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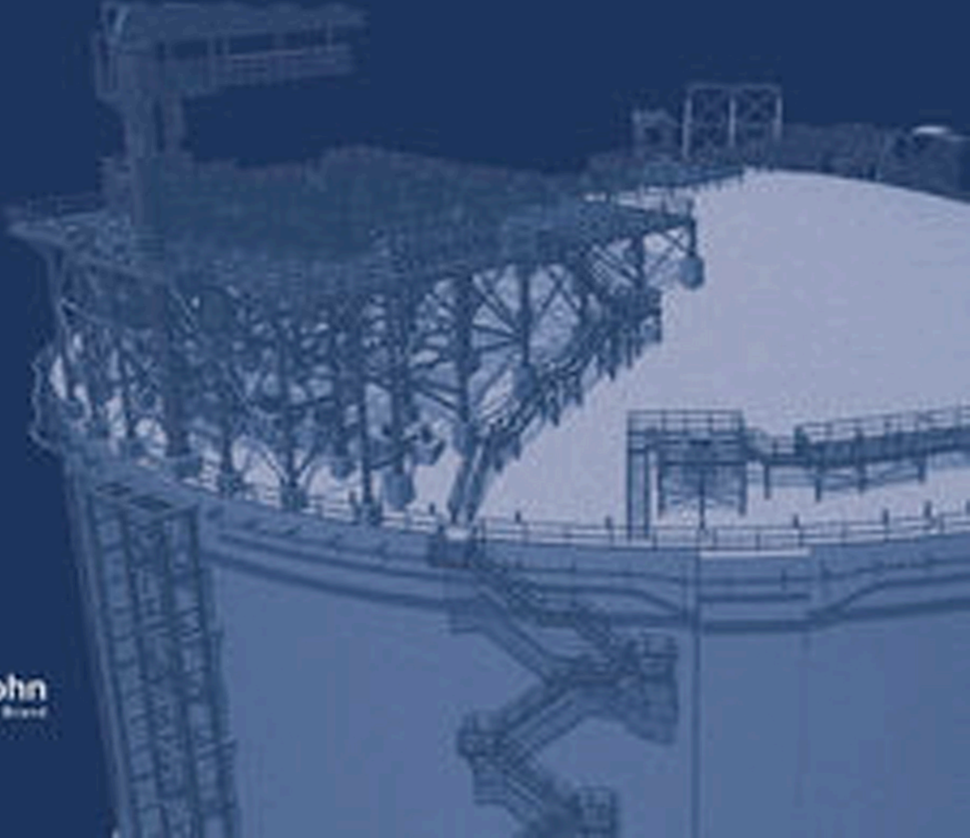
BetonKalender

Design and Construction of LNG Storage Tanks

Josef Rötzer



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Cover: LNG tank with typical steel structure
Photo courtesy: Günther Sell, TGE Gas Engineering GmbH, Munich

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Contents

Editorial

The *Concrete Yearbook* is a very important source of information for engineers involved in the planning, design, analysis and construction of concrete structures. It is published on a yearly basis and offers chapters devoted to various, highly topical subjects. Every chapter provides extensive, up-to-date information written by renowned experts in the areas concerned. The subjects change every year and may return in later years for an updated treatment. This publication strategy guarantees that not only is the latest knowledge presented, but that the choice of topics itself meets readers' demands for up-to-date news.

For decades, the themes chosen have been treated in such a way that, on the one hand, the reader gets background information and, on the other, becomes familiar with the practical experience, methods and rules needed to put this knowledge into practice. For practising engineers, this is an optimum combination. In order to find adequate solutions for the wide scope of everyday or special problems, engineering practice requires knowledge of the rules and recommendations as well as an understanding of the theories or assumptions behind them.

During the history of the *Concrete Yearbook*, an interesting development has taken place. In the early editions, themes of interest were chosen on an ad hoc basis. Meanwhile, however, the building industry has gone through a remarkable evolution. Whereas in the past attention focused predominantly on matters concerning structural safety and serviceability, nowadays there is an increasing awareness of our responsibility with regard to society in a broader sense. This is reflected, for example, in the wish to avoid problems related to the limited durability of structures. Expensive repairs to structures have been, and unfortunately still are, necessary because in the past our awareness of the deterioration processes affecting concrete and reinforcing steel was inadequate. Therefore, structural design should now focus on building structures with sufficient reliability and serviceability for a specified period of time, without substantial maintenance costs. Moreover, we are confronted by a legacy of older structures that must be assessed with regard to their suitability to carry safely the increased loads often applied to them today. In this respect, several aspects of structural engineering have to be considered in an interrelated way, such as risk, functionality, serviceability, deterioration processes, strengthening techniques, monitoring, dismantlement, adaptability and recycling of structures and structural materials plus the introduction of modern high-performance materials. The significance of sustainability has also been recognized. This must be added to the awareness that design should focus not just on individual structures and their service lives, but on their function in a wider context as well, i.e. harmony with their environment, acceptance by society, responsible use of resources, low energy consumption and economy. Construction processes must also become cleaner, cause less environmental impact and pollution.

The editors of the *Concrete Yearbook* have clearly recognized these and other trends and now offer a selection of coherent subjects that reside under the common "umbrella" of a broader societal development of great relevance. In order to be able to cope with the corresponding challenges, the reader can find information on progress in technology, theoretical methods, new research findings, new ideas on design and construction, developments in production and assessment and conservation strategies. The current selection of topics and the way they are treated makes the *Concrete Yearbook* a splendid opportunity for engineers to find out about and stay abreast of developments in engineering knowledge, practical experience and concepts in the field of the 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*

About the Author

Dr.-Ing. Josef Rötzer (born in 1959) studied civil engineering at the Technical University of Munich and later obtained his PhD at the Bundeswehr University Munich. From 1995 onwards, he worked in the engineering head office of Dyckerhoff & Widmann (DYWIDAG) AG in Munich. His area of responsibility included the detailed design of industrial and power plant structures. The DYWIDAG LNG Technology competence area, focusing on the planning and worldwide construction of liquefied gas tanks, was integrated into STRABAG International in 2005.

Josef Rötzer is a member of the Working Group for Tanks for Cryogenic Liquefied Gases of the German Standards Committee and a member of the committee for the American code ACI 376.

1

Introduction

The use of natural gas as an independent branch of the global energy supply sector began in the early 1960s. Prior to that, natural gas had only been regarded as a by-product of crude oil production; there was no use for it and so it was either pumped back into the ground or flared. But all that has changed in the meantime – natural gas currently accounts for 22% of global energy supplies. Huge deposits in Australia are now being exploited and deposits in the USA will soon be coming online, which will increase that global share (Fig. 1.1). There are many reasons for this development – economic, political and ecological: Australia is close to the growing Asian economies, the USA is aiming to reduce its dependence on foreign oil and energy supplies by developing its own resources, and global efforts to replace fossil fuels by gas apply throughout the world.

The International Maritime Organisation (IMO), a specialised agency of the United Nations, has drawn up new rules that have been valid from 2015 and are particularly strict for the North Sea and Baltic Sea. Complying with emissions requirements is difficult when using diesel and heavy oil as marine fuel. But using liquefied natural gas (LNG) as a marine fuel results in – compared with diesel – about 90% less nitrogen oxide, up to 20% less carbon dioxide and the complete avoidance of sulphur dioxide and fine particles [1]. Det Norske Veritas (DNV), the Norwegian vessel classification body, therefore expects that there will be about 1000 new LNG-powered ships by 2020, which amounts to almost 15% of predicted new vessel orders. This change is heavily influenced by the huge drop in the price of natural gas, which has been brought about by the global production of shale gas (Fig. 1.2, Fig. 1.3).

The use of natural gas involves transport and storage difficulties. Transport via pipelines is economic up to a distance of 4000–5000 km, depending on the boundary conditions. In the case of difficult geographic circumstances, such as supplies to islands, e.g. Japan and Taiwan, or where it is necessary to cross mountain ranges, supplying gas via a pipeline is much more difficult and costly. Therefore, the method of liquefying natural gas and then transporting it over great distances in ships had already become established by the mid-20th century.

LNG technology takes advantage of the physical material behaviour of natural gas, the main constituent of which is methane. At the transition from the gaseous to the liquid state, the volume is reduced to 1/600. However, this requires the temperature of the gas to be lowered to -162°C. Only this extreme reduction

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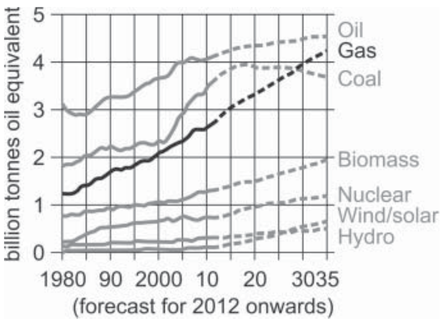

Fig. 1.1 Development of energy demand [1].

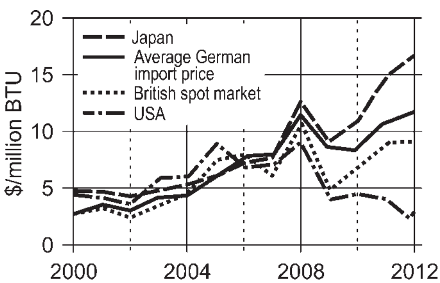

Fig. 1.2 Gas price developments since 2000 [1].

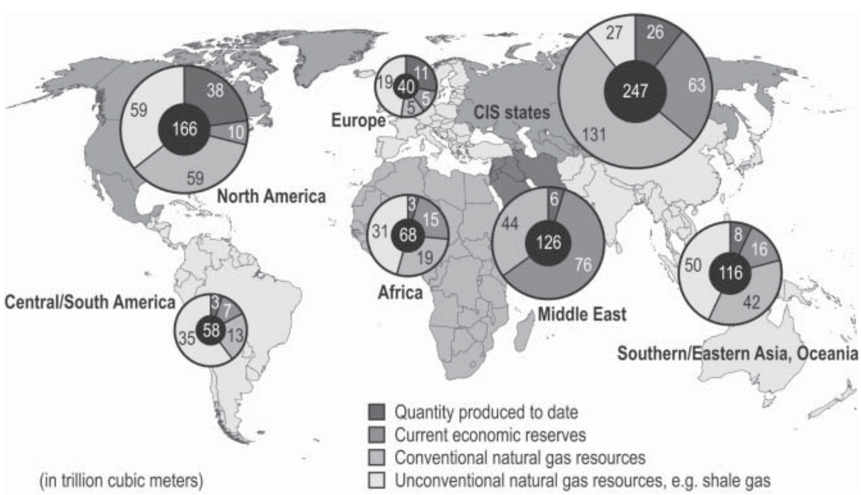

Fig. 1.3 Regional distribution of natural gas potential [1].

in volume makes transport in ships economically viable. The entirety of the elements required for transporting LNG in ships is known as the “LNG chain”, which consists of the liquefaction plant in the country supplying the gas, LNG tanks for intermediate storage of the liquefied gas, jetties as berths for the special LNG transport vessels, tanks for the intermediate storage at the receiving (i.e. import) terminal and a regasification plant in the country importing the gas.

It is common practice these days to build full containment tanks, which consist of an outer concrete secondary container surrounding an inner steel primary container. The prestressed concrete outer container serves to protect the thin-wall steel inner container against external actions and also functions as a backup container in the event of the failure of the primary container. The outer container must prevent uncontrolled leakage of vapours into the environment and must also be able to contain the liquefied gas and withstand any overpressure.

