered

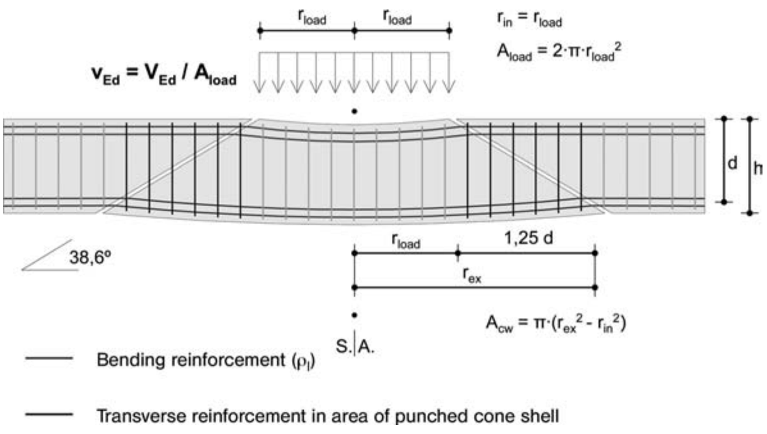

Fig. 6.1 Punching shear cone (area of direct effects of actions)

In calculating the punching shear reinforcement, the decisive shear force V_{Ed} is taken as the maximum load resulting on the load area A_{load}. The verification may assume as the failure figure a punching shear cone with an effective surface area $A_{cw} = \pi \times (r_{ex}^2 - r_{in}^2)$ and angle of inclination of the punching shear cone $ß_r$ (generally cot $ß_r = 1.25$) (Figure 6.1).

For the reinforcement, it must be shown that the relationship

$$V_{Ed} \leq V_{Rd,sy} \tag{6.12}$$

is satisfied.

The design value for the required shear force reinforcement $V_{Rd,sy}$ consists of a contribution of concrete load-bearing $V_{Rd,c}$ (with the contribution of longitudinal reinforcement) and a contribution of punching shear reinforcement, i.e.

$$\begin{aligned} V_{Rd,sy} &= V_{Rd,c} + \kappa_s \cdot A_{sw} \cdot f_{td} \\ &= [c_d \cdot 0.17 \cdot \kappa \cdot (100 \cdot \rho_l)^{1/4} \cdot f_{ck}^{1/3}] \cdot d \cdot u_{ex} + a_{sw} \cdot \kappa_s \cdot A_{cw} \cdot f_{td} \end{aligned} \tag{6.13}$$

Where

A_{sw} is the effective cross-sectional area of the vertical punching shear reinforcement in area A_{cw}
a_{sw} is the effective cross-sectional area of the punching shear reinforcement $a_{sw} = A_{sw}/A_{cw}$
A_{cw} is the effective projection area of the punching shear cone; $A_{cw} = \pi \cdot (r_{ex}^2 - r_{in}^2)$
f_{ck} is the characteristic concrete compression strength $f_{ck} \leq 35$ [N/mm^2]
u_{ex} is the circumference for r_{ex} ($u_{ex} = 2 \cdot \pi \cdot r_{ex}$)
f_{td} is the design value of the tensile strength of the concrete steel; $f_{td} = f_{tk}/\gamma_s$ (γ_s in Table 6.2)
k Coefficient: $\kappa = 1 + \sqrt{\frac{200}{d}} \leq 2,0$
k_s Coefficient to allow for the influence of structural member depth on the effectiveness of the reinforcement, where

$$\kappa_s = 0,65 + \frac{d - 400}{400} \begin{cases} \geq 0,65 \\ \leq 1,00 \end{cases}$$

(average useful height in mm)
c_d Prefactor reflecting requirement category
ρ_l Degree of longitudinal reinforcement in tension area

6.4 Design instructions for steel components

The current steelwork standards (DIN 18800-1 [55] and/or DIN EN 1993-1-1 [56]) with the new partial safety factor have only been reflected in KTA rules [14] for steel structures to a limited extent to date: so KTA status report KTA-GS-78 [51] advises relating steelwork load cases H, HZ, HS1, HS2 and HS3, and requirement categories A1, A2 and A3 (cf. Table 6.4).

Table 6.4 Assigning steelwork load cases to requirement categories

Steelwork Load Cases	Design Criteria	Requirement Category
H, HZ	Fully fit for use, can be stressed repeatedly and always reused	A1
HS1		A1, A2[a)]
HS2	Meets stability requirements, maintains necessary functions (e.g. bearing play), limits deformation, generally reusable	A2, A3[a)]
HS3	Major plastic deformation permitted, reuse not proposed	A3

[a)]To be classified on a case-by-case basis

How steelwork structures are designed depends on which KTA rule is to be used for the structure in question: so verifications may be required either by the global safety concept or the partial safety concept.

Fundamentally, the design procedures in DIN 18800-1 may be used (Table 6.5). Stability verifications must also be considered here, such that either with beam structures the limits of slenderness must be observed in all cross-sections, or with plates and shell structures buckling safety must be verified to DIN 18800-3 [57] or DIN 18800-4 [58].

The plastic–plastic design procedure, as shown in Table 6.5, reflects the plastic hinge analysis as a simplified method. More precise design procedures, such as using non-linear calculation methods reflecting realistic steel material laws, may also be used. When using plastic cross-section or system reserves, the design criteria in Table 6.5 must be observed.

6.5 Particularities of containment design

6.5.1 Requirements of containments

The containment (safety container or enclosure) in the reactor building of a nuclear power plant is the essential structural barrier involved in containing radioactive

Table 6.5 Design procedures to DIN 18800-1

Design Procedure	Determining	
	Internal forces due to actions	Capacity of action effects
Elastic–elastic	Elasticity theory	Plasticity theory
Elastic–plastic	Elasticity theory	Plasticity theory
Plastic–plastic	Plastic hinge analysis	Plasticity theory

substances safely (cf. Sections 2.5 and 4.2). The verifications required for this barrier are:

– bearing capacity
– serviceability in the sense of functional ability
– integrity (gas-tightness).

Bearing capacity and serviceability for use can be combined as a single overall concept, structural integrity. The structural integrity of a containment is tested once it has been made, while gas-tightness is tested regularly every three to five years.

The verifications must take account of the actions when operated as intended (normal and abnormal operation) and those of incidents (cf. Section 2.5). Containment design is governed in particular by the possibility of a loss of coolant accident, with its high pressure of the order of 0.5 MPa accompanied by temperatures of approx. 150 °C.

6.5.2 Reactor containment of steel

Except for two blocks at Gundremmingen nuclear power plant, all containments in Germany are made of steel. The containments of the more recent German PWR plants (Convoy and pre-Convoy plants) consist of a steel sphere 56 m in diameter with walls 30–40 mm thick. These dimensions are based on a design pressure in the range 4–5 bar overpressure at a design temperature of approx. 150 °C. The guideline values for the maximum permitted leakage rate are 0.25–0.50% per day.

Steel containments are designed in accordance with KTA rules. KTA 3401 [59–62] covers materials, design conditions, design, production and testing. The material that KTA 3401.1 [59] requires is 15 MnNi 63 steel, whose mechanical characteristics, with a yield point of 330–370 N/mm^2 and a tensile strength of 490–630 N/mm^2 are comparable with those of construction steel S355.

Design is governed by KTA 3401.2 [60], and is based on permitted stresses, departing from the partial safety concept. Permitted stresses are defined for four stress levels and the various stress categories, allowing for how steel characteristics change at high temperatures. A loss of coolant accident as the dominant verification demand of the containment is put in the operating stress level and hence not regarded as a failure case.

Stability studies are also required to cover the possibility of a partial vacuum arising in the containment. The pressure tests here assume a partial vacuum of 45 mbar and a partial vacuum of 5–30 mbar in normal operation.

6.5.3 Pre-stressed concrete containments with steel liners

Not all structural sections are pre-stressed, even in pre-stressed concrete containments. Pressurised water reactors have cylindrical containments with cupolas on top. The cylinder walls and cupola are pre-stressed, the base slab is not. Boiling water reactors, on the other hand, have flat cylindrical covers; only the cylinder walls being pre-stressed. Pre-stressing increases the containment's serviceability, i.e. it keeps deformation and cracking low; but the cross-section of the concrete cannot be overpressed completely, to ensure integrity in problem areas such as transitions between structural sections or around openings, so today's pre-stressed concrete containments are fitted with steel liners to guarantee their integrity. One example of a pre-stressed concrete containment with a steel liner is EPR containment.

The steel liner is anchored to the concrete structure via headed studs and/or steel profiles to give a composite steel-concrete structure. To avoid affecting the pre-stressing, the steel liner is made of thin plate, $t = 6\,mm$, for example. When verifying the structural strength of a containment, the steel liner is only taken into account if its effects are adverse.

Pre-stressing the concrete structure induces a compressive strain in the steel liner. Pre-stressing also induces a time- and stress-based concrete creep which devolves the stresses involved and puts an additional compressive strain on the steel liner. The liner also expands under the influence of dissipation of the heat of hydration and as the concrete shrinks and also under the effects of operating conditions and in incident cases. The verification of liner integrity is obtained by limiting the liner strains and the action effects of the connectors.

Pre-stressed concrete containments with steel liners can be designed using DIN 25459 [16].

6.5.4 Reinforced concrete containments with steel liners

With reinforced concrete containments with steel liners, the reinforced concrete structure ensures the structural integrity, and the steel liner the gas-tightness. A reinforced concrete containment is about as strong as a pre-stressed concrete one if the stressing steel is replaced with the concrete steel in proportion to their respective yield stresses. With the massive concrete cross-sections usually found in building nuclear power plants, this can be done without further ado.