The great hazard potential of LNG is the risk of fire. If LNG changes to its gaseous state and mixes with air, the result is a combustible gas that can explode, and certainly burns very fiercely. Safe transport and storage are the technical challenges of LNG. At these low temperatures, the materials normally used in the construction industry exhibit a distinctly brittle behaviour and fail abruptly. During normal operation, the steel inner container takes on the temperature of the liquefied gas and cools to -165°C. In order to guarantee sufficient ductility at this temperature, the inner container must be made from 9% nickel steel or stainless steel. Thermal insulation about 1 m thick is placed between the steel inner and concrete outer containers.

Between the underside of the steel inner tank and the base slab of the concrete outer tank, the thermal insulation consists of loadbearing cellular glass (often called foam glass). The annular space between the inner and outer containers is filled with perlite, and a layer of elastic material (resilient blanket) is installed to compensate for the horizontal thermal deformation of the inner container. The insulation on the aluminium roof of the inner container is made from glass fibre or perlite. What at first sight seem to be very generous dimensions are necessary in order to keep the boil-off rate below 0.05% by vol. per day. Should the inner container fail, the inside face of the concrete outer container cools to -165°C, and that calls for the use of special reinforcement that can resist such low temperatures. The dynamic design for the seismic load case must take into account the action of the sloshing of the liquid and the interaction with the concrete outer container. The tank must be designed to withstand a so-called operating basis earthquake (OBE), i.e. is not damaged and remains operable, and also for a so-called safe shutdown earthquake (SSE).

Reference

1 Flüchtige Zukunft. Wirtschaftswoche, No. 32, 2012, pp. 58–65.

2

History of Natural Gas Liquefaction

History shows us how the present circumstances have evolved; every new development builds on previous situations. The demand for gas has developed with the demand for energy in general. Technical progress led to the development of the liquefaction of gases, and after this process had been realised for various gases, so it became possible to liquefy natural gas, too. That was followed by the development of storage and transport methods for the liquefied natural gas (LNG), which in turn evolved into a global LNG market. The history of LNG outlined in sections 2.1 to 2.4 below is essentially based on the book by Matthias Heymann: *Engineers, markets and visions – The turbulent history of natural-gas liquefaction* [1].

2.1 Industrialisation and Energy Demand

The process of the industrialisation of the production of energy, iron and steel, which began in England and reached the rest of Europe in the early 19th century, required a transition from wood-fired ovens and waterwheels to coal and oil as the energy sources. The start of the 20th century saw another considerable rise in the demand for oil and gas; oil was used as a fuel for many different means of transport, as a fuel for heating and as a raw material for the petrochemicals industry. The widespread use of natural gas did not come about until pipeline technology had been established, which then led to an increase in gas consumption in the USA during the 1930s and in Europe after 1945.

At first, gas was used for lighting only. The destructive distillation of coal produced gas and coke. This synthetic gas was therefore known as coal gas or, indicating its usage, town gas. It gave off a much brighter light and brought about a considerable change to people's living and working conditions, as they were no longer reliant on daylight alone. The operation of gas lighting was, in many respects, unchartered territory. It called for a complex infrastructure that was linked with high costs, a restriction to just one supplier for a defined area, political approvals and also society's acceptance of this new form of energy. Economic operations required the signing of long-term contracts so that the costly investments could be recouped. Municipal or national bodies were set up in order to prevent monopolies from being abused.

The first gasworks were built in Europe in 1812 (London and Amsterdam) and in the USA in 1816 (Baltimore); the first German gasworks followed in 1826

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(Berlin and Hannover). During the second half of the 19th century, competition for the gasworks appeared in the form of petroleum and electric lighting, to which the gasworks responded by creating new usage options such as heating, cooking and the provision of hot water. As the type of usage shifted from lighting to heating, so very pronounced fluctuations in consumption appeared between summer and winter (which exceeded a factor of five). In the 1920s a new welding method enabled the use of seamless pipes for pipelines, meaning that it was now possible to transport natural gas over greater distances. Pipeline networks were built in the USA which connected the gasfields of Texas and Louisiana with the centres of population in the north-east of the country.

Gas consumption in the former West Germany increased from 2 billion m^3 in 1964 to 16 billion m^3 in 1970. This rise is connected with the changeover (or "conversion") from town gas to natural gas. Matthias Heymann [1] calls this a "complex systemic change", because it involved much more than just changing the type of gas. Instead of small, local gas networks run by the municipalities, there was now a supraregional network with new pipelines that joined the local networks together. These new networks also needed high-pressure pipelines to bring the gas from the supplying countries and intermediate compressor stations to generate the pressure gradient. And last but not least, the appliances of the end consumers had to be converted or renewed. Conversion work in the former West Germany was carried out between 1967 and 1972.

The reasons for changing over to natural gas were its better gross calorific value (roughly twice that of town gas) and its much cleaner combustion with fewer pollutants and less carbon dioxide. During this process of growth and industrialisation, two opposing requirements emerged for operators aiming to guarantee availability: base load and peak load. The base load problem was that consumption was growing faster than new sources of gas could be brought online or pipelines laid. However, this disparity eased over time. The peak load problem arose due to the use of gas primarily for heating and the associated, very distinct, seasonal fluctuations. Suppliers had to expand their existing and create new storage capacities. One option was to liquefy the gas and store it in the form of LNG.

2.2 The Beginnings of Gas Liquefaction

We have to go back a few centuries to find the beginnings of gas liquefaction. By the end of the 18th century it had become possible to convert gases into their liquid state through a combination of pressure and cooling. In the first half of the 19th century, all known gases – with the exception of oxygen, hydrogen, nitrogen, nitrous oxide, carbon monoxide and methane – could be liquefied. Around 1860, the prevailing view was that a gas could only be liquefied when its temperature dropped below a temperature specific to that gas – its boiling point. The liquefaction of oxygen was first achieved in 1877 by Louis Cailletet in France and Raoul Pictet in Switzerland working independently of each other. Cailletet discovered a physical phenomenon of gases which we call expansion. This means that the temperature of a gas subjected to a high pressure drops considerably when its volume is increased and hence the pressure is suddenly reduced. It was already generally known that gases heat up when subjected to high pressure.

If the two methods were now combined, i.e. first pressurising the gas, then waiting until the gas had cooled to the ambient temperature and, in a third step, increasing its volume, the gas could be cooled below the ambient temperature. The cooling achieved is proportional to the pressure applied. Cailletet's method was based on the fact that by controlling the magnitude of the pressure, it was possible to achieve the cooling required for the particular type of gas. Using this method it was possible to liquefy small amounts of oxygen at -183°C and nitrogen at -196°C. Pictet's method was based on the same physical principles. His idea was to arrange the cooling processes in series, as a cascade. In doing so, he made use of the different boiling points of different gases. In the first stage, a combination of pressure, cooling and expansion was used to liquefy sulphur dioxide. This liquid sulphur dioxide was then used as a coolant for carbon dioxide, which was subsequently expanded and hence liquefied. In the following cascade stage, the carbon dioxide was used as a coolant to liquefy oxygen. Although Pictet's method required different coolants, it worked with a lower pressure. Over the coming years, no further methods were developed, instead industrial usage and applications were improved. The precursors to Linde AG and Air Liquide were founded.

Natural gas, the main constituent of which is methane, was first liquefied by Godfrey Cabot in the USA in 1915. However, natural gas consists of other constituents apart from methane which liquefy or solidify at temperatures much higher than the boiling point of methane (-162°C). Therefore, natural gas liquefaction plants require various stages to purify the gas by removing these constituents, which would otherwise impair the liquefaction process and clog the plant. It was many years before natural gas liquefaction could be operated on an industrial scale.

In 1937 H. C. Cooper, president of the Hope Natural Gas Company, initiated studies of the liquefaction, storage and regasification of natural gas. A small pilot plant was built in Cornwell, West Virginia, to test the method. A cascade process was chosen, with water, ammonium and ethylene as the coolants. Trial operations began in early 1940 and continued uninterrupted for four months without any problems. At the same time, north-eastern USA experienced a very cold winter, which presented many suppliers with difficulties in trying to cover the peak load. Therefore, the East Ohio Gas Company, a subsidiary of Hope Natural Gas, decided to build a natural gas liquefaction plant, storage tanks and a regasification plant in Cleveland, Ohio. Three double-wall spherical tanks, with cork as insulation, were built to store the gas; each tank was 17 m in diameter and thus had a capacity of 2500 m^3. The Cleveland plant had a total capacity of 41 million m^3 of natural gas and was therefore the first large natural gas liquefaction plant in the world; it went into operation at the start of February 1941. At times of low gas demand, LNG was produced and stored, and when demand increased, the LNG was regasified and fed into the network. No malfunctions occurred during the first year of operation and so it was decided to increase the total capacity by building a further tank. The new tank No. 4 was planned with a capacity of 4500 m^3, which would increase the capacity of the plant by 80%. A spherical tank was seen as unsuitable for a tank of this size, and so a 23 m dia. x 12 m

Fig. 2.1 The scene of the Cleveland accident with tanks 1 and 2 still intact.

high double-wall, cylindrical, flat-bottom tank was designed. Like the spherical tanks, the inner container of this new tank was made from 3.5% nickel steel, which exhibited better material properties at low temperatures.

On Friday, 20 October 1944, roughly one year after being commissioned, a terrible accident took place which was caused by the failure of the new, cylindrical tank. How this catastrophe actually came about was never able to be fully resolved because the area affected was so large that all eye-witnesses were killed (Fig. 2.1). The reconstruction of the disaster resulted in the following supposed order of events: It started with the failure of tank No. 4, from which 2400 m^3 of LNG leaked out, vaporised and floated over the sloping ground in the form of a white cloud 4 m deep. This mixture of gas and air ignited and a fire broke out. Some 20 minutes after the fire started, the neighbouring spherical tank No. 3 failed and a further 2500 m^3 of LNG escaped. The ensuing fire reached a height of 800 m. A maximum temperature of 1650°C was calculated based on the molten materials found. Not until the next day could the fire be brought under control and most of it extinguished so that investigators could get an idea of what had happened. The damage was spread over an area with a radius of about 400 m from tank No. 4, and everything within a radius of 200 m had been incinerated. The two remaining spherical tanks were still in operation, but smoke was rising from them as the insulation had ignited. Solid carbon dioxide was used to extinguish these fires.

The question as to what caused tank No. 4 to fail in the first place was never able to be fully and unequivocally answered. The investigations revealed that prior to the failure of the tank, patches of frost had been noticed on the outer surface. Frost appears when either the insulation is not functioning properly, and the outside surface is affected by the cold liquid in the tank, or when defects in the inner container allow LNG to leak into the space between the inner and outer containers. Patches of frost are therefore a warning, which should be taken very seriously and investigated immediately to discover the causes. Studies undertaken to identify the cause of the disaster also looked at the behaviour of the building materials used and led to the realisation that the 3.5% nickel steel used should be classed as unsuitable for LNG tanks. The terrible accident at Cleveland had such momentous repercussions that the topic of LNG to cover the peak load was not taken up again during the following decade.