The steel liner is anchored to the concrete structure via headed studs and/or steel sections so that a steel composite construction exists once it is completed with this concept; but the expansion of the steel liner and stresses on the laminate are less critical, as there is no pre-stressing. The reinforced concrete structure also enables a thicker steel liner to be used. Increasing the inherent rigidity of the steel liner makes it easier to install, and improves its strength and hence its integrity. As with a pre-stressed concrete containment, the design can be based on DIN 25459 [16]. The steel laminate effect must be considered in particular here, especially if using relatively thick steel liners, as with the non-pre-stressed containment of the KERENA BWR reactor model (steel liner $t = 10\,mm$).

7 Fastening systems

7.1 Fastening types

In nuclear power plants and other nuclear installations, a basic distinction is made between safety-related and non-safety-related fastenings. Safety-related fastenings are those used to attach safety-related structural and system components. In nuclear power plants, safety-related structural and system components are those required to meet safety goals (controlling reactivity, cooling fuel elements and containing the radioactive materials) and to limit radiation exposure as part of incident management.

Important safety-related fastenings performed using anchors also include those to which structural and system components are attached which are not safety-related in themselves, but which could cause unacceptable effects on safety-related structural and system components in case of their failure.

Today, that means that the fastenings used in nuclear power plants and other nuclear installations are overwhelmingly safety-related ones. These fastenings can be divided into cast-in fastenings and subsequently mounted (or post-mounted) fastenings.

7.1.1 Cast-in fastenings

The best-known and most used cast-in fastenings are anchor plates of steel to which system components are welded. Where fastening points are already known at the construction stage, the anchor plates used are mainly headed stud anchorings which are built in along with the reinforcement, before pouring concrete (Figure 7.1).

Headed stud anchorings have a general approval issued by the German Institute for Construction Technology (in German; Deutsches Institut für Bautechnik – DIBt), for general building construction, but not for accidental actions such as earthquakes.

Fig. 7.1 Anchor plate with headed stud anchorings (source: www.halfen.de)

Cast-in fastenings must be planned precisely before construction starts, defining the load to be absorbed and where the fastening points are to be positioned.

Design and Construction of Nuclear Power Plants. First Edition.
Rüdiger Meiswinkel, Julian Meyer, Jürgen Schnell.

7.1.2 Post-mounted fastenings

Post-mounted fastenings are used to transmit loads wherever detail design has not provided for any cast-in fastening points before construction has started or where new fastenings become necessary in the course of refitting work. However, there are also design challenges here: although loads, dimensions and positions of system components are known precisely, high density of reinforcement, poor access and closeness to other fastening points or structural section edges may make planning, designing and installing fastening points extremely complex. The options available for post-mounted fastenings are basically as follows:

- run-through anchors
- spreading anchors
- undercut anchors
- composite anchors
- cast-in anchors.

For safety-related fastenings, only fastenings with sufficient mechanical grip should be used [63]. With metallic anchors, this can be achieved very well by using form-locking undercut anchors, so that a general authoritative approval only exists for undercut anchors, although work is underway to achieve an approval for composite anchors in the near future.

7.1.3 Load-bearing capacity

The load-bearing capacity of fastenings in concrete sufficiently far from edges of structural components can be divided into modes of failure as follows.

- tension failure:
  - steel failure
  - extrusion
  - concrete cone failure
  - gaps
- transverse tension failure
  - steel failure
  - concrete pry-out
  - concrete edge fracture.

The design principles can be found in the following bodies of rules:

- ETAG 001(Guideline for European Technical Approval) [64,65]
- CEN/TS (Comité Européen de Normalisation/Spécification Technique) [66]
- Guidelines on assessing anchors fastenings to be used in nuclear power plants and other nuclear installations DIBt [63],
- Guidelines for anchor fastenings in nuclear power plants and other nuclear installations, DIBt, June 2010 [67].

7.2 Fastening with headed studs

7.2.1 History

The use of headed studs in fastening systems dates back to the early 1970s. The first headed studs used in building nuclear power plants were PECO concrete anchors. In

1971, Peco Bolzenschweißtechnik GmbH was acquired by Nelson StudWelding Co. and had its name changed to Nelson Bolzenschweiss-Technik GmbH. After a long time as part of the TRW-group, Nelson Bolzenschweiss-Technik moved to the Fabri-Steel Group in Michigan, USA, in 2000, and has belonged to Doncasters Group Ltd, UK, since 2009. However, the company label, Nelson, remains unchanged to date.

The headed studs found in older nuclear power plants under the names 'Peco', 'Nelson' and 'TRW-Nelson' are all the same product. Following extensive tests and expert opinions by Professors *Roik* and *Bode*, the DIBt granted the first general approval for headed studs (anchoring steel plates using welded-on Nelson headed studs, approval no. Z-21.5-82) in 1983.

Not long after that, the DIBt issued further headed stud licences for the Köster & Co. of Ennepetal (Z-21.5-280) and Riss AG of Dällikon, Switzerland (Z-21.5-296).

Headed stud anchors were still designed based on permissible tension and lateral loads at the time. Existing edge and plane factors were allowed for via reduction factors (kappa method). When the licence was amended in 1995, an extended calculation method (CC method) was introduced and the semi-probabilistic safety concept was used instead.

The DIBt issued the first European Technical Approval for fastenings in November 2003. The two licence notices ETA-03/0039 (KÖCO headed studs) and ETA-03/0041 (Nelson headed studs) were extended in 2008, and are the current state-of-the-art rules.

The particular safety requirements involved that mean using headed studs in nuclear power plants must be approved by the licensing authorities in each case.

7.2.2 Usage and characteristics

Anchor plates with headed studs are found in power plants, in both safety-related and non-safety-related applications. They are used for example as supporting and load transmission points for platforms as well as for mounting devices for pipe and cable racks, risers etc. Figure 7.2 shows a schematic anchor plate with headed studs.

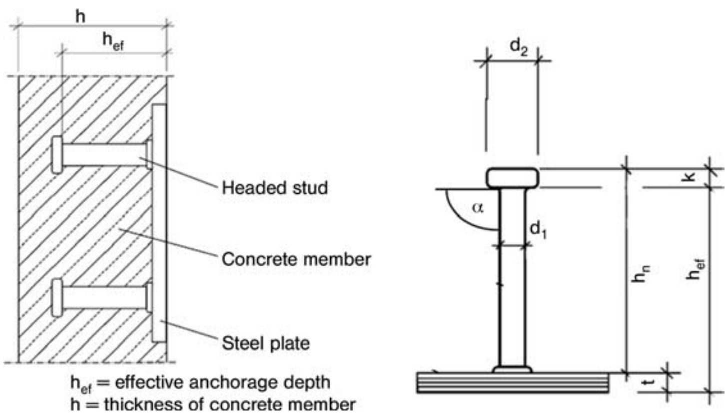

Fig. 7.2 Fastening an anchor plate, schematic [68]

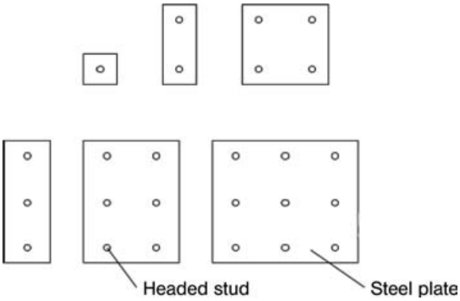

Fig. 7.3 Possible arrangements of headed stud anchors [68]

Anchor plates vary in size from small two-bolt units to groups with nine headed studs; there are even larger sizes, though, such as 4×4 or 5×5 headed studs up to 2.0 m long anchor strips with up to 25 headed studs. Larger anchor plates often need additional reinforcement to back-anchoring of tensile forces.

Large anchor plates can be used for a great variety of connection arrangements, but their load transmission is always limited to a local area.

In former times headed stud diameters ranged from 3/8″ (9.52 mm) to 7/8″ (22.22 mm) with a ratio of head to shaft diameter of between 1.60 and 2.0. Headed studs 175 mm

Fig. 7.4 Typical anchor plates fixed in a power plant [71]

long were preferred: if a greater anchoring length was required, or if it was wished to anchor the headed studs within the background bending compression area, two or even three headed studs would be welded together. This method can still be used today, but it should be noted that, for the extended anchor length to be effective, cushion rings must be fitted at all bolt heads except the final one welded on.

Table 7.1 Headed stud types to ETA-03/0041 [68]

Headed stud type	Shaft ∅ d_1-0.4 mm	Head ∅ d_2 mm	Nominal length min. h_n mm	Nominal length max h_n mm	Head height k mm
10	10	19	50	200	7.1
13	13	25	50	400	8
16	16	32	50	525	8
19	19	32	75	525	10
22	22	35	75	525	10
25[a)]	25	40	75	525	12

[a)] Headed stud type only available in material S235J2

Headed studs were then made in accordance with DIN 17100 of structural steel St37-3K with a minimum tensile strength of 450 N/mm^2. Stainless steel was used in areas where increased corrosion protection was required for plates (1.4571, 1.4401) and headed studs (1.4301, 1.4303). At the time when German nuclear power plants were built, stainless steel bolts were only available up to 16 mm diameter.

For new construction projects, headed studs are available in any length from 50 mm to 525 mm today. Beyond the standard range, special lengths are even available for special applications up to 750 mm, including the welding tools required. The characteristics of the material properties remain unchanged. Under the amended standards, the material designation for the unalloyed bolt steel is S235J2 + C450 according to [69]. The material code for alloy steel is unchanged [70].