2.3 The First Steps Towards Transport in Ships

William Wood Prince was the chairman of Union Stockyards, abattoirs in Chicago, at this time the centre of the USA's meat processing industry. In the early 1950s he had a very bold idea, which, if successful, would be very promising. The meat industry required a great deal of energy to cool its cold stores. The cheapest energy form at that time was natural gas, huge deposits of which were available in the south of the country. Furthermore, a network of canals, the Illinois Waterway, had connected Chicago to the Mississippi, and hence the Gulf of Mexico, since 1848, and was used to transport bulk goods. Prince asked himself the question of whether it would be possible to transport LNG to Chicago in ships via the Mississippi. The missing piece in his jigsaw was the ships themselves. In 1952 he appointed Willard S. Morrison, an engineer and refrigeration specialist, to carry out studies to find out which materials would be the most suitable for tanks and insulation at a temperature of -165°C.

There are two basic ways of building a ship's tank to cope with low temperatures: with a low temperature-resistant tank material or with a normal tank material. In the case of the former, the tank material is in direct contact with the LNG, and the insulation is attached to the outside of the tank. The main difficulties result from the combination of material and low temperature. The material must therefore be suitable and exhibit sufficient toughness at low temperatures. As the temperature drops, so the tank material contracts, which leads to problems where the tank is fixed to the structure of the ship.

When using the other option, i.e. normal tank material, the insulation is attached to the inside of the tank, so the tank material is not in direct contact with the LNG. It is also easier to fix the tank to the ship's hull because only minimal contraction takes place. However, the insulation now plays the main role. Failure of the insulation not only impairs the serviceability of the tank, but also its structural integrity, and in the extreme case leads to failure, because neither the tank nor the structure of the ship are suitable or designed for such low temperatures. Nevertheless, Morrison decided to opt for this method with insulation on the inside and, following preliminary trials, selected balsa wood as the insulating material.

Following successful tests of the materials, five vertical, cylindrical tanks were installed on a barge and lined with balsa wood at Pascagoula, Mississippi. Before being granted an operating licence, the American Bureau of Shipping, as the approving body, called for tests to check the system's fitness for purpose. To do this, two of the tanks were filled with LNG and left for two months. During this trial, patches of frost appeared on the outside of the tank. The tanks were emptied and examined. The innermost layers of the balsa wood exhibited considerable

Fig. 2.2 The *Methane Pioneer* after its conversion.

wear and damage. It seemed that the project was doomed to fail. But alongside this work, Morrison had also appointed the J. J. Henry Company, consultant naval architects, to conduct a feasibility study and draw up designs for seagoing vessels. Experienced companies were appointed to improve the balsa wood insulation and prepare working drawings for the tanks.

These activities were generally known throughout the gas industry and aroused interest and curiosity. Therefore, the state gas supply body in the UK, the British Gas Council, decided to send an employee to the USA, get in touch with those carrying out this work and gather information. That marked the start of close cooperation between Union Stockyards and the British Gas Council. Further investors were sought and one was found – the Continental Oil Company. The year 1956 saw the founding of the Constock Liquid Methane Corporation, in which Continental Oil had a 60% stake, Union Stockyards 40%. Working together with the British Gas Council, the aim was to build an LNG tanker and test LNG storage tanks. Constock was in charge of the engineering. A decision was taken to convert an existing ship (Fig. 2.2) in order to save time and money and thus concentrate on the main aspect, which was to develop a low temperature-resistant LNG tank. Would it be better to place low temperature-resistant insulation on the inside or to build the tank from a low temperature-resistant material, and which material should be chosen?

In the end it was decided to build the tank from low temperature-resistant material. The thermal contraction of the tank would have to be compensated for by movement joints in the structure. The clear advantage of this was that it protected the insulation against direct contact with the LNG. Stainless steel, aluminium and 9% nickel steel were the options considered for the tank material. Although stainless steel exhibited the necessary properties for such temperatures, it was very expensive. Aluminium presented welding difficulties and 9% nickel steel had already been used previously for a liquid oxygen tank.

There were no reliable principles on which to base the choice. Therefore, Constock decided to carry out series of tests on weld seams between components made from aluminium and 9% nickel steel in liquid air subjected to impact and bending loads. Both materials fared well in the tests. Gamble Brothers, a specialist firm from Louisville, Kentucky, carried out further development work on the balsa wood insulation, which in this design was not in direct contact with the LNG. Arthur D. Little from Cambridge, Massachusetts, produced the design for the tank. He built a test tank with a capacity of 75 m^3, which was then filled with liquid nitrogen and tested under the most diverse scenarios. The next step involved converting the freighter *Nomarti* and equipping it with five tanks with a total storage capacity of 2000 m^3. Both the significance and the risks of this work were well known, and so great care was taken to achieve a very high quality. The balsa wood insulation was installed in an air-conditioned workshop with a low humidity. All weld seams were tested under pressure and inspected using x-rays.

It was then time to carry out test voyages in the Gulf of Mexico with a filling of LNG; afterwards, the tanks were emptied and examined. All those involved expressed great concern regarding the weld seams, because welding of the aluminium tanks gave rise to small pits. No one had any knowledge about whether this pitting, at the low temperatures, with full tanks and under wave action, might lead to cracking or failure. Despite the many concerns and warnings, an Atlantic crossing was planned. On 25 July 1959 the *Methane Pioneer*, fully laden with LNG, embarked on its voyage across the Atlantic to the UK. Five further crossings followed, which enabled many measurements to be taken and broadened the knowledge base considerably. The main thing, however, was to demonstrate the feasibility of transporting LNG by ship. The LNG chain had therefore been closed and there was nothing more standing in the way of industrial-scale operations.

2.4 Algeria Becomes the First Exporter

The developments in the USA spread to other countries. In the mid-1950s several European countries intensified their research into LNG. Shell regarded this research work as so important that both of its head offices, in London and The Hague, worked on the project at the same time; the significance and perspectives were rated so high that both countries wanted to be involved. In France research work was controlled by the government in Paris. The various activities were given a clear objective and speeded up as it became clear that Algeria was ready to conclude long-term supply contracts with European countries. Those contracts were signed by Algeria, France and the UK in 1962. Whereas all the previous projects had been merely feasibility studies or concerned smaller plants, the new contracts meant that a truly industrial scale was now involved.

For the first time, an LNG chain was necessary, i.e. the whole series of components and plant elements required for the pumping, liquefaction, storage, transportation and regasification of natural gas:

- natural gas production on land or at sea,
- pipelines to the export terminal,
- an export terminal with gas purification and liquefaction,
- tanks for intermediate storage,
- jetties for berthing the ships,

– LNG tankers,
– a receiving terminal, with jetties, tanks and regasification plant, for importing and storing the LNG, and
– connections to gas networks or consumers.

France's state gas supply corporation, Gaz de France, had already investigated the possibility of gas imports from Algeria as early as 1954. Work on the newly created research establishment at Nantes – a test facility with an LNG tank capacity of just 500 m^3 – began in 1960. The intention was to investigate all the materials, processes and methods that would be needed to develop a transport chain. The tests carried out covered the same ground as those conducted in the USA.

Constock was not prepared to grant licences or build and sell LNG tankers, and so France was forced to carry out its own development work regarding the design of such vessels. In France they opted for a joint, coordinated procedure in which all interested parties could take part and enjoy a full exchange of the knowledge gained. A freighter, *Beauvais*, was converted and fitted with three different tanks developed by different manufacturers. The test voyages finally took place in the spring and summer of 1962.

Conch Methane International, in which Shell held a 60% stake and Constock 40%, was founded to undertake and coordinate the work on the British side. After the Algerian contracts had been signed, the UK started building two ships, the *Methane Princess* and the *Methane Progress*; shortly afterwards, the French began building the *Jules Verne*. The British ships had aluminium tanks with balsa wood insulation, the French ship a 9% nickel steel tank with synthetic insulation. The lengths of the ships ranged from 190 to 200 m and all three were 25 m wide. Receiving terminals to import the LNG were built on Canvey Island in the Thames estuary (Fig. 2.3) and in Le Havre at the mouth of the Seine. In Algeria, pipelines were laid from the Hassi R'Mel gasfield in the northern Sahara to the port of Arzew, where a liquefaction plant, storage tanks and other necessary port facilities were built. The first liquefaction plant had a capacity of 7000 m^3 LNG per day, i.e. 2.5 million m^3 LNG annually, which was equivalent to 25 times the volume of the Constock plant at Lake Charles, Louisiana, or 40 times that of the Cleveland plant. The first LNG from Algeria arrived in the UK in October 1964, in France in April 1965.

2.5 Further Development with Peak-Shaving Plants

The next step in the ongoing development and spread of the LNG industry began in the early 1960s. Natural gas production was growing by 10% every year. Consumption, however, exhibited very large seasonal fluctuations. In 1968 the Boston Gas Company calculated that the relation between peak load and minimum daily consumption had risen from a factor of three to a factor of six over the previous decade and forecast a further rise to a factor of nine within the next three years. These marked peak load problems could not be solved simply by building additional pipelines, instead required additional gas storage capacity. Initially, the obvious solution was to use existing caverns and depleted

Fig. 2.3 Canvey Island receiving terminal.

gas deposits for intermediate storage. Once these options had been exhausted, the use of so-called peak-shaving plants became the preferred method of storing LNG. Such facilities consist of a natural gas liquefaction plant, one or more storage tanks and a regasification plant.

The first four peak-shaving plants were completed in the USA in 1965. That was followed by a distinctive growth phase, which resulted in 61 peak-shaving plants in operation in the USA and Canada by 1978; Germany had 10 by that time, the first of which had been built in Stuttgart in 1971. No further plants were built after 1978. The reasons for this were a decline in gas consumption, improvements to the supply situation and also technical problems at a few plants. It was the process engineering that presented difficulties. Natural gas consists mainly of methane, but contains traces of many other gases. As each of these gases liquefies at a different temperature, the process engineering must be exactly tuned to the respective gas composition.

Many peak-shaving installations were built near to consumers, in some cases in city centres (e.g. Boston, New York, Portland, San Diego, Stuttgart). Such locations led to stricter safety stipulations. An accident like that in Cleveland in 1944 had to be avoided at all costs.

2.6 The First German LNG Tank in Stuttgart

The tank in Stuttgart had a capacity of 30 000 m^3 and the LNG was stored in an inner container made from 9% nickel steel (Fig. 2.4). The prestressed concrete outer container was designed and built by DYWIDAG. Prior to commissioning, the inner container was tested by filling it with water to the intended liquid level. Based on the ratio between the densities of the two materials, the test load was higher by a factor of two. The concrete outer container was designed to withstand the loads due to a leaking inner container. In addition, the tank was surrounded by an 18 m high earth embankment for safety.

Fig. 2.4 Peak-shaving plant in Stuttgart, Germany.

2.7 Wilhelmshaven – the Attempt to Establish a German Receiving Terminal

Planning work for a German LNG receiving terminal at Jadebusen near Wilhelmshaven began in the 1980s. The owner was the Deutsche-Flüssigerdgas-Terminal-Gesellschaft (dftg), the majority shareholder of which was Ruhrgas AG (later E.ON). Their plans included three storage tanks each with a capacity of 80 000 m^3 of LNG (Fig. 2.5), and the terminal was designed for an LNG intake capacity of 12 000 m^3/h and a natural gas regasification capacity of 1.2 million m^3/h. During the 1980s the design and construction of large LNG tanks were still undergoing development. The tank system chosen had an inner container open at the top and a closed prestressed concrete outer container that was protected against direct LNG contact by a layer of polyurethane foam that extended across the base slab and over the full height of the wall.