Bolt diameters were changed nominally to the metric system, with a new size being added of 25 mm. Stainless bolt heads are currently available in sizes from 10 to 22 mm, and there are plans to add 25 mm diameter bolts to the stainless range also (Table 7.1).

Headed studs are now designated SD for short, and are governed by DIN EN ISO 13918 (formerly DIN 32500-3:1979). As for bolt materials, there is a cross-reference to standard ISO/TR 15608. Under this, material properties are subdivided into groups SD1, SD2 and SD3. The bolt material prescribed under ETA approval is equivalent to group SD1, with a minimum tensile strength of 450 N/mm^2. Group SD2 covers materials with a reduced tensile strength of 400 N/mm^2 and a yield point of 235 N/mm^2. The approvals do not cover group SD2 materials for applications in anchor plates.

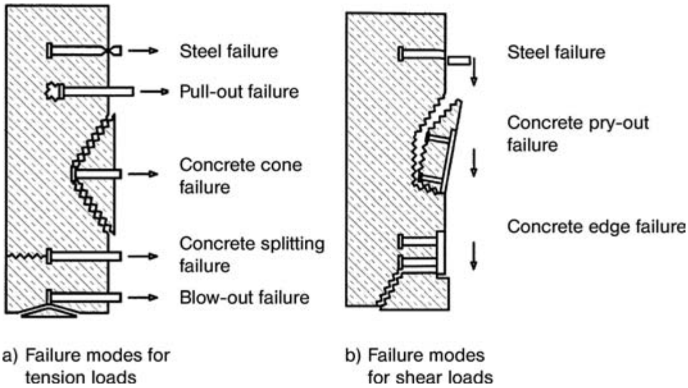

Fig. 7.5 Potential failure modes of headed studs according to [72]

Material group SD3 covers stainless steel materials (1.4301, 1.4303) with a yield point of not less than 500 N/mm^2.

For all three of these groups, the carbon content is limited to 0.20% and they must contain at least 0.02% of relaxing elements.

7.2.3 Load-bearing behaviour of headed studs

Headed stud fastenings are relatively rigid anchor constructions. The welding of headed studs to steel plates and then concreting those steel plates into reinforced concrete components gives a direct interlocking between steel and concrete. Due to forced spreading, the anchor area will not be affected by drilling or splitting forces. Headed studs show a ductile load-bearing behaviour and good deformability at the ultimate limit state.

The structural strength of headed studs depends on many factors, primarily the strength characteristics of the steel and the compressive strength class of concrete. Geometric factors such as diameter and length of the headed studs and distances from free component edges or adjacent anchorings exhibit a major influence, too.

Potential failure modes of anchorings with welded on headed studs as a function of the type of stress are shown in Figure 7.5.

7.2.4 Standards and approvals

The rules to be observed when using anchor plates with headed studs are as follows:

- European Technical Approval for steel plates with cast-in headed studs for the manufacturer in question, e.g. ETA 03/0041 for Nelson headed studs [68]
- ETAG 001, Guidelines for European Technical Approval of metal anchors for use in concrete, Annexe C, Design methods for anchorages, February 2008 [65],
- DIN EN ISO 13918:2008, Welding – Studs and ceramic ferrules for arc stud welding [73]
- DIN EN 14555:2006, Welding – Arc stud welding of metallic materials [74]
- DIN EN 10025-1:2005, Hot-rolled products of structural steels – Part 1: General technical delivery conditions [69]

– DIN EN 10088-1:2005, Stainless steels – Part 1: List of stainless steels [70]
– Deutsches Institut für Bautechnik, using anchors in nuclear power plants and nuclear installations, guidelines for assessing anchor fastenings when granting approvals in individual cases under German Federal States Building Regulations 1998:09 [63]
– Deutsches Institut für Bautechnik, Guidelines for anchor fastenings in nuclear power plants and other nuclear installations, June 2010 [67]
– DIN SPEC 1021-4-2:2009, Designing anchoring of fastenings for use in concrete – Part 4.2: Headed studs, German version CEN/TS1992-4-2:2009 [75].

7.2.5 Planning and design

7.2.5.1 Basics

During the construction period of the first nuclear power plants no general authoritative approvals for headed studs had existed. Anchorings were designed and built based on consent orders from the building authorities concerned. These orders and their associated expert opinions governed the permissible combinations of stresses for tension and shear loads for headed studs and covered any boundary or group influences individually.

When designing anchor plates, a distinction is made between safety-related and non-safety-related units.

The design and construction of fastenings for non-safety-related components is based today on the general technical approval (ETA) for the significant combination of characteristic loads concerned.

Beyond the provisions of the General Technical Approvals, safety-related components are subject to additional requirements in terms of the structural load-bearing and deformation behaviour of the anchorings.

As well as for the effects of actions from characteristic loads, safety-related anchorings with headed studs must also be designed to withstand the effects of actions from accidental external events such as earthquakes or for internal anomalies. Designing anchorings for special load cases also includes the effects of impulsive actions and the occurrence of cracking in reinforced concrete structure with wide cracks.

The design of anchor plates within headed studs for attaching safety-related components is not covered by General Technical Approvals; nor has any such General Technical Approval for accidental actions (K-approval) yet been applied for at the Deutsches Institut für Bautechnik. That means using anchor plates for safety-related components is still subject to approval by the supreme building authority in each case. This involves verifying structural strength and serviceability of anchor plates in accordance with DIBt Guidelines [67], the successor to [63]. Finally, the suitability of the anchoring for the specific intended purpose in nuclear power plants must then be assessed by an expert opinion.

7.2.5.2 Verification of load-bearing capacity and serviceability

It is a general rule for all structural fastenings that they must be designed by engineers. Deciding how to design them at the execution stage is not enough. So anchor plates with headed studs must be designed in an engineering-like manner also. This includes providing analytical verification that they have sufficient structural strength and serviceability and producing detailed drawings concerned, such as anchoring and formwork drawings.

Within the scope of general engineering construction and for non-safety-related anchorings, verification of the load-bearing capacity is based on the General Technical Approvals. Such approvals contain all the details required of specific product strengths, permitted edge and centreline distances and instructions and cross-references to the calculation methods to be used.

According to the current status of the General Technical Approvals, anchor plates with headed studs are designed in the same way as anchors based on Annexe C to European Guidelines ETAG 001 [65]. Design includes verification of headed studs against steel failure and verification of the anchor base against concrete failure. In terms of concrete failure, there are a number of failure modes to be considered, depending on the effects of actions involved, such as concrete cone failure, concrete edge fracturing and pry-out failure. The individual failure modes and calculation procedures are described in detail in [76].

In addition to the provisions of ETAG 001, General Technical Approvals for anchor plates with headed studs include additional reinforcement for anchoring the headed studs within the background bending compression area. Particularly where anchorings are close to edges, in new buildings, specifying such additional reinforcement can avoid broken edges and improve structural strength. Further studies on the effectiveness of back-tying reinforcement are described in [77].

During the construction period of the first nuclear power plants there were not yet any rules on using back-tying reinforcement for anchorings with headed studs. For post-calculating anchor plates as part of repairs and retrofits, more recent rules in DIN SPEC 1021-4-2:2009 [75] govern the co-function of existing structural component reinforcement. The existing reinforcement must meet certain design principles such as a sufficient anchoring length inside and outside the fracture cone and should be in the influence area up to 0.75 h_{ef} by the headed studs. If these conditions for use are satisfied, anchorings close to edges in existing structures can often be shown to have a greater structural strength. The more extensive rules in DIN SPEC 1021-4-2:2009 are expected to be incorporated in individual approvals for headed studs by the end of 2010.

General Technical Approvals do not cover the verification of the structural strength of anchorings for safety-related components; this must therefore be approved on a case-by-case basis (see section 7.2.5.1).

The notice of consent given by the authorities concerned and the associated expert opinion lay down the calculation methods to be used and the rules to be followed. In exceptional cases, they also include details of the materials to be used, the number and size of the studs to be used for the anchor groups concerned and additional provisions on partial safety factors, if required.

One essential feature here is the data on the structural strength of headed studs in cracked reinforced concrete structural components under accidental actions. This data is based on additional tests and assessments in line with DIBt Guidelines [63], and provides the design basis for fastenings in requirement category A3 to DIN 25449:2008 [15].

The notice of approval also contains additional general rules on accounting for existing reinforcement.

The serviceability of anchor plates with headed studs is less important with non-safety-related fastenings. The loads to be induced are generally relatively small, and the components to be attached are not sensitive to the low levels of deformation to be expected. Reference values for the deformations to be expected in the headed studs are included in the General Technical Approvals.

With safety-related fastenings, the deformation and the sensitivity to deformation of the components to be attached facing accidental actions must be matched to one another. The resilience of the fastenings must be taken into account in calculations when dimensioning components which are vulnerable to deformations, such as pipes.

Reference values for the deformations to be expected in the headed studs under accidental actions are included in the notices of consent and the associated opinions.

7.2.6 Quality assurance, material quality

General Technical Approvals state that headed studs for anchor plates must only be made of unalloyed steels in materials group SD1 and stainless steels in group SD3 to DIN EN ISO 13918 [73].

For quality assurance purposes, and to certify compliance, materials and production are inspected continuously in the workshop; the production is also monitored by an independent certification body. General Technical Approvals require headed studs to be marked with the appropriate works code on their heads, so that they can be identified easily on site. They must also be stamped with the material used, if using stainless steel. Packs must be CE-marked, stating the products approved.

Steel fastening plates are generally made of non-alloyed steel of strength class S235JR to DIN EN 10025 [69]. Unless improved characteristics are required in the direction of thickness, material quality is certified by a works certificate 2.2 to DIN EN 10204 [78], which must show the as-delivered condition to DIN EN 10025 and the melt analysis and tension test results as a minimum requirement. Additional notched bar impact bending tests must be conducted when using S235J2.

Where plates more than 30 mm thick have structural components welded on, the welding seams of which are subjected to tension, a welded-on bending test must be conducted to SEP 1390 [79] and proven by an acceptance test certificate 3.1B.

Ultrasound testing is not part of the minimum requirements, but to avoid lamination, ultrasound testing is recommended for steel plates 15 mm and over thick on a 200×200 mm matrix, even if no quality certificate as per Z-quality is required.

Ultrasound-tested steel plates must be used where mainly live loads apply or where certificates of quality are required under DASt Guideline 014 [80].

Where improved characteristics are required for plates in the direction of thickness, material qualities are proven by acceptance test certificates 3.1B. The certificate of Z-quality required must be testified, stating the reduction of area, by appropriate tension tests in the direction of thickness to DIN EN 10002 [81]. The requirements as laid down in KTA 3205.2 [82], Table 7-1 must be observed.

For plates over 15 mm thick, under tension and bending tension stresses, ultrasound testing must be carried out according to DIN EN 10160 [83] on a 100×100 mm matrix.

Material tests also include notched bar impact bending tests to DIN 10045-1 [54].

Anchor plates of S355J2 are subject to basically the same requirements. Materials must have acceptance certificates 3.1B to DIN EN 10204, stating the condition as supplied to DIN EN 10025, and the following test results at as a minimum:

- melt analysis
- tension test
- notched bar impact bending test
- weld-bead bending test.

Where improved properties in the direction of thickness are required, compliance with the carbon equivalent ($CEV < 0.45\%$) must be proven.

Ultrasound testing on a 100×100 mm matrix and appropriate tension tests in the direction of thickness are also required in order to verify the certificate of Z-quality.

Anchor plates of non-stainless materials are made of alloyed steel 1.4571 to DIN EN 10088 [70]. The testing required is governed by DIN EN 10088, and must be documented by a certificate of acceptance 3.1 B, including the heat-treated condition.

7.2.7 Production and installation

7.2.7.1 Manufacturing of anchor plates with headed studs

Headed studs are welded onto steel plates by stud welding with arc stud welding to DIN EN ISO 14555 [84] using barrier gas or ceramic ferrules.

The welding contractors concerned must hold appropriate welding certificates to DIN 18800-7 [85] extended for bolt welding to DIN EN 14555.

Ensuring that welded joints meet quality requirements is carried out in accordance with the provisions of DIN EN ISO 14555 in conjunction with DIN EN 3834 [86].

For anchor plates which are stressed in the direction of thickness, the requirements of KTA 3205-2 [82], Table 7-1 on pre-heating welding areas must be observed.

7.2.7.2 Installing anchor plates on site

Anchor plates must be installed by skilled personnel in accordance with formwork or specific installation drawings. Suitable steps must be taken to prevent them shifting during concreting, such as being bolted or nailed to the formwork.

The tack welds are often observed being used between headed studs and reinforcement but this is not permitted. Spot welding may cause local brittleness and softening of the material the studs are made of, and this may also lead to unwanted notch effects.

When placing anchor plates within the formwork and the reinforcement cage, care must be taken to ensure that the headed studs and the reinforcement are in the right position. In particular, back-tying reinforcement must be installed as specified in drawings, observing carefully the anchoring lengths shown. Adequate spacing is required to avoid cavities or shrink holes in the load induction area.

With horizontally fixed embedded parts, there is a risk of air penetrating during pouring, so the General Technical Approvals require ventilation bores to be made from an edge length of 400 × 400 mm. As feeding in fresh concrete under a horizontal anchor plate from one side is not a reliable method of avoiding air inclusions, it is advisable to provide ventilation bores from edge lengths as little as 200 × 200 mm.

Checks must be made to ensure that anchor plates are correctly installed and that reinforcement is in the right position, with installation records as evidence.

7.3 Fastenings with metallic anchors

7.3.1 History

Anchors come in many shapes and sizes, from domestic rawlplugs to heavy load anchors carrying several tons.

The first industrially made anchors were invented by John Joseph Rawlings in 1910. The UPAT company first made similar anchor of hemp string with metal sleeves in 1926. Fritz Axthelm of NIEDAX applied for the first patent for a metal spreading anchor two years later (DRP 555 384).

In building nuclear power plants, metallic anchors were also an essential structural element right from the start for attaching light to moderately heavy system parts and components. Path-controlled spread anchors of the Spit-Gold, TiFiX and Hilti-HKD types were installed in large numbers in accordance with the manufacturers' guidelines. The Liebig company supplied the Liebig safety anchor the first force-controlled force-spread heavy-load anchor used in German nuclear power plants. The first precursor of a General Technical Approval was issued by what was then the IFBt, today's DIBt, in 1972. This was the first anchor ever to have a General Technical Approval, issued in 1975. The Liebig safety anchor was first used in nuclear power plants in 1973, due to the particular requirements involved for attaching safety-related anchorings, based on approvals on a case-by-case basis. Liebig then released the Ultraplus, the first displacement-controlled anchor on the market. Together with the FZA anchor developed by the Fischer company, these two path-controlled anchor types were a major step forward in attachment systems in nuclear power plants.

Static verification for the anchoring with anchors was made until then as per approvals or consents on a case-by-case basis, but the so-called Kappa-method from 1988 and the introduction of the DIBt Guideline 'Design methods for anchors for anchoring in concrete' [87] based on the so-called CC method in 1993 revolutionised the verification of attachments in concrete.

In 1998, with the aim of creating a uniform evaluation basis for awarding consents in all German Federal States, the DIBt published the guidelines on ‘Using anchors in nuclear power plants and nuclear installations’ [63]. These guidelines recommended that only form lock anchors should be allowed for attaching safety-related components and system parts. Based on these guidelines, the first General Technical Approvals were issued, for Fischer’s Zykon-Bolzenanker FZA-K [88] in 1999 and for Hilti’s HDA undercut anchors [89] in 2000. These two approvals, and their current versions, provide a uniform design basis for the respective anchor models for all German nuclear power plants.

In June 2010, the DIBt published new guidelines entitled ‘Guidelines for anchor attachments in nuclear power plants and other nuclear installations’. This supersedes the 1998 version, and provides more differentiated details of the tests to be conducted, methods of verification and handling for anchors and anchoring to be approved for safety-related attachments subject to particularly high requirements in the event of accidental actions. It does not limit itself to undercut anchors so that other types of anchors may be approved for use in nuclear installations.

A KTA status report on allowing for the particularities of nuclear installations is expected in the near future.

7.3.2 Overview of anchor types

Anchor attachments may be divided, depending on how they work, into undercut, spread and composite anchors although there are also anchor system that combine two of these methods, such as composite spread anchors.

Undercut anchors can be divided into self-undercutting anchors and those in which the undercut is made in a preceding step. The preceding undercut is made using a special undercut tool or a special drill, swinging the drill out in a circular motion. The anchor is then placed using a setting tool (e.g. Fischer FZA-K, Figure 7.6). With self-undercutting anchors the undercut in the concrete is made by hard metal cutting at

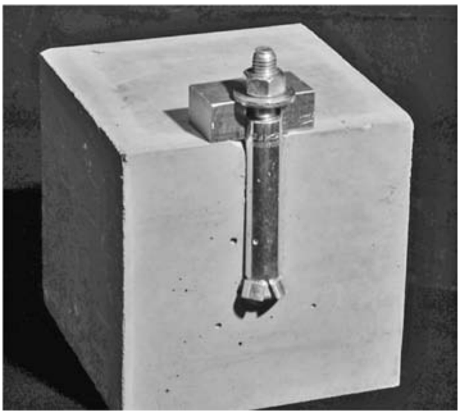

Fig. 7.6 Fischer Zykon bolt anchor FZA 18×80 KM12 [88]

Fig. 7.7 Hilti undercut anchor HDA-T-22-M12 × 125/30 [89]

the anchors sleeve (e.g. Hilti HDA Figure 7.7). Undercut anchors transmit tension loads, even if there are wide cracks, such as in the event of an earthquake, as they are anchored in a form lock way within the concrete: so the undercut must be inspected very carefully. With the products available on the market today, the undercut can be checked via coloured markings which must be visible if the anchor is installed correctly.

Spread anchors can also be divided into path-controlled and force-controlled (or even torque-controlled) anchors. Path-controlled spread anchors are installed by being hammered in or driven in by machine and then checked by measuring the set depth. Anchors can be marked to ensure they are installed to the set depth required. Force-controlled spread anchors are inserted using a torque wrench, which applies the tensile or spread force required for the anchor to grip as it should (Figure 7.8).

Fig. 7.8 Anchoring detail Liebig force-controlled safety anchor [90]

Spread anchors transmit tensile loads via the grip between the spread anchor and the surrounding concrete.

Composite anchors consist of a composite mortar containing an embedded metal component. The composite material may be made of synthetic mortar, cement mortar or a mixture of the two. In practice, cartridge and injection systems are used. With cartridge systems, glass or synthetic capsules are fitted into the borehole. When the anchor is inserted, the cartridge is destroyed and the chambers it contains are mixed with the two components of the mortar.