The inner container was 62 m in diameter and 28 m high, the outer container 66 m in diameter and 41 m high overall. The system was designed for an operating pressure of 200 mbar, with the safety valves being actuated at 300

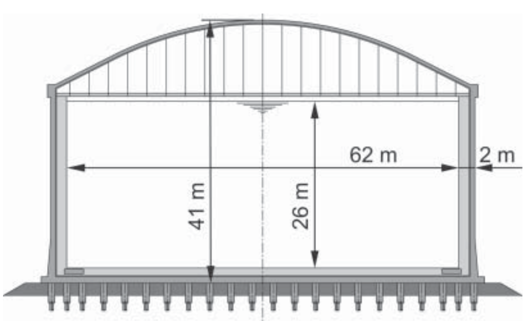

Fig. 2.5 Section through the 80 000 m^3 LNG tank in Wilhelmshaven, Germany.

mbar. The ground conditions called for piles. Linde was responsible for the process engineering, Noell for the steel inner container and DYWIDAG for the concrete outer container. At that time there were no German nor international codes and specifications available which stipulated the requirements to be met by the construction materials used, so many experts were appointed in the course of the approval procedure and many tests were conducted on the materials. A special testing regime was prescribed for the materials in order to verify their suitability at cryogenic temperatures.

A comparison between the technical documents submitted for the building approval back then and the calculations and specifications sometimes required these days in many countries leaves this author with the impression that thinking in line with engineering principles and a sense of responsibility were more pronounced in those days. In order to achieve a high level of safety, studies were undertaken to assess hypothetical actions on the tank system such as "complete failure of the liner system and simultaneous flooding of the annular space with LNG". Owing to a lack of experience with the storage of LNG in large tanks, there were many more concerns and misgivings regarding potential incidents than is the case today. In many respects these were uncharted waters and nobody wanted to take any risks. Safety and protection were the number one priorities.

Over the following decades, the LNG market expanded in stages; either the capacity of an export terminal or a receiving terminal was increased or a new country came online as an exporter or importer.

In this author's opinion, there were three further developments that resulted in significant changes: the extensive use of coal seam gas by means of LNG for exports, the use of shale gas, primarily promoted by the USA, and the establishment of Emission Control Areas (ECAs) in the North Sea and Baltic Sea. These points will be considered in detail below.

2.8 The Liquefaction of Gas in Australia

The information and quantities mentioned in this section have been taken from the Australian Energy Resource Assessment compiled by the Australian Bureau for Agricultural and Resource Economics (ABARE) [2] and the market analysis carried out by EnergyQuest [3]. The conversion of energy quantities in the deposits [petajoule, PJ], gas volumes [tcf], LNG volumes [m^3] and liquefaction capacities [Mtpa] are based on an averaged gas composition. The conversion factors used are listed in Table 2.1.

Extent, climate, location and geography have determined the boundary conditions for the creation of Australia's coal, oil and gas reserves. Extensive inland coal deposits have formed in Queensland and New South Wales, whereas large subsea deposits with conventional natural gas have developed off the west coast in the Carnarvon, Browse and Bonaparte basins. These three basins contain 92% of Australia's conventional gas reserves, which can be divided into so-called economic demonstrated resources (EDR) and subeconomic demonstrated resources (SDR).

Table 2.1 Conversion of units of measurement for energy.

Trillion cubic feet	[tef]	1 tef	=	1.05 EJ	exa	= $1E^{18}$
Million tonnes per annum	[Mtpa]	1 Mtpa	=	54.8 PJ	peta	= $1E^{15}$
British thermal unit	[BTU]	1 BTU	=	1.054 kJ		

Table 2.2 LNG export terminals in Australia in operation as of 2012.

	In operation since	Liquefaction [Mtpa]	Storage	
			Tanks	[m^3]
North West Shelf	1989	5.0	4	260 000
	1992	2.5	1	130 000
	2004	4.4	1	130 000
	2008	4.4	1	130 000
Darwin	2006	3.5	1	188 000
Pluto	2012	4.8	2	240 000

In order to exploit these reserves, the first export terminal was built in Western Australia in the 1980s. This has since been extended and now has five lines (so-called trains) with a capacity of 16.3 Mtpa LNG (2008 figure). Exports from Australia's second LNG terminal at Darwin on the north coast began in 2006 (see Table 2.2). These two facilities have allowed the LNG export capacity to reach the same order of magnitude as the domestic demand, which is 19.5 Mtpa and corresponds to an energy quantity of 1100 PJ. As a comparison, in 2012 gas consumption in Germany was 84.4 billion m^3 of gas, or 3.21 PJ [4], i.e. less than 0.3% of the Australian figure.

Coal seam gas (CSG) has also been produced in Queensland since the mid-1980s, but only used as a low-price supplement to conventional gas for local consumption. Since then, several national and international companies have carried out a number of different projects with more or less success. Only after the Australians realised the potential of CSG, and it was shown that Queensland could supply much more CSG than was needed locally, did they really start to search for other usage options. This attracted the attention of international oil and gas companies who were looking for gas reserves in the Asia-Pacific region to supply the populations in that part of the world. Those companies were also familiar with the characteristics of unconventional gas, the large-scale use of which began in the USA. As a result, several international companies are participating in CSG LNG projects (see Table 2.3 and Fig. 2.6). The gas market on the east coast is therefore tracking the development on the west coast and is gradually aligning itself with global demand.

Whereas conventional natural gas, shale gas and coal seam gas are identical in terms of their transport and usage, they differ considerably when it comes to reserves and the geology of the deposits. To illustrate this, the characteristics of the various gas reserves will be briefly outlined.

Table 2.3 CSG LNG export terminals planned as of 2012.

			Storage	
	Planned start	**Liquefaction [Mtpa]**	**Number of tanks**	**Capacity [m^3]**
Queensland Curtis	2014	8.5	2	280 000
Gladstone LNG	2015	7.8	2	280 000
Australia Pacific	2015	7.5	2	320 000

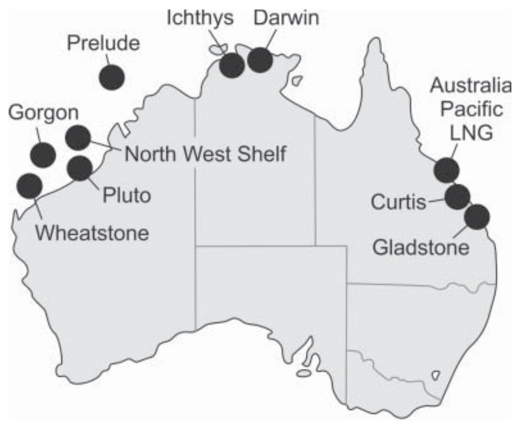

Fig. 2.6 Australian LNG projects.

The natural gas deposits that are easiest to exploit, and therefore the most common type, are the so-called conventional gas reserves. These are deposits of natural gas contained in porous and permeable rock strata found below denser, even impermeable, rock strata. In these situations the gas has risen from greater depths but become trapped in the permeable rock strata as it is prevented from rising further by the impermeable strata above. The prerequisite for the formation of such deposits is geological formations that prevent the gas from escaping laterally and bypassing the overlying rocks. Such formations are known as natural gas traps and ensue as a result of sedimentation processes or tectonic events (see Fig. 2.7).

Owing to its lower density, natural gas is frequently found in the highest regions of crude oil deposits. Natural gas can rise (migrate) into higher rock strata more easily than crude oil, which means that deposits containing natural gas only are therefore very common. Where natural gas is found in deposits together with oil, this gas is known as conventional, associated natural gas; where it is found alone, it is known as conventional, non-associated natural gas. Deposits are known as unconventional when the natural gas is not held in natural gas traps, instead is trapped in shale and argillite formations or in sandstone and limestone, also gas in coal seams. The gas trapped in porous sandstone and limestone formations is known as tight gas. Such strata are generally more than 3000 m below the surface. The viability of a sandstone reservoir is determined by its porosity, i.e. the empty

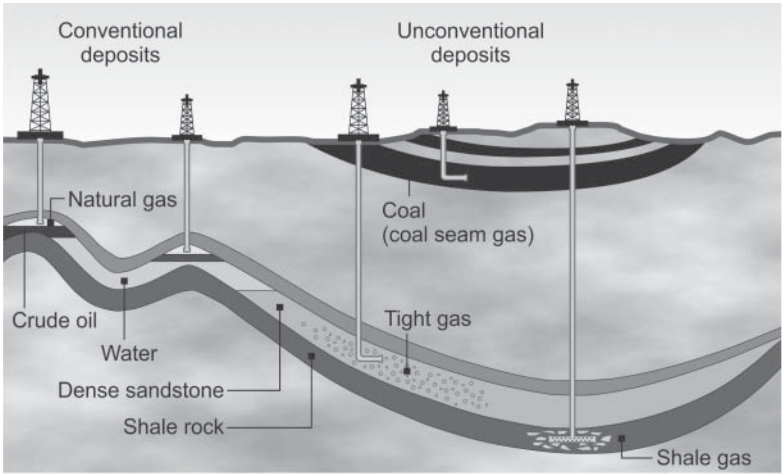

Fig. 2.7 Gas deposits.

spaces between the grains, and its permeability, i.e. how easy it is for the gas to move through the rock [5].

The term shale gas comes from the colloquial use of the word shale for argillite in the English language. In geological circles, on the other hand, shale is used as a collective term for metamorphic rocks (which, as a rule, do not contain gas) and not for sedimentary rocks such as argillite. Shale gas is the name given to natural gas trapped in argillite. It ensues from the organic substances contained in the rock strata which have turned into methane over time. Generally, shale and argillite strata do not exhibit the permeability needed to generate an adequate flow of gas when using a vertical well as is used for conventional sources of gas. The gas is trapped in fissures and joints, contained in pores or bonded to organic constituents in the argillite. In order to release it, wells are drilled into the argillite strata and cracks and fissures created in the rock by applying very high pressure. This method is not new; it has been employed in the USA since 1949 and in Germany since 1961.

Originally, vertical wells were drilled into the gas-bearing strata. However, the yields obtained with this method were mostly low and only a few deposits were economically viable. Drilling methods underwent significant developments around the turn of the 21st century and this considerably increased the potential applications and quantities of gas that could be produced economically. The improvements involve a combination of horizontal well completion and hydraulic cracking in the shale gas strata, so-called hydraulic fracturing, or fracking for short.

Horizontal completion takes place at depths of 1000–4500 m, in the middle of the argillite strata and hence often 1 km or more below the groundwater-bearing strata. The network of horizontal wells not only avoids the work involved with and cost of many metres of unproductive wells, but also considerably reduces the intervention in the rock strata above the gas-bearing strata. In the next step, the shale is split apart by hydraulic pressure and a special fracking fluid is forced into the ensuing cracks.

The fluid consists of approx. 90% water, approx. 9% sand, quartz or ceramic particles and 0.5–2% chemicals that prevent bacteria growth and help to reduce the friction during injection. The solid constituents (sand, quartz or ceramic particles) are included to keep the cracks open once the bursting pressure has dropped so that the gas can escape.

For the environment, the disadvantages and possible consequences of fracking are [6]:

– a huge consumption of freshwater, because in order to burst open the argillite, five or six times more water is required than is the case for tight gas in sandstone,
– the use of up to 20 different chemical additives, some of them toxic,
– the treatment of the return flow out of the well, which besides the chemicals introduced, can also include substances, heavy metals and benzenes that have been dissolved out of the soil, and
– the risks to the groundwater reservoirs that are drilled through.