Injection systems consist of the metal component to be inserted and a two-chamber injection cartridge. The composite material is mixed when it is expelled from the cartridge and injected into the borehole. The metal component is then inserted by hand or mechanically, depending on the anchor system involved.

Composite anchors work via the grip between the metal part and the composite mortar and the grip between the composite mortar and the borehole wall. The grip with the

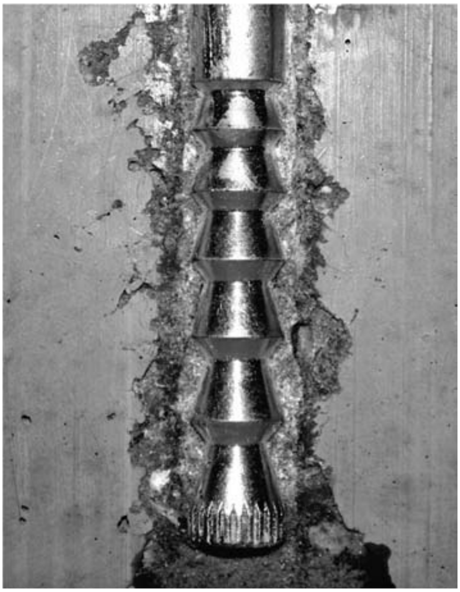

Fig. 7.9 Anchoring detail composite spread anchor MKT-VMZ M16

concrete means that great attention must be paid to cleaning the borehole particularly thoroughly in accordance with installation instructions.

Composite anchors can also include combinations with undercut or spread anchors (Figure 7.9).

The loads in a composite spread anchor are transmitted via a combination of bonding and spreading, in which the spreading is also achieved through its particular shape. This enables it to bridge even broad cracks up to 1.5 mm and shows a ductile load-bearing behaviour.

The load–deformation diagram for a composite spread anchor shows the wavelike course of the load very well. This is caused by the individual spread cones penetrating into the composite mortar. With smaller crack widths, all spread cones in a anchor can be assigned to the individual waves in the load–deformation diagram.

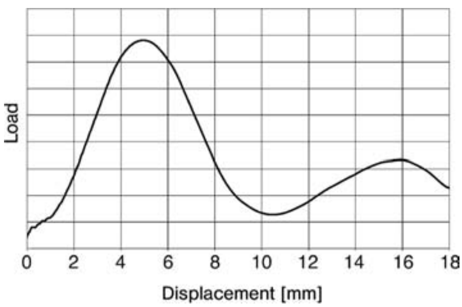

Fig. 7.10 Load-deformation diagram of a composite spread anchor (MKT-VMZ crack width = 1.5 mm)

With undercut composite anchors, the load is transmitted by a combination of the bonding of mortar to the borehole walls and the mechanical form locking of the mortar in the undercut of the concrete.

7.3.3 Safety concept

The safety concept in DIN 1055-100 [52] must be applied for the governing design situation:

$$S_d \leq R_d \tag{7.1}$$

The design actions are to be calculated as per DIN 25449 [15], which defines the specific actions for nuclear power plants and nuclear facilities.

The design resistance is measured the equation:

$$R_d = R_k/\gamma_M \tag{7.2}$$

The partial safety factor γ_M of resistance must be considered more closely here, as this has to be determined, not generally but specifically for the failure mode and requirement category. For anchors, the partial safety factor γ_{Mc} for concrete failure is obtained from three other partial safety factors γ_c, γ_2 and γ_A:

$$\gamma_{Mc} = \gamma_c \cdot \gamma_2 \cdot \gamma_A \tag{7.3}$$

where γ_c is the partial safety factor for concrete, depending on the requirement category. Partial safety factor γ_2 reflects the installation safety of the anchor and γ_A is the specific anchor partial safety factor for attachments in nuclear power plants. In requirement categories A2 and A3, the only anchors that may be used are those with a high installation safety $\gamma_2 = 1.0$. The value γ_A is set to give a partial safety factor for concrete failure of $\gamma_{Mc} = 1.5$ under all conditions.

Turning to steel failure, the partial safety factor γ_{Ms} is determined as a function of the load direction and material properties. For tensile loads applies:

$$\gamma_{Ms} = \frac{1,2}{f_{yk}/f_{uk}} \geq 1,4 \tag{7.4}$$

For shear loads:

$$\begin{aligned} \gamma_{Ms} &= \frac{1,0}{f_{yk}/f_{uk}} \geq 1,25 \\ & f_{uk} \leq 800\,\text{N/mm}^2 \text{ and } f_{yk}/f_{uk} \leq 0.8 \end{aligned} \tag{7.5}$$

$$\begin{aligned} \gamma_{Ms} &= 1.5 \\ f_{uk} &\geq 800\,\text{N/mm}^2 \text{ or } f_{yk}/f_{uk} \geq 0.8 \end{aligned} \tag{7.6}$$

7.3.3.1 Installation safety

In various nuclear power plants and nuclear facilities, there are many anchors that were not installed in the correct position, so they were replaced. According to [67], the anchors used to attach safety-related components must be designed such that they can be checked easily to verify that they have been installed correctly from easily recognisable, objective and doubtless criteria when setting and once installation is completed.

If plans are not followed, the structural engineer must be consulted.

Anchor fastenings must always be installed in accordance with the manufacturers' instructions, but there are numbers of other conditions which must also be observed:

- Incorrect drillings and damage to existing reinforcement should be avoided by detection.
- The anchoring plate connection area should be even, which can be achieved by applying a thin mortar smoothing layer.
- The distances required from edges causing disturbances must be maintained.
- Bores must be done at right-angles.
- Incorrect drillings must be closed with high strength concrete.

As evidence that anchor fastenings have been properly installed at nuclear power plants and nuclear facilities, an installation report must be produced for each attachment which must be verified by the client/operator and by a licensed structural engineer or the construction inspector. Instructions as to the content of such reports must be taken from those for use in nuclear power plant approvals. The data to be recorded for each location of anchor fastening are:

- application to change/notice of change
- date installed
- client's/operator's representative
- installation contractor plus professional construction manager for dowelling
- construction inspector
- fitter (training certificate)
- building
- area
- system
- anchor plate ID no.
- layout drawing
- workshop drawing
- anchor manufacturer
- product designation
- size
- material
- tools used
- borehole checks
  - clean
  - right-angled
  - depth
  - diameter
  - incorrect drilling present/closed

  - reinforcement damaged
  - detectable cracks/local damage
  - corrosive environment
- check torque
- check anchor plate
  - made to workshop drawing
  - plate thickness
  - axis-edge distances
  - through bore diameter
  - concrete surface/thickness of smoothing layer at anchor
- check surroundings
  - distances from adjacent fastenings
  - geometric constraints.

7.3.4 Approvals

7.3.4.1 General

In Germany, non-standard construction products are qualified by a so-called General Technical Approval given from the Deutsches Institut für Bautechnik (DIBt). A General Technical Approval confirms that a non-standard construction product or construction method may be used under German Federal State building regulations.

Since Council Directive 89/106/EEC of 21 December 1988 on the adjustment of laws, regulations and administrative provisions of the European Member States relating to construction products was introduced, the DIBt can also issue European Technical Approvals (ETAs). An ETA for metallic anchors is issued under ETAG 001 [64]. These guidelines for the European Technical Approval for metallic anchors for anchoring in concrete contains documents for assessing anchors and are divided into six parts and three annexes.

7.3.4.2 Tests according to DIBt guideline

Issue 9/98 of the Deutsches Institut für Bautechnik's Guideline on 'Using anchors in nuclear power plants and nuclear installations' [63] constitutes for the first time what tests are required over and above the General Approval in order to facilitate use of anchors for safety-related attachments in nuclear power plants.

This Guideline contains details of the tests needed to simulate the extraordinary stress situations involved in an earthquake. To prove their suitability, anchors must withstand a monotonic tension load, alternating loads at constant crack widths and varying crack widths at constant loads at a crack width of 1.5 mm. These test loading conditions simulate comprehensively the accidental stresses due to earthquake actions. To obtain the characteristic tension loads, monotonic tensile tests are conducted at an open crack width of 1.0 mm. The characteristic shear strength is determined by alternating shear load tests, in which anchors are exposed to 15 times the alternating shear loads in the direction of the crack. The residual load-bearing strength is then determined by a monotonic shear tension test.

The new edition of the Guideline of June 2010 [67] revised the crack widths. This also allows for the specific crack widths that are expected at the place of use to be used in tests. However, this requires a detailed verification of the characteristic crack widths under accidental actions. An additional section has also been added on testing to determine realistic anchor shifts. Under the DIBt Guideline, anchor shifts are determined by tests in opening and closing cracks at a constant tensile load acting on the anchor and tests at alternating loads on the anchor with the crack opened, varying the crack widths as well as variable alternations of crack widths and loads. To pass these tests, anchors must satisfy a numbers of requirement defined in the DIBt Guideline.

7.3.5 Design and dimensioning

Anchor fastenings for use in nuclear power plants and nuclear facilities must be designed in accordance with the DIBt Guideline.

7.4 Corrosion protection

As fastenings cannot be accessed once they are installed, safety-related components in particular are subject to particular corrosion protection requirements, depending on the ambient conditions involved.

Inner areas will very generally be dry, so under these conditions, protective coatings or galvanising provides sufficient corrosion protection.

Stainless steel materials must be used in outside areas and in inside areas where there are corrosion factors. Under DIN EN ISO 13918 [73], headed studs are in group SD3 with materials 1.4301 and 1.4303 to DIN EN 10088 [70]. For steel plates, materials 1.4571 and 1.4401 to DIN EN 10088 are used.

Where particularly corrosive influences are present, such as chemical pollution, stainless materials must be checked to see whether they can be used in each case.

Anchor plates made of currently standardised materials must not be used in chlorinated atmospheres.

7.5 Fire resistance

Anchor plates with headed studs or metallic anchors consist of non-flammable materials and can therefore be assigned to fire protection class A1 under DIN EN 13501 [91]. Should anchor plates be subject to specific requirements in terms of fire resistance time, structures must be tested in accordance with the test procedure prescribed for their class and their fire resistance class is to be specified according to DIN EN 13501.

8 Waterproofing of structures

8.1 Purposes on waterproofing structures

Structures are waterproofed primarily to protect them against penetrating water, which may appear as soil humidity, non-accumulating seepage water, accumulating seepage water, unpressurised surface water and water pressing in from outside. Waterproofing is also used to contain radioactively contaminated liquids arising inside them, particularly in safety-related structures in nuclear power plants.

Structural waterproofing as a 'black tank' is applied to the outside of structures – on the side facing the water – and encloses them as a basin or trough with a tightly waterproofed skin. Where a structure is designed as a 'white tank', on the other hand, the reinforced concrete structure serves not only to bear the load but also to waterproof the structure.

8.2 Requirements of waterproofing structures

With nuclear power plants, system components must be protected against effects so that they can do their job in operating conditions as intended and if accidents arise [92]. This puts additional requirements on waterproofing structures. Including external effects (earthquakes, aircraft impact, explosion pressure waves) and internal effects from accidents as the case may be, structure waterproofing is subject not only to static, but also to higher transient dynamic loads.

8.3 Black tank

As a general rule, waterproofing structures to protect against water penetration are carried out in accordance with the DIN 18195 series of standards. How this structural waterproofing behaves under what are normally long-term static loads is sufficiently known; but the design constraints developed from the series of standards above are not always sufficient for nuclear structures. Special load cases which act on the structures which carry the structural waterproofing can cause deformation and displacement which affect the structural waterproofing.

In these special load cases, as well as the localised high levels of pressure from the working load, a number of other types of stress can also arise:

- transient higher transverse compression stress
- transient intermittent transverse tension stress (gaping gap opening/nominal fracture point between structures and their environments)
- transient intermittent shear stress at the waterproofing level.

How structural waterproofing behaves in terms of bridging cracks is also important.

8.3.1 Waterproofing methods and materials

The specifications of DIN 18195-6 [93] 'Waterproofing buildings, proofing against outside pressing water and accumulating seepage water, design and execution' largely represents the current state of the art. Continuing intensive technological developments have led to both new waterproofing materials and new waterproofing methods.

Design and Construction of Nuclear Power Plants. First Edition.
Rüdiger Meiswinkel, Julian Meyer, Jürgen Schnell.

Building waterproofs with different kinds of waterproofing strips and adhesives made in suitable combinations may be described as part of the state of the art. The individual components from which building waterproofs are made are defined in the materials tables in DIN 18195-2 [94] 'Structural waterproofing'. The state of the art in science and technology is defined in KTA 2501 [92] 'Structural waterproofing in nuclear power plants'. Waterproofing structures of various kinds are defined in Table 8.1.

Table 8.1 Types of waterproofing structure, from [92]

<table>
<tr><th rowspan="2">Struc-ture Type</th><th rowspan="2">Pressure Loading Condition, p_{stat}. and Flow Path, R</th><th colspan="2">Layer Sequence independent of the penetration depth</th></tr>
<tr><th>Foundation Slab (from top to bottom)</th><th>Wall (from inside to outside)</th></tr>
<tr><td rowspan="9">1</td><td rowspan="9">$\leq$0,6 MN/m^2 and $\geq$10.0 m</td><td>(Protective Concrete)</td><td>(Structural Concrete)[a),b)]</td></tr>
<tr><td colspan="2">Adhesive layer of unfilled B 85/25, 1.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937</td></tr>
<tr><td colspan="2">Adhesive layer of unfilled B 85/25, 1.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Bitumen sheeting for waterproofing of roofs G 200 DD, in accordance with DIN 52130</td></tr>
<tr><td colspan="2">Adhesive layer of unfilled B 85/25, 1.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937</td></tr>
<tr><td colspan="2">Adhesive layer of unfilled B 25, 1.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td>(Subconcrete)[a)]</td><td>(Protective Coating)</td></tr>
<tr><td rowspan="9">2</td><td rowspan="9">$\leq$1.5 MN/m^2 and $\geq$2.0 m</td><td>(Protective Concrete)</td><td>(Structural Concrete)[a),b)]</td></tr>
<tr><td colspan="2">Adhesive layer of filled B 85/25, 2.0 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937</td></tr>
<tr><td colspan="2">Adhesive layer of filled B 85/25, 2.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Calotte-checkered copper band CU-DHP 0.1, in accordance with DIN EN 1976</td></tr>
<tr><td colspan="2">Adhesive layer of filled B 85/25, 2.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td colspan="2">Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937</td></tr>
<tr><td colspan="2">Adhesive layer of filled B 25, 1.5 $kg/m^2 \pm 0.5$ kg/m^2</td></tr>
<tr><td>(Subconcrete)[a)]</td><td>(Protective Coating)</td></tr>
</table>

3	$\leq 2.0\,MN/m^2$ and $\geq 1.0\,m$	(Protective Concrete)	(Structural Concrete)[a),b)]
		Adhesive layer of filled B 85/25, $2.0\,kg/m^2 \pm 0.5\,kg/m^2$	
		Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937	
		Adhesive layer of filled B 85/25, $2.5\,kg/m^2 \pm 0.5\,kg/m^2$	
		Calotte-checkered copper band CU-DHP 0.1, in accordance with DIN EN 1976	
		Adhesive layer of filled B 85/25, $2.0\,kg/m^2 \pm 0.5\,kg/m^2$	
		Bitumen sheeting for waterproofing of roofs G 200 DD, in accordance with DIN 52130	
		Adhesive layer of filled B 85/25, $2.0\,kg/m^2 \pm 0.5\,kg/m^2$	
		Calotte-checkered copper band CU-DHP 0.1, in accordance with DIN EN 1976	
		Adhesive layer of unfilled B 85/25, $2.5\,kg/m^2 \pm 0.5\,kg/m^2$	
		Plasticized-PVC sheet, bitumen compatible, 2 mm, in accordance with DIN 16937	
		Adhesive layer of unfilled B 25, $1.5\,kg/m^2 \pm 0.5\,kg/m^2$	
		(Subconcrete)[a)]	(Protective Coating)

[a)] The subconcrete of the foundation slab and the structural concrete of the wall shall be prepared such that an average adhesive tensile strength $\beta_{HZ} \geq 1.5\,N/mm^2$ (single values $\geq 1.0\,N/mm^2$) is achieved. The certification shall be performed in accordance with Appendix 2, ZTV-SIB 90.
[b)] In case the wall region of the structural concrete must be poured against the waterproofing, then special measures to enhance the adherence shall be provided for the border surface between the structural concrete and the waterproofing (e.g., in accordance with Sections 5.12 and 5.13 ARBIT-Brochure No. 61 "Waterproofing with Bitumen").

Note: In the case of other types of waterproofing structures and the use of other waterproofing materials, cf. Section 4.1.7.

High soil pressures from permanent loads cause the bitumen in bituminous waterproofs to 'extrude' laterally. This effect can be counteracted by inserting copper ripple plates in the bitumen strips.

Which waterproofing structure can be used depends primarily on what stresses act on a building waterproof. As well as the verifications required, installation and design issues and particular aspects of execution must also be taken into account.

8.3.2 Designing structural waterproofing

Structural waterproofing must be designed to withstand the stresses acting immediately on it. These include, on the one hand, the permanent compression, composed of water pressure and ground pressure or soil pressure as the case may be, and on the other hand the cracking in the structures bearing on the structural waterproof. As the manner in which the reinforced concrete structures which carry the structural waterproof crack depends on their design: there is a connection between designing those reinforced concrete structures and the design of the structural waterproof.

When designing structural waterproofing, the behaviour must also be considered at pressures which are significantly greater than the limits stated in the rules. More extensive requirements of structural waterproofs in nuclear installations also result from the special load cases mentioned above. The stresses that these cause on structural waterproofs must be determined (cf. [92], Section 4.1.1 et seq.).

Design rules for structural waterproofs for bridging cracks are defined in [93]. They apply to stress cracks not more than 0.5 mm at their point of origin and opening gradually over long periods of time up to 5 mm. These design rules cannot be used for cracks several millimetres wide that open spontaneously or open and close rapidly due to the risk of the waterproof structure suffering fatigue cracks, even though tests on some waterproofing structures have proven that a structural waterproof is able to bridge such cracks to a limited extent (cf. [92] and Section 4.1.6).

8.3.3 Structural detailing

The structural geometry around the structural waterproofing must always be defined in the knowledge of the waterproofing structure to be made, involving the specific characteristics of the waterproofing structure.

In this context, we would refer in particular to the rules and regulations for structural waterproofs using bitumen adhesives.

As well as the structural design in principle, there are a number of other factors which play a role:

- designing the concrete base and protective layers (see DIN 18195-10 [95])
- designing the structural joints (movement joints)
  - structural joints in common sealed tanks
  - structural joints between separate sealed tanks
- design of embedded parts.

8.3.4 Designing the structural waterproofing

Structural waterproofing is best designed in two chronologically separate phases [96]:

- inspection and permitting design phase
- execution design phase.