In order to counteract the risks, the current state of the art includes installing a multilayer casing of steel pipes and cement injection. The aim of this is to guarantee the integrity of the well and the casing as well as the surrounding rock formation and also create an impermeable barrier between the well and the groundwater zones. The Austrian company OMV has for many years been working on a method to replace the additives by safe, biodegradable constituents such as cornflour. ExxonMobile has developed and laboratory-tested a fracking mixture whose only additives are choline chloride and DEG monobutyl ether; the former is used in animal feed, the latter in household cleaners and paints. The German company TouGas is developing a gel that allows the use of saltwater instead of drinking water. This would enable the water to be reused and reduce the water consumption significantly. These developments promise considerable improvements, but field tests are necessary to make progress with these methods.

Natural gas is also found in coal seams. Considerable quantities of methane are adsorbed on the large specific surface area of coal. As pressure increases with depth, the coal at such depths can also hold more natural gas. Like conventional natural gas, coal seam gas (CSG) is composed mainly of methane, with traces of carbon dioxide and nitrogen. CSG has biogenic or thermogenic origins. Biogenic methane is generated by bacteria from the organic substances present in the coal. Thermogenic methane, on the other hand, forms when organic material inside the coal is converted into methane through the application of heat and pressure. Biogenic methane is found down to a depth of 1 km, thermogenic methane at greater depths [3].

The natural joints and fissures in coal seams create a large surface area on which larger quantities of gas can accumulate than is the case with conventional sandstone reservoirs. For example, 1 m^3 of coal can hold six or seven times more natural gas than 1 m^3 of conventional deposits. But the production of CSG is more involved, more costly than the production of conventional gases. At conventional gas deposits, the wells can be closed off and re-opened again without the need to take any additional measures. At CSG deposits, on the other hand, in order to pump off the gas, drainage and dewatering measures must be carried out prior to recommencing gas production. Furthermore, CSG requires considerably more production wells than is the case for a comparable quantity of conventional gas, albeit with much lower costs per well.

One prime reason behind the decision to proceed with three large CSG liquefaction projects on Curtis Island more or less simultaneously was a better understanding of the local conditions, which helped the teams involved to identify the resources available and understand how they could exploit them to best effect [3]. Their work was based on a constant increase in the amount of geological data obtained from the production fields and a large number of wells – 600 new ones in 2008 alone. Estimates of gas price developments play a significant role. The quantity of gas reserves is sensitive to changes in future gas prices. As the price of gas rises, so we also see an increase in the quantity of the reserves that can be developed economically for that price. Moreover, the assessment of the economic demonstrated resources (EDR) is based on the assumption that much higher gas prices can be realised in the future by exporting LNG.

One major difference between conventional and CSG LNG projects is the use of the gas in the start-up phase. LNG projects require a considerable annual volume of gas amounting to about 200 PJ per train. In the case of a conventional LNG project, between six and eight wells are sufficient to supply this quantity. The wells can be drilled and subsequently closed off until the liquefaction plant goes into operation. However, some 500 to 700 wells are needed to produce the same quantity of CSG! The reasons for this are the limited catchment area of the wells and the much lower flow rates per well. These wells take a number of years to drill and bring online. During the initial phase, water has to be pumped off first before gas production can begin; and once CSG production has started, it is generally difficult to interrupt it or close it down without later having to go through the whole start-up process once again. The upshot of all this is that considerable quantities of CSG had to be produced in advance of the Queensland CSG projects going into operation. By the time the liquefaction plant was commissioned, large quantities of gas had been sold at relatively low prices for private consumption and for generating electricity.

Despite the aforementioned difficult boundary conditions, the preliminary studies and the front end engineering design (FEED), i.e. preliminary structural design and sizing of components, were carried out for four CSG liquefaction projects. Construction of three facilities – Queensland Curtis, Gladstone and Australia Pacific on Curtis Island north of Gladstone on Australia's east coast – began in 2009 and 2010. Costs for qualified personnel and materials, but also for accommodation and general living expenses, rose significantly during the course of the work. Thereupon, several companies postponed their CSG projects in Australia indefinitely. However, many conventional LNG projects remain at various stages of development [7, 8], see Table 2.4.

Other CSG projects are expected to follow in other countries. Some 10% of the natural gas in the USA is obtained from coal seams – about 40 billion m^3 in 2002; that required about 11 000 wells to be drilled! Reserves of natural gas in coal seams worldwide are estimated at 100 000–200 000 billion m^3.

Table 2.4 LNG export terminals planned in Australia.

	Planned start	Liquefaction [Mtpa]	Number of storage tanks
Bonaparte FLNG	2016	2.0	1
Browse LNG	2016	10.0	2
Gorgon LNG	2015	20.0	4
Ichthys LNG	2016	8.4	2
Prelude FLNG	2016	3.5	1
Sunrise LNG	2017	3.5	1
Tassie Shoal	2017+	3.0	1
Wheatstone LNG	2016	9.0	2

2.9 Pollutant Emissions Limits in the EU

Air pollutants emitted by ships do not just remain in the skies above the world's seas and oceans, instead are carried over great distances and thus contribute to air pollution everywhere. In its *Thematic Strategy on Air Pollution* [9] dating from 2005, the EU came to the conclusion that by 2020, within the EU, sulphur emissions from ships will exceed those generated on land. For this reason, further measures to protect human health and the environment were seen as necessary and were instigated.

The first step was to revise directive 1999/32/EC [10], which regulates sulphur emissions from ships by limiting the maximum sulphur content of marine fuels. In the subsequent directive 2005/33/EC, the Baltic Sea, North Sea and English Channel were declared sulphur emission control areas (SECAs) where considerably stricter emissions requirements apply than is the case for all other seas and oceans around the world (see Fig. 2.8). These stipulations also apply to passenger vessels operating regular scheduled services outside the SECAs. Even as the directive was being passed, the ensuing reductions in emissions were seen

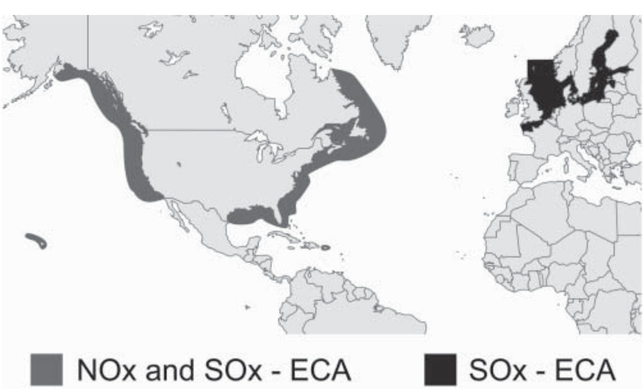

Fig. 2.8 Emission control areas (ECAs).

by many to be insufficient. Owing to the international nature of the shipping branch, all environmental, protection and safety standards are drawn up by the International Maritime Organisation (IMO), a specialised agency of the United Nations. Besides making ships and sea travel safer, another of the IMO's tasks is to prevent shipping operations polluting our seas and oceans, or at least reduce that pollution.

One important regulation produced by the IMO is the *International Convention for the Prevention of Marine Pollution from Ships* (MARPOL). The convention as such contains only general wording; the more precise provisions and figures are laid out in six annexes, with annex VI covering air pollution.

The updated directive 1999/32/EC implements the provisions of MARPOL annex VI. The European Commission demanded additional measures to reduce emissions yet further. To this end, an amended annex VI was adopted in October 2008, which further reduces the maximum sulphur content of marine fuels inside and outside the SECAs.

The European Parliament and Council of the European Union asked the European Commission to monitor the implementation of the directive, produce reports and, if necessary, to tighten the rules. That resulted in directive 2012/33/EU, which was published in the *Official Journal of the European Union* on 17 November 2012. Member States had to bring their legislation into line with this by 18 June 2014. The stricter sulphur directive came into force on 1 January 2015 (Fig. 2.9).

The emissions limits valid from 1 January 2015 and the worldwide changes in the price of LNG have created the boundary conditions that have generated additional demand for LNG on a totally new scale. Years before the directive came into force in 2015, it seemed certain that LNG would replace a large proportion of the heavy oil that is used as marine fuel. The question was not whether LNG would be used, but rather, how much would be needed [11].

As this development gets underway, so our prime concern should be safety. The large, international oil and gas companies that have been active in the LNG sector for decades, and have invested huge sums in their projects, require and fulfil the existing, high safety standards. However, the new, small tanks and terminals call for much lower levels of investment. It is obvious that these developments will

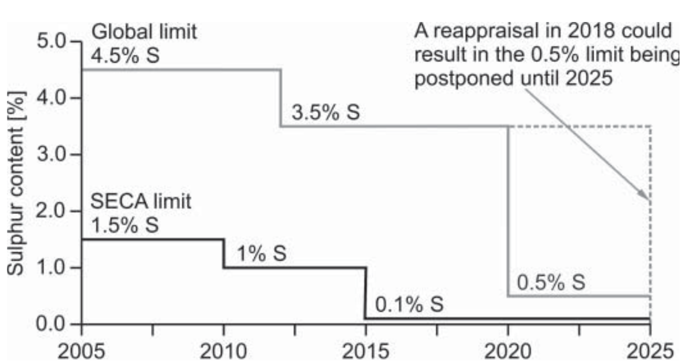

Fig. 2.9 Development of the maximum permissible sulphur content.

attract investors from outside the industry who are less aware of the risks and who are accustomed to lower levels of safety. Some newly erected tanks have developed faults that can be attributed to poor quality in design and construction.

Therefore, we can only hope that all those involved, from the approving bodies to the operators, will work together responsibly and reach the level of safety achieved in the past.

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3

Regulations and their Scope of Applicability

The codes, standards and other regulations for liquefied natural gas (LNG) plants and the parts thereof are divided into the same three groups worldwide: The first group contains regulations for such plants as a whole and focuses on hazards, safety aspects and approval requirements. The second group comprises regulations for the design of individual LNG plants and components of such plants, divided up according to outer container, inner container or insulation. The third group contains the detailed design codes for the individual parts of the plants or structures. This chapter will only look at the first two groups, as the detailed design codes are normally the generally applicable design codes of the particular country in which the facility is located, e.g. EN 1992-1-1 or ACI 318. To gain a better understanding of current standards and regulations, their historical development is first explored before describing the relevant content of the design and configuration documents currently valid.

3.1 History of the Regulations

The American Petroleum Institute was founded in 1919 and began publishing the series of specifications API 12A to API 12G in 1928, which dealt with riveted, bolted and welded tanks for the storage of oil. API 12C (welded tanks) led to the drawing up of one of the most influential standards for steel tanks – API 620 *Design and Construction of Large, Welded, Low-pressure Storage Tanks* [1]. This publication regulates the design and configuration of welded flat-bottom tanks for the storage of liquids at ambient temperature and pressures up to 1 bar. It includes two important appendices: Appendix Q [2] and Appendix R [3]. Appendix Q covers design procedures for the storage of liquefied ethane, ethylene and methane at temperatures down to -165°C and max. 1 bar pressure. Appendix R deals with the design of tanks for the storage of refrigerated products at temperatures down to −50°C. Throughout the world, most of the inner containers for LNG storage tanks are still designed and constructed according to this publication.

British Standards (BS) followed the American lead and employed the same classification according to liquefied gases and temperature ranges. The first two standards were BS 4741 [4] for temperatures down to −50°C, which was published in 1971, and BS 5387 [5] for temperatures down to −196°C, which followed

Design and Construction of LNG Storage Tanks, First Edition. Josef Rötzer.

in 1976. The requirements in those standards corresponded to the state of the art of that period – a tank with just one wall, a so-called single containment tank.

In the next step the Engineering Equipment and Material Users Association (EEMUA) drew up its *Recommendations for the Design and Construction of Refrigerated Liquefied Gas Storage Tanks* [6], which appeared as publication No. 147 in 1986. The classification as single, double und full containment tanks was used for the first time in this document, something that was taken up by all subsequent standards and specifications. Prompted by the failure of a tank in Qatar, it was decided to raise the EEMUA regulations to a more binding, higher legal level and publish them as BS 7777, parts 1 to 4, [12–15]. The provisions of BS 7777 (since withdrawn) included references to other British Standards and was harmonised with those. Concrete structures were designed according to BS 8110. As European standards appeared, many of the provisions of BS 7777 were incorporated in EN 14620, which was published in 2006. Links to and alignment with other regulations were lost to some extent. Therefore, EN 14620 has a number of gaps, such as the definition of partial safety factors and load case superpositions, which then have to be specified in project documentation. Working group WG 9 of CEN TC265 initiated a revision of EN 14620 and the work began in the spring of 2015. Five countries are involved.

The adaptation and development of LNG standards was pursued vigorously in the USA. The aim was to draw up a completely new standard that would close the gaps in the existing standards, which tended to focus on steel tanks, and provide self-contained rules for LNG tanks made from concrete. It was during the ACI Convention in San Francisco in the autumn of 2004 that the newly established ACI 376 Committee met for the first time. By 2011 they were ready to publish the all-new ACI 376 *Code Requirements for Design and Construction of Concrete Structures for the Containment of Refrigerated Liquefied Gases* [11]. In terms of content and the many details provided, it goes far beyond the scope of EN 14620.

3.2 EEMUA Publication No. 147 and BS 7777

Flat-bottom, vertical, cylindrical steel tanks built in situ for the storage of refrigerated liquefied gases were normally designed with a single-wall shell. They were surrounded by an earth embankment at a considerable distance. If a second steel single-wall shell was required, this was built to fix the insulation in position and protect it against the weather, thus maintaining its insulating function. The design and configuration of such tanks was carried out according to two standards:

– BS 4741 (1971): Specification for vertical, cylindrical, welded, steel storage tanks for low-temperature service: single-wall tanks for temperatures down to −50°C
– BS 5387 (1976): Specification for vertical, cylindrical, welded storage tanks for low-temperature service: double-wall tanks for temperatures down to −196°C

Up until the 1970s it was usual to store all liquefied gas products in single containment tanks. After that there was a trend towards adding an earth embankment, wall or outer container around tanks for hydrocarbons and ammonia. If the inner container leaked, the enclosure or outer container would prevent the liquefied products from escaping uncontrolled into the surroundings. Although the earth embankment solution – either a smaller embankment at a greater distance from the tank or a higher embankment very close to the tank – increased the footprint, it did lead to enhanced protection for the surrounding area. It is still customary these days to store liquid oxygen, nitrogen and argon in single containment tanks.

BS 4741 and BS 5387 only applied to single-wall tanks; they did not contain specifications or requirements for the choice of material, design, calculations, load cases, construction details, etc. for double or full containment tanks. In order to rectify this shortcoming, the EEMUA's Storage Tank Committee published its *Recommendations for the Design and Construction of Refrigerated Liquefied Gas Storage Tanks* as publication No. 147 in 1986. The aim of the EEMUA here was to create a basis for a subsequent British Standard – and in 1993, BS 7777 was introduced to replace BS 4741 and BS 5387. BS 7777 was divided into four parts:

- Part 1: Guide to the general provisions applying for design, construction, installation and operation
- Part 2: Specification for the design and construction of single, double and full containment metal tanks for the storage of liquefied gas at temperatures down to −165°C
- Part 3: Recommendations for the design and construction of prestressed and reinforced concrete tanks and tank foundations, and for the design and installation of tank insulation, tank liners and tank coatings
- Part 4: Specification for the design and construction of single containment tanks for the storage of liquid oxygen, liquid nitrogen and liquid argon

Part 1 [7] showed simplified typical drawings and various configuration options for the three types of tank and contained details of loads. It defined which load cases had to be considered depending on tank type and component.

Part 2 [8] contained details of steel tanks. Various gases were assigned to potential tank types in Table 1, and Table 2 listed the steel material required for the inner container depending on product temperature and type of tank. The inner container could be made from carbon-manganese steel, steel with a low nickel content, 9% nickel steel, aluminium or stainless steel. Table 4 prescribed a maximum thickness of the metal shell, e.g. 30 or 40 mm for 9% nickel steel. This part contained many details of and information on welding.

Provisions for the design and configuration of concrete tanks were given in part 3 [9]. The most important of these were the definition of the much lower permissible stresses when using conventional steel reinforcement at temperatures down to −165°C and how to carry out tensile tests on notched reinforcing bars at cryogenic temperatures. Part 3 defined the permissible differential settlement as 1/300 and the permissible tilt as 1/500, figures that were also repeated in subsequent standards.

Part 4 [10] covers the storage of liquid oxygen, nitrogen and argon.

3.3 LNG Installations and Equipment – EN 1473

Euronorm EN 1473 *Installation and equipment for liquefied natural gas* [12] is the umbrella European document for the planning, construction and operation of all onshore LNG plants. It covers installations for liquefaction and regasification as well as facilities for storage, which are generally referred to as tanks. EN 1473 defines the terminology and prescribes requirements regarding environmental compatibility, safety demands, risk assessments and safety engineering to be taken into account during design. The various LNG facilities are described in the standard and in Annex G:

– LNG export terminal,
– LNG receiving terminal,
– LNG peak-shaving plants, and
– LNG satellite plants.

Some sections of this standard have a direct influence on the design and configuration of concrete tanks. One of those is chapter 4, which contains recommendations regarding the assessment of safety and environmental compatibility. Once the location has been decided upon, a detailed environmental impact assessment (EIA) must be carried out. This assessment involves looking at all the emissions from the plant in the form of solids, liquids and gases during both normal operation and accidents. Plants must be designed in such a way that gas is not continuously flared or vented, instead that as much gas as possible is recovered, and the risks for people and property inside and outside the facility are reduced to a generally acceptable level. The analysis of the location might result in load cases that are relevant for the design, e.g. tsunami or blast pressure wave. Geological and tectonic soil surveys must include information on the presence of karst, gypsum, swelling clays, the susceptibility to soil liquefaction, the physical formation process and the potential for seismic activities in the future.

A risk assessment must be compiled when planning an LNG plant. Annexes I, J and K (provided for information purposes only) contain advice on defining frequency ranges, classes of consequence, levels of risk and acceptance criteria. The plant is assigned to one of three risk categories depending on the analysis of frequency ranges and classes of consequence. Those categories define whether the risk is acceptable, or must be reduced to a level that is as low as reasonably practical (ALARP), or is unacceptable. The values prescribed in the annexes are minimum requirements that may be raised by national regulations or project specifications.

The risk assessment is often part of a hazard and operation study (HAZOP), but approaches such as failure mode effect analysis (FMEA), event tree method (ETM) or fault tree method (FTM) are also permitted. Plant systems and components must be classified with respect to their relevance to safety within the scope of the risk assessment. A distinction is made here between class A, systems vital for plant safety or protection systems that must remain in operation to assure a minimum safety level, and class B, systems performing functions vital to plant operation or systems whose failure could create a hazard for the plant and in turn cause a major impact on the environment or lead to an additional hazard.

Sections 6.3 and 6.4 are relevant for the design of concrete tanks. Section 6.3 and Annex H contain details and examples of the various tank types, information that is supplemented by the more detailed provisions of EN 14620 Part 1. The information given in EN 1473 goes further than that of EN 14620 in that it includes spherical tanks as well as concrete tanks in which both the primary and secondary containers are made from prestressed concrete. Section 6.4 specifies the design principles, which cover requirements regarding fluid-tightness, maximum and minimum pressures, tank connections, thermal insulation, instrumentation, heating and liquid level limits. These principles allow design requirements to be derived covering the layout of the facility, the minimum spacing of tanks and the consideration of sources of risk such as fire or blast pressure wave.

3.4 Design and Construction of LNG Tanks – EN 14620

EN 14620, *Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0°C and −165°C*, is divided into five parts:

Part 1: General
Part 2: Metallic components
Part 3: Concrete components
Part 4: Insulation components
Part 5: Testing, drying, purging and cool-down

Part 1 [13] defines general requirements regarding the conception and selection of tank types and general performance criteria. The conception and selection of tanks is explained in detail here. The scope of applicability covers temperatures from 0 to −165°C and overpressures up to 500 mbar. Where the pressure exceeds 500 mbar we speak of a pressure vessel, which falls within the scope of EN 13445.

From the constructional viewpoint, this standard is restricted to primary containers of steel only, and explicitly excludes inner containers made from prestressed concrete. Large amounts of methane, ethane, propane, butane, ethylene, propylene, liquefied natural gas (LNG) and liquefied petroleum gas (LPG) are stored in these tanks. All these gases fall under the heading of "refrigerated liquefied gases" (RLGs). The physical properties of these gases are given in Table 3.1 (taken from EN 14620-1). EN 14620 does not apply to the storage of argon (−186°C), oxygen (−183°C) or nitrogen (−196°C); these gases will be covered by EN 14620 Part 6, which is currently in preparation.

The possible variations in these tanks with respect to stored product, volume and configuration are enormous, and so the content of the EN 14620 series cannot cover every eventuality, every detail. In the definition of the scope of the standard given in Part 1, it is explicitly mentioned that if complete requirements for a specific design are not provided, it is up to the designer to agree the design principles and details plus the appropriate reliability with the purchaser's authorised representative. What happens in practice is that the configuration is specified as part of a front end engineering design (FEED) for an LNG terminal.

Table 3.1 Physical properties of gases according to EN 14620-1.

Name	Chemical formula	Boiling point [°C]	Liquid density at boiling point [kg/m^3]	Volume of gas liberated by 1 m^3 of liquid [m^3]
n-butane	C_4H_{10}	−0,5	601	239
Isobutane	C_4H_{10}	−11.7	593	236
Ammonia	NH_3	−33.3	682	910
Butadiene	C_4H_6	−4.5	650	279
Propane	C_3H_8	−42.0	582	311
Propylene	C_3H_6	−47.7	613	388
Ethane	C_2H_6	−88.6	546	432
Ethylene	C_2H_4	−103.7	567	482
Methane	CH_4	−161.5	422	630

A specification for the LNG storage tanks for a particular project is drawn up which defines the regulations, assumptions, analyses and construction details.

LNG storage tanks normally consist of a steel inner container and concrete outer container which are designed and built by different specialist firms. The design and, more specifically, the fabrication/construction cannot be carried out separately. Therefore, section 7 clearly assigns the responsibilities for the steel, concrete and insulation components as well as the overall responsibility for the coordination. The design and configuration details are outlined in the respective sections.

EN 14620 Part 2 [14] specifies the general requirements relevant to the materials, design, fabrication, welding methods, welding, construction and installation of metal components for tanks. The types of steel required are defined depending on the liquefied gas to be stored, and hence the respective temperature and type of tank (Table 3.2).

The permissible stresses in plates and weld seams during normal operation and testing are defined, also the minimum thickness of the metal shell, which is 40 mm for butane and propane tanks, 50 mm for ethane and LNG tanks. The maximum stress in the metal container results from the volume of liquid in the tank and

Table 3.2 Type of steel depending on stored product and type of tank.