8.3.4.1 Inspection and permitting design phase

Inspection and permitting design is part of the construction and Atomic Energy Act approval procedure. Designs must generally pass inspection before they can be approved.

Inspection and permitting application documents should contain, as a minimum requirement:

- details of the structural waterproof
- layout plan
- overview drawings

– standardised design details
– list of annexes.

The structural waterproofing design should include:

– a list of the structures with structural waterproofing
– foundation depths
– details of ground surface level, power plant zero level, design water levels and flood water levels, design water levels, high water levels (permanent high water level to KTA rule 2207 [23])
– details of waterproofing strategy
– details of waterproofing method
– service and special loads
– design rules for service loads
– making penetrations
– verification of suitability.

8.3.4.2 Execution design phase

As the structural waterproofing execution design and the static load calculations and formwork drawings are dependent on one another, the execution design is carried out at more or less the same time. The structural waterproof must be designed to meet the detail of the stresses acting on it. Any verifications of suitability not to hand must be provided. All the data required for execution must be recorded in overview and detail drawings.

Ground plans, sections, views and even developments, if required should include as a minimum:

– axes, main dimensions, heights
– details of the number and type of layers (designed to meet compression stresses and the flow path of the bitumen adhesive to [92])
– general details, such as the sub-concrete and protective layers
– details of corrosion-protecting steel components
– references to detail drawings and standard details to be used
– references to connecting drawings
– details of settlement differences and other movement processes at structural joints.

8.3.5 Construction of the structural waterproofing

To provide permanent protection, structural waterproofing not only must it be designed professionally and quality assured, but it must also be built accordingly. When building the structure, the external conditions involved while making the structural waterproof play a governing role.

The issues of importance in connection with building the structure, that make a major contribution to quality assurance (as below), are:

– particular measures to be taken if constructing in bad weather
– temporary protection measures:
  – protecting the structural waterproof on horizontal and gently sloping surfaces
  – protecting waterproof connections

- protecting against thermal effects
- protecting the structural waterproof on wall surfaces when placing reinforcement
- protecting against penetrating groundwater, accumulating and surface water during construction
- protecting against the penetration of harmful substances.

8.3.6 Quality assurance

For structural waterproofs to work perfectly, their design and execution must be quality assured. Both the processes and the materials involved must be tested and monitored. There are a number of methods available for testing completed structural waterproofs. Test results showing that a structural waterproof is as it should be must be recorded in test reports.

Inspecting the structural waterproof should be entrusted to a separate expert as part of the construction law approval process.

8.4 White tank

White tank designs may be considered as structural waterproofing in new building projects, supplemented by black waterproofing if necessary. Additional measures may be required to run off standing water or other liquid media, by way of drainage etc. [97].

Below, we will look at the design principles for building white tanks, focusing on their waterproofing effects against water penetrating from outside.

8.4.1 System specification

8.4.1.1 General requirements

Reinforced concrete structures with high resistance to water penetrating prevent water penetrating permanently in liquid form. As well as bearing loads, they also act as structural waterproofing.

Designing and constructing white tanks in Germany is governed by the DAfStb's guidelines on 'Water-impermeable concrete structures', (WU guidelines) [98], as the generally accepted rules of the art. The WU guidelines provide instructions on requirements for fitness for use of water-impermeable reinforced concrete structures. DAfStb vol. 555 'Explanatory notes to the DAfStb guidelines' [99] contains notes to the WU guidelines. Instructions are contained in DBV bulletin on 'High-grade use of basement floors – building physics and indoor climate' [100].

There are a number of constraints to be considered when building a white tank:

- nature of the moisture penetration
- design water levels
- type, characteristics and permeability of the subsoil
- chemical characteristics of the water
- establishing the type of use involved, particularly in the light of the stresses and special loads from external and internal events.

Supplementary drainage measures may be taken to protect the structure, particularly against non-pressured water or temporarily accumulating water. For drainage design, dimensioning and execution rules, see DIN 4095 [101].

8.4.1.2 Engineering principles

Conditions of use
When establishing the basic design concept for a white tank, there are a number of local conditions to be considered:

– height of the reference groundwater level (and/or height of accumulating seepage water)
– height of the minimum groundwater level before starting use
– chemical composition of the groundwater.

If there are any liquid factors to be dealt with from inside, the following must also be established:

– maximum possible height of the liquid level surface
– chemical composition of the liquid
– maximum possible temperature of the liquid
– duration of the liquid load.

Particular attention must be paid to limiting any water or other liquids escaping in the event of an incident.

Establishing categories of use under the WU guidelines cannot be applied directly to nuclear power plants, particularly because of what is required in the event of an incident. Instead, unique findings must be made for each individual structure, broken down by structural components, such as floor slab, walls and so on, if possible.

In terms of groundwater loads from outside, we need to define to what extent limited local access of groundwater can be accepted:

– in the phase of structural works, shell and core
– before commissioning
– in use
– during and after incidents.

Requirements for liquid stresses from inside must be defined analogously.

Except in the event of an incident, external structures should be designed and dimensioned in accordance with the DAfStb guidelines on concrete structures when dealing with water pollutants [102].

Design principles
If separating cracks arise in WU structures, these can admit large quantities of water [103]. Even fine cracks usually admit more water than can be displaced by air from the inside. How much water gets in depends mainly on how wide a crack is, how thick a structure is and how high the water level is.

When dealing with separating cracks, the design principles which can be used for nuclear power plant structures exposed to standing groundwater are:

– avoiding separating cracks
– designing to waterproof cracks on schedule prior to commissioning
– cracks waterproofing through self-healing
– designing to run off penetrating water.

Whether individual design principles can be implemented must be considered in the light of the construction timetable.

The design principles selected must be justified and recorded, and the findings made must be included in contracts between the parties involved.

8.4.2 Particular requirements

The inside surfaces of white tanks must be kept permanently free for inspection purposes and to carry out any waterproofing which becomes necessary. Should they only be accessible by dismantling replacement parts, equipment and so on, this must be capable of being done with operations on the run. If access cannot be assured in the area of floor slabs under machinery and so on, structural design measures must be taken to ensure that, if any water does penetrate, it cannot cause damage and can be run off as designed. Wall claddings on the inside of WU structures (liners and tiles) are not allowed.

8.4.3 Design and calculation

For optimum design of white tanks in terms of demands and use, the design principles as stated in the WU guidelines must be followed. Other factors to be taken into account in the design concept include in particular when plant is to be commissioned, the water effects when it is in use and the nature of its use.

8.4.4 Joint detailing

Joints in reinforced concrete structures which are highly resistant to water penetration must be permanently water-impermeable with regard to the reference water level and subsoil conditions.

8.4.5 Penetrations

Penetrations must be waterproofed against water pressure as a matter of course, by using standard systems and/or certified products, for example with sleeve tubes and annular waterproofs, flange pipes with rigid pipe connections or core drills with medium pipes and annular waterproofs.

8.4.6 Responsibilities

When deciding on a white tank, everyone involved, and in particular those involved in the design, plus the client and any contractors already involved when the design stage starts, must be familiar with the design principles involved in a white tank. All findings and decisions required under the WU guidelines in terms of design, implementation and quality assurance must be recorded. All those involved *must* work together. Competences and responsibilities must be laid down and recorded in writing, clearly and unambiguously, before starting work.

8.4.7 Quality assurance

Erecting and supervising the construction of structures in nuclear power plants are governed largely by laws and enacting provisions. Supervision is enshrined in law in Federal State building regulations.

As well as tests conducted by experts instructed by clients and approval authorities, testing is also conducted extensively based on companies' internal quality assurance systems.

8.4.8 Repairs

Waterproofing separating cracks and leaking joints and repairing faults is carried out using waterproofing agents to DIN 1504-1 [104] and DAfStb guidelines on 'Protection and repair of reinforced concrete structures' [105]. Such works must be accessible at all times.

Retrospective waterproofing is carried out using supervised and tested crack fillers such as epoxy resins (EP-I), polyurethane resins (PUR-I) and cement-bound systems (cement adhesive, ZL-I, and cement suspensions, ZS-I).

Water-bearing separating cracks must be waterproofed with polyurethane resin.

Polyurethane resin as a filler can be used to:

– close
– waterproof
– limit joint stretching.

Specific material conditions must be taken into account when using this filler.

Cement-bound fillers can only be used to a limited extent, as the joint waterproofs they create are rigid and can only stretch to a limited extent.

Where repairs are required – especially to safety-related structures – the measures required must follow the latest state of the art of science and technology as far as is necessary at each stage, and carried out in accordance with repair regulations for power plants on site.

8.5 Waterproofing concept using the example of the OL3 nuclear power plant

We will now look in outline at the waterproofing concept selected at the OL3 nuclear power plant in Finland.

Buildings always conduct their loads via base slabs into the underlying rock. The difference in heights between the rock excavated and the underside of the building floor slab, which may be considerable in some cases, is made up for by using concrete with light constructive reinforcement.

The nuclear island structures below the 0.00 m level are made partly as white tanks, restricting crack widths accordingly ($\leq$0.2 mm in part). A largely stress-free support for the main buildings was achieved, and waterproofing was also provided against seepage and stratum water which can get to the structure via cracks and spaces in the rock in the shape of an external black waterproofing on all vertical wall surfaces from below the top of ground height up to +0.60 m, and all horizontal upper structural surfaces, such as upper duct connections below ground height.

The waterproof is built up as follows:

Waterproofing on walls below ground surface level

- plastic-modified bitumen undercoat 0.2–0.3 l/m^2 (cold)
- one layer of plastic-modified bitumen strip.