Stored product	Single containment tank	Double or full containment tank	Membrane tank	Normal storage temperature of liquefied gas
Butane	type II	type I		−10°C
Ammonia	type II	type I		−35°C
Propane/propylene	type III	type II	type V	−50°C
Ethane/ethylene	type IV	type IV	type V	−105°C
LNG	type IV	type IV	type V	−165°C

seismic action. The minimum thickness for the metal plates indirectly limits the volume of the tank. Part 2 also includes information on design and calculations, fabrication and welding. Minimum plate thicknesses or cross-section dimensions are prescribed for many parts depending on the diameter of the tank.

Part 3 [15] describes principles and details for the design and construction of concrete components, i.e. the secondary or concrete outer container, as according to the definition in Part 1, the primary (inner) container is made of steel. Requirements regarding the materials (concrete, conventional reinforcement, prestressing steel) take up only one page. In the case of concrete, the user is referred to EN 1992-1-1 and EN 206. The information provided in Annex A.1 merely calls for concrete class C40/50 for prestressed concrete components, a low water/cement ratio and a suitable percentage of entrained air, and permits the use of a reduced expansion coefficient and thermal material properties in the calculations.

Prestressing steel, anchorages and ducts must comply with EN 1992-1-1. Furthermore, it is necessary to verify that the prestressing steel and the anchorages are suitable for the low temperatures to which they will be exposed. The section on conventional steel reinforcement distinguishes between temperatures above and below −20°C, as in the preceding standard, BS 7777. Conventional reinforcement for design temperatures that do not drop below −20°C during normal operation or abnormal conditions only has to comply with EN 1992-1-1. Reinforcement and socket couplers in tension components and subjected to temperatures below −20°C must satisfy additional requirements.

"Cryogenic reinforcement", i.e. reinforcement with a higher content of nickel and other alloying constituents, is normally used for the inside face of the concrete wall because the temperature at the level of the reinforcement can drop to about −150°C during the "liquid spill" load case. The base slab is not affected by this requirement as it is protected against such temperatures by a so-called secondary bottom made from 9% nickel steel placed within the insulation. Normal reinforcement can be used in the outside face of the wall, even if temperatures below −20°C can occur in winter. It should be remembered that a temperature range of −40 to +100°C is defined in EN 1992-1-1, Annex C.

Annex A.3 provides details of tensile tests at low temperatures. Annex B contains very general information on prestressed concrete tanks, does not specify any particular requirements. Theoretically possible fixed (= monolithic), sliding and pinned joints are illustrated for the junction between the wall and the base of the tank. In the case of LNG, the boundary conditions with regard to subsoil, loads and temperature are such that only monolithic connections will satisfy the ultimate and serviceability limit state analyses.

Part 4 [16] contains details of the design requirements for and selection of insulating materials, the design of the vapour barrier against the infiltration of water vapour from outside and the vapour of the stored product from the inside, the design of the insulation system, the installation of the insulation, commissioning and maintenance. The liquefied gas stored in LNG tanks has a boiling point that is below the ambient temperature. It is therefore essential to prevent the uncontrolled or excessive infiltration of heat of evaporation. The primary functions of the insulation are to maintain a defined temperature below the boiling point, protect the components of the outer container which are not designed for such low temperatures and to limit the boil-off rate. The thermal insulation and foundation heating systems prevent the soil from freezing and the ensuing frost heave plus the formation of condensation and ice on the surfaces of the outer container. The annex to Part 4 contains recommendations for the use of various insulating materials for individual tank components and different types of tank.

In the case of LNG tanks, the thermal insulation is by no means an unimportant component, instead a vital element that is necessary if the functionality and economics of the tank system are to be guaranteed. The standard does not specify a permissible value for the quality of the thermal insulation, i.e. the maximum boil-off rate per 24 h. The value customarily taken is 0.05% of the tank volume. The boundary conditions for the analysis are ambient temperature, solar radiation and wind speed, which are laid down in the tank specification.

Part 5 [17] defines the requirements regarding testing, drying, purging and cool-down of tanks. Tank tests are divided into hydrostatic and pneumatic tests. When using single-wall tanks, these two tests are carried out together. The testing pressure is applied in the vapour space above the water. In the case of double-wall and full containment tanks, the two tests can be performed simultaneously or separately. The pressure test involves applying a pressure that is 1.25 times the design pressure. Prior to testing, pressure-relief valves must be installed and set to this pressure; they are removed again after the test. The tank is also tested for a partial vacuum, which corresponds to the design negative pressure of the tank, normally 5 or 10 mbarg. The partial vacuum is achieved with a pump or simply by lowering the level of the water.

The liquid-tightness test distinguishes between the hydrostatic pressure at full height (FH) and at partial height (PH). In the former, the inner container is filled with water to its maximum design level. In the latter, the filling level results from the product of 1.25 times the maximum design liquid level and the density of the respective liquid gas.

A combination of filling with water and internal pressure increases the load on the base slab and foundation on the one hand, but, on the other, a tank filled with water considerably reduces the volume to which the internal pressure can be applied. In addition, it reduces the duration. The decision regarding the method depends very much on the local conditions.

3.5 API 620 – the American Standard for Steel Tanks

API 620 [1] describes the design and construction of large, welded, site-built steel tanks for storing petroleum intermediates (gases or vapours), finished products and other liquid products required by various branches of industry. The standard applies to tanks that have a single vertical axis of revolution, metal temperatures not exceeding 120°C and 1 bar overpressure. The provisions of the standard are valid for tanks that are intended to store both liquids and gases or vapours above the surface of the liquid and also gases and vapours alone.

API 620 [1] includes two important appendices: Appendix Q [2] and Appendix R [3]. The provisions in these appendices form a guide for the selection of tank materials and the design and construction of tanks for storing liquefied gases.

A tank for liquefied gases can have a single- or double-wall construction, the latter consisting of an inner container for storing the liquid and an outer container to protect the insulation and to accommodate a low gas pressure. A double-wall tank is a two-part structure in which the outer container is not designed to accommodate the product in the inner container. Therefore, different requirements apply to the materials, design and testing of the inner and outer containers of a double-wall tank.

Appendix R prescribes the requirements for designing tanks for the storage of products at temperatures down to −50°C, whereas Appendix Q contains stipulations for designing tanks for the storage of liquefied ethane, ethylene and methane at temperatures down to −165°C, with an internal pressure of max. 1 bar in each case. Very many inner containers for LNG tanks are still designed and built according to this standard.

3.6 API 625 – Combining Concrete and Steel

The first edition of API 625 [18] appeared in August 2010 and, together with API 620, appendices Q [2] and R [3], and ACI 376 [11], constituted a consistent American code. API 625 is the American equivalent of EN 14620. It regulates the responsibilities between purchasers and suppliers and defines areas where both sides have to reach an agreement. Furthermore, it provides recommendations regarding the selection of the storage concept on the basis of a risk assessment, gives examples of single, double and full containment tank systems and outlines the different requirements. Various configurations of the three different types of tank system are shown and, in terms of information, these do not differ from the definitions given in BS 7777 or EN 14620-1.

API 625 also contains sections on insulation, hydrotests, pressure testing, purging and cool-down plus a chapter on design and performance criteria. It therefore covers the content of EN 14620, parts 1, 4 and 5. In the case of metal containers, it refers to API 620 for the selection of materials, design and calculations, fabrication, construction, inspections and testing. The applicable appendix, Q or R, depends on the temperature range and design temperature of the metal. ACI 376 is referred to for the selection of materials, design and calculations, fabrication, construction and testing of concrete tanks.

3.7 ACI 376 – the American Standard for Concrete Tanks

ACI 376 [11] lays down requirements for the design and construction of reinforced and prestressed concrete structures for the storage and retention of refrigerated, liquefied gases with operating temperatures between +4 and −200°C. It therefore also covers tanks for oxygen and nitrogen. Furthermore, it also permits the rules it contains to be used for the concrete foundations of double-wall steel tanks. The constructional details embrace the configuration of the tank wall, base slab, roof and foundation. The standard also regulates many other aspects – from the scope of the soil survey, to construction requirements, to commissioning and decommissioning.

Reinforced and prestressed concrete tanks can be used for two basic functions:

– Their use as secondary containers is the one most frequently encountered. The reason for this is that they protect the stored product against actions from outside and protect the surroundings against accidents inside the tank.
– The regulations also allow concrete tanks to be used as primary containers. Whereas all the other regulations remain very general when talking about concrete inner containers, ACI 376 specifies the minimum requirement and details in section 6.2.

The definition of the scope of the standard explicitly excludes membrane tanks, and neither the content nor the commentary contains any recommendations in this respect. In a membrane tank the inner container consists of a non-self-supporting thin metal layer (membrane) that is supported by the concrete outer container through the insulation (see section 4.5). Work on a revision of the standard to include membrane tanks began in spring 2015.

With additional considerations and calculations, and taking into account the hydrostatic pressure on the concrete wall as an operating condition, it is possible to employ the criteria to design concrete tanks as primary and secondary containers. ACI 376 does not include any information on steel primary or secondary containers, which must be designed according to API 620.

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3 API 620: Appendix R: Low-pressure Storage Tanks for Refrigerated Products. American Petroleum Institute, 12th ed., 2016.
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4

Definitions of the Different Tank Types

4.1 Definitions and Development of the Different Types of Tank

The different types of tank for the storage of liquefied gases are defined in a number of standards and regulations, which differ in terms of when they were published and also with regard to the level of detail they provide. The two standards in German, DIN EN 1473 and DIN EN 14620, even differ in terms of the terminology used. This section will make use of either the terms given in the British equivalent, BS EN 1473, or those used in API 625. From a practical viewpoint, the expression used in API 625 [1], the "containment tank system", seems to be the most appropriate, as the different, yet coordinated, components, through their interaction, give rise to an integrated system.

The regulations EEMUA [2], BS 7777 [3], EN 1473 [4], EN 14620-1 [5], NFPA 59A [6] and API 625 [1] distinguish between single, double and full containment tank systems. Only the European standards EN 1473 and EN 14620 describe a further tank type in somewhat more detail: the membrane tank. Table 4.1 lists the sections of the regulations dealing with each particular type of tank.

Single-wall tanks were the only type built up until the 1970s. The subsequent further development of the different types of tank or tank system as well as the associated requirements placed on the materials and construction details were derived from the hazard scenarios due to abnormal actions, e.g. failure of the inner tank, fire, blast pressure wave and impact. Owing to the risks involved for the adjoining areas due to a tank failure, it is important to choose the right type of tank system.

Taking the example of the failure of the inner container, the effects on the tank as a whole and the surroundings for the three commonly used tank systems will be described and their successive development outlined.

a) Single containment tank system
   The liquefied gas leaks out into the area (impounding basin) enclosed by the bund wall. According to EN 14620-1, the distance between the primary container and the bund wall may be as much as 20 m. This results in a large area surrounded by a low wall which is filled with LNG. The evaporation of the LNG and the (highly likely) ensuing pool fire will affect a large part of

Design and Construction of LNG Storage Tanks, First Edition. Josef Rötzer.

Table 4.1 Definitions of the tank types in the regulations.