Additional thermal insulation

- Polystyrol XPS foam, compressive strength $\geq$180 kN/m^2
- thickness depending on where applied, between 100 and 160 mm.

Waterproofing upper structural surfaces below ground surface level

- sloping concrete at least 2%, at least 100 mm thick, with constructive reinforcement (steel mat c/c100, d = 6 mm),
- plastic-modified bitumen undercoat 0.2–0.3 l/m^2 (cold)
- one layer plastic-modified bitumen coating.

Additional thermal insulation

- Polystyrol XPS foam, compression strength >250 kN/m^2
- thickness 140 mm.

The equalising concrete has drains fitted to drain the joint between it and the rock. This drainage was tailored to suit the local rock structure (depths/interfaces): it leads any water which occurs into the drains surrounding the building, which in turn lead it to the pump shafts from which the water is then pumped out.

The drains around the building consist of perforated PP pipes DN 2 × 110 or 200 in drainage gravel 6/16, enclosed in geotextile wool. In areas concreted against vertical rock surfaces, drainage mats are used to ensure that the rock surface is drained.

9 Ageing and life cycle management

9.1 Overview

To ensure that nuclear power plants are safe and reliable, the system components and structural systems must be built to the quality required. Life cycle management is generally defined as a combination of ageing management and financial planning. Ageing management is thus part of life cycle management, and covers everything that the operators need to do to ensure that their nuclear power plants remain safe as they get older. The fundamental principles here are laid down by KTA 1403 [106] which gives rules for ageing management at nuclear power plants.

The main role of ageing management in structural installations is to map what may happen to building materials as they age and prevent their having harmful effects specifically and effectively. These ageing mechanisms are also known as physical ageing: so ageing management deals primarily with physical ageing, with measures to ensure permanence and in particular the structural safety of building structures in use.

The term life cycle management is used instead of ageing management if the focus is on financial aspects as well as structural safety. Life cycle management thus goes beyond ageing management: as well as physical ageing, it also includes conceptual ageing and technological ageing (see Figure 9.1).

Conceptual ageing covers changes to design standards due to introducing new rules such as standards and guidelines with altered requirements or changes to safety philosophy. These changes must be assessed to see how far they are relevant to the structural safety of buildings designed to the 'old' rules.

Technological ageing covers changing findings in terms of possible harmful operating mechanisms and material characteristics. It also covers innovations in verification methods (changes to calculation standards) and testing and calculation methods. It is relatively unimportant in structural engineering compared with plant engineering, as the characteristics of the principal materials, concrete, reinforced concrete and building steel, have remained fundamentally unchanged for the past 40 years.

Like life cycle management, ageing management involves maintaining buildings as knowledge-based preventive maintenance. This preventive maintenance covers the different measures involved in terms of maintenance, inspection, repairs and strengthening as shown in Figure 9.2.

9.2 Ageing management of buildings

Ageing management of nuclear power plants as built structures focuses on the structures that are safety-related. These structures are made of solid reinforced concrete, with the structural component dimensions not being governed by the effects of actions occurring involved in normal operation, but rather in terms of the requirements of protective effects against radiation and designed to cope with internal events, such as coolant loss, or rare external events, such as earthquakes, explosion pressure waves and aircraft impact.

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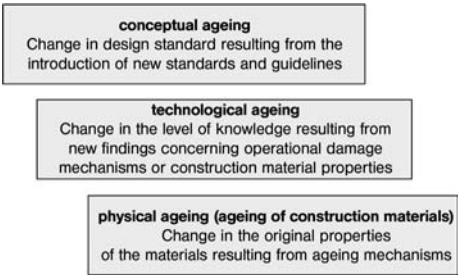

Fig. 9.1 Ageing phenomena

Many of the factors that have to be considered as part of ageing management occur in conventional structures just as much as they do in nuclear power plants, so that the knowhow and experience of harmful mechanisms in conventional structures can be applied directly. What is peculiar to nuclear power plants are the radioactivity and increased ambient temperatures which can cause materials to decay if exposed to them.

KTA 1403 [106] divides structural systems into structures/substructures, construction systems and structural components. Structures/substructures are complete buildings or larger sections of buildings to be identified via the power plant coding system (see Section 4.2.1). Construction systems are groups of structural components that perform a common function, such as steel platforms, sealing against water pressure and structural fire protection elements. Construction systems consist of structural components such as anchor plates, fire doors and fire stop valves.

Structures/substructures, construction systems and structural components need to be classified in accordance with their safety requirements. For ageing management purposes, those that need to be considered are those that are safety-related.

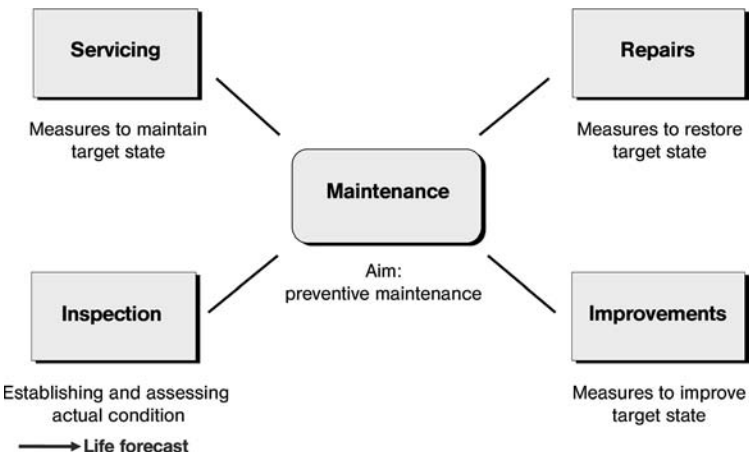

Fig. 9.2 Ageing management concept

Building structures and buildings need to be checked regularly to see if they have departed from nominal in any way, such as faults due to subsoil settlement or cracking, corrosion or plastics losing their seal and protective functions. Conducted as preventive maintenance, these inspections can be used to detect ageing mechanisms in good time and take appropriate follow-up measures to prevent serious damage.

9.3 Ageing mechanisms in building materials

Ageing mechanisms in building materials (physical ageing) involve damage mechanisms which cause material characteristics to deteriorate, and which can be caused by the exposures below:

– mechanical attacks, such as stresses imposed by temperatures
– physical attacks (frost, temperature changes and humidity)
– chemical attacks (acids, alkalis etc.)
– biological attacks (bacteria and fungi).

These effects set off ageing mechanisms which, for essential building materials, include:

– concrete: cracking, creep and shrinkage, swelling, secondary curing, carbonisation, damage due to chloride or sulphate attack, alkali reaction, solvent attacks, such as by acids and salts, swelling sulphate attacks, growths (e.g. algae), radiation
– reinforcement steel, pre-stressing steel, construction steel: corrosion
– pre-stressed concrete: loss of tension due to creep and shrinkage
– plastics: fatigue
– coatings: bubbling, cracking, chalking.

Ageing mechanisms can also be caused by changes to the subsoil, such as faults in building structures (cracks), operating problems (skewing turbine foundations etc.) due to the subsoil settlement under load, or deformations or changes of form in dykes through soil consolidation, sagging, settlements or external events.

9.4 Implementation and documentation

The measures required for ageing management purposes depend on the maintenance strategy implemented (cf. Figure 9.2). As well as the maintenance work itself, these include the inspections with regular tests and special tests, repairs and strengthening in the event of structural changes.

The basic procedure for conducting ageing management to KTA 1403 is shown in Figure 9.3 (PDCA cycle). Ageing management is knowledge-based, incorporating all information on structural systems and being constantly updated.

Inspections and structural examinations are used to establish what condition each structure is in, and to detect faults and changes. They are conducted at regular intervals in accordance with specified structural lists and the results recorded in status reports, so that trends can be followed in status reports and any faults observed can be assessed and repairs made.

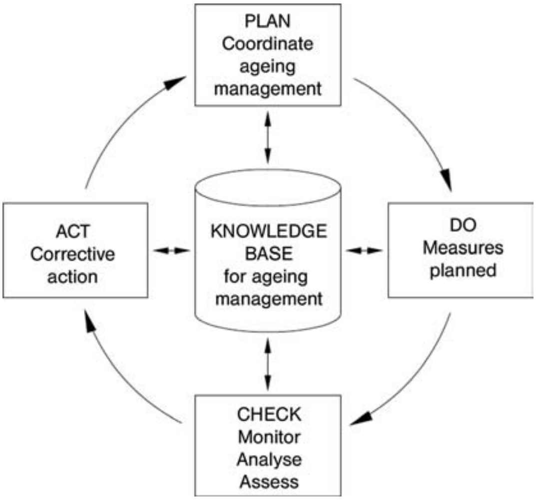

Fig. 9.3 PDCA ageing management cycle [106]

Structural examination methods are laid down in instructions. Each examination is normally followed by visual checks, simple measurements or as part of geodetic survey programmes, using preset criteria to specify more intensive examination methods.

In terms of reporting, KTA 1403 requires specific system basic reports, annual status reports and building status reports for structural systems (every 10 years). The basic report describes the process of ageing management, including organisation, and includes ageing relevant findings. Status reports contain details of ageing relevant activities and measures, findings and results during the reporting period. Building status reports are designed to show that safety-related structures have been assessed to see how they have aged.

Basic and status reports, which both cover mechanical engineering, electrical engineering and structural engineering, may also include individual specialist reports. For structural systems, for example, a ‘Basic report – structural engineering’ and ‘Status report – structural engineering’ (including building status report issues), can be prepared.

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