	EN 1473 (2007)	EN 14620-1 (2006)	NFPA 59A (2013)	API 625 (2010)
Single containment tank	6.2, Fig. H.1	4.1.1, Fig. 1	3.3.5.6	3.2.1.4, 5.2, Figs. 5.1–5.4
Double containment tank	6.2, Fig. H.3	4.1.2, Fig. 2	3.3.5.1	3.2.1.1, 5.3, Figs. 5.5–5.6
Full containment tank	6.2, Fig. H.4	4.1.3, Fig. 3	3.3.5.3	3.2.1.2, 5.5, Figs. 5.7–5.10
Membrane tank	6.2, Fig. H.5	4.1.4, Fig. 4	3.3.5.4	

the facility. There is nothing to prevent the heat radiation affecting nearby buildings, structures and other parts of the plant.

b) Double containment tank system
In order to reduce the potential risks, the primary container is enclosed within another wall. The distance between the primary and secondary containers may not exceed 6 m. So the liquefied gas leaks out into the secondary container, which must be much taller than the bund wall needed for a single containment tank in order to accommodate the volume of liquid in a much smaller annular area. Therefore, the primary container is surrounded by a concrete wall in most designs. This arrangement results in a much smaller area flooded with liquefied gas and vapour. The concrete wall protects the neighbouring installations. A pool fire is restricted to a smaller area and tends to evolve upwards instead of sideways. There is less heat radiation affecting nearby buildings, structures and other parts of the plant.

c) Full containment tank system
The liquefied gas leaks out into the secondary container, which must hold this and at the same time not allow any liquid or vapour to escape. A pool fire cannot break out in this situation. Gas vapour can only escape via the emergency relief valve. The effects for the neighbouring installations are very much lower.

The following descriptions of the various tank systems are based on EN 14620-1 [5] and API 625 [1].

4.2 Single Containment Tank System

A single containment tank system is the designation for a liquid- and vapour-tight container. It can be built as a liquid- and vapour-tight single-wall structure or as a combination of inner and outer containers. In the latter case, the inner container is open at the top and liquid-tight.

Where an outer container is being used, this is essentially needed to enclose the insulation, protecting it against moisture, and to accommodate the gas vapour overpressure. It is not intended or designed to contain leaking LNG. A single containment tank must be surrounded by some form of safety enclosure, normally an earth embankment, in order to prevent the liquid from flowing uncontrolled into the surroundings.

EN 14620 stipulates that the inner container must be made of steel, whereas API 625 also permits the use of prestressed concrete. If an outer container is being used, it is normally made from carbon steel. Fig. 4.1 shows the various design options based on API 625.

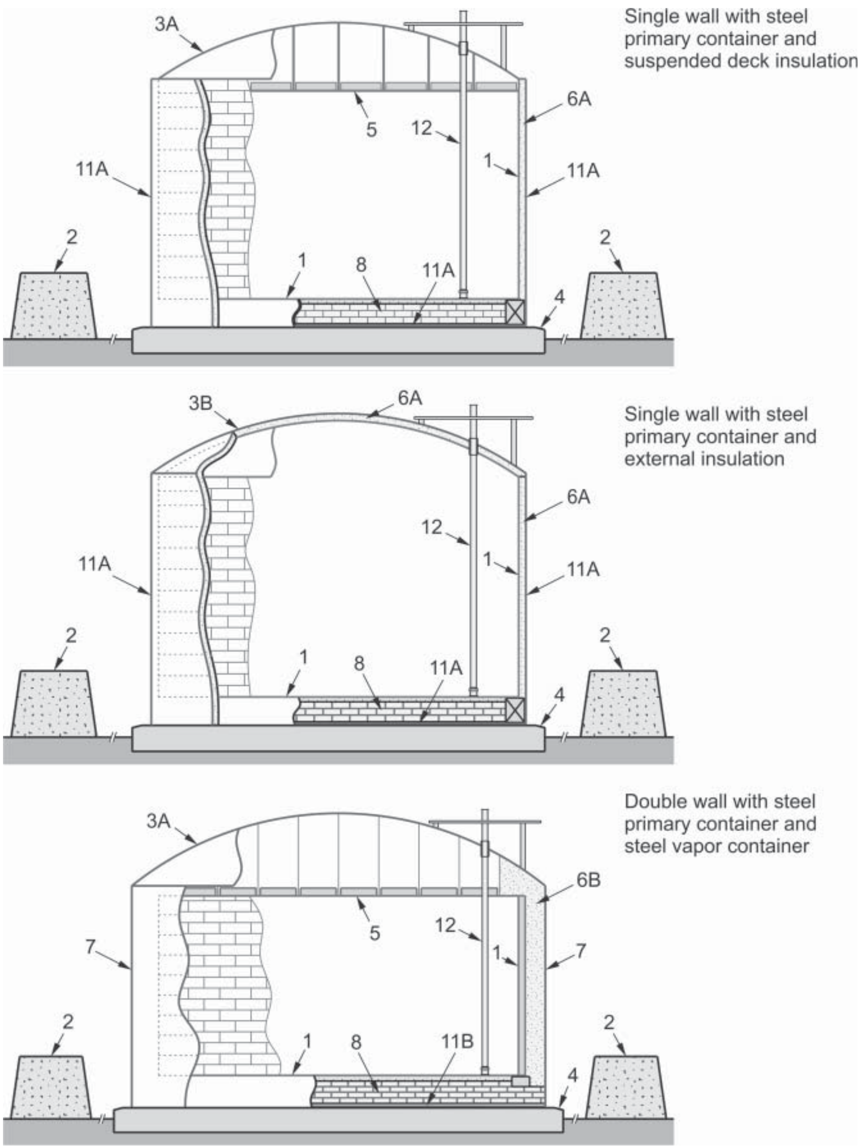


Fig. 4.1 Single containment tanks.

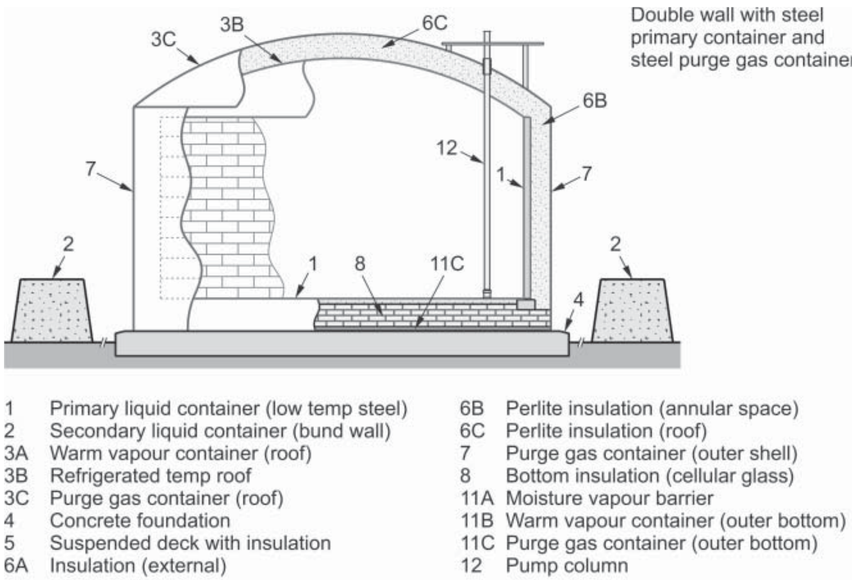


Fig. 4.1 (*Continued*)

4.3 Double Containment Tank System

A double containment tank system consists of a liquid- and vapour-tight primary container that satisfies the requirements for a single containment tank system on its own but is built within a secondary container (Fig. 4.2). The latter is open at the top and it must be able to hold the escaping liquefied gas in the event of a leak. However, it is not designed to prevent the escape of gas. The annular space between the primary and secondary containers may not be more than 6 m wide. API 625 permits the use of steel and prestressed concrete for both containers.

4.4 Full Containment Tank System

A full containment tank system consists of primary and secondary containers which together form an integrated, complete storage system. The primary container is a self-supporting, cylindrical steel tank with just one shell. It can be open at the top and thus unable to contain any vapour, or it can be built with a dome roof and in that case prevent vapour from escaping.

The secondary container must be a self-supporting tank made of steel or concrete with a dome roof. Where the primary container is open at the top, the secondary container must constitute the primary vapour containment of the tank during normal operation. In the case of a leak from the primary container, the secondary container must be able to hold the liquefied gas and remain liquid-tight while still acting as the primary vapour containment structure. Controlled venting via the pressure relief system is permitted. Where the outer container is built of concrete, API 625 says that “product losses due to the

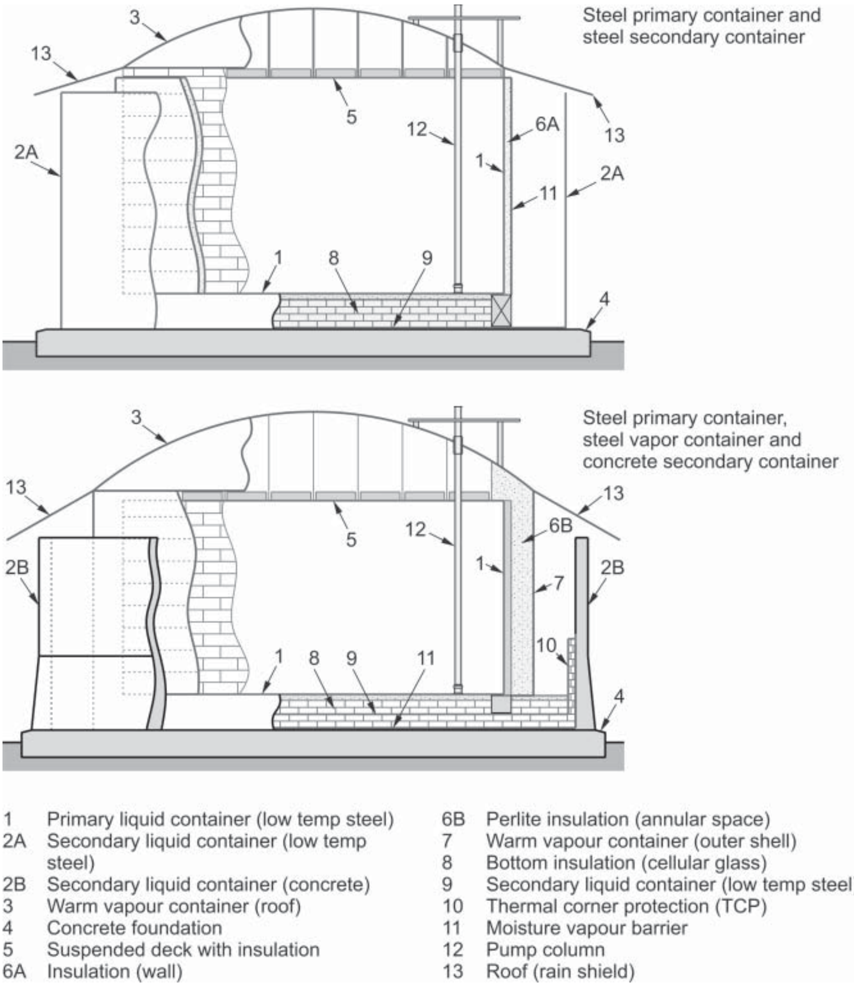


Fig. 4.2 Double containment tanks.

permeability of the concrete are acceptable". API 625 permits the use of steel and prestressed concrete for both containers. Vapour-tightness is required for normal operation. Fig. 4.3 shows a number of design options, including one with a prestressed concrete inner tank.

EN 14620-3 (Annex B) and ACI 376 (Appendix A) describe sliding, pinned and fixed joints between the wall and the base slab and show details of these. This author sees the use of sliding or pinned joints as being only possible for small tanks at best, operating at less extreme low temperatures and hence a lower overpressure. For LNG tanks, the monolithic wall/base slab junction is the only viable option.

The standard full containment tank with a concrete outer container and rigid monolithic connection between wall and base slab requires two constructional features in order that the system still functions reliably in the event of failure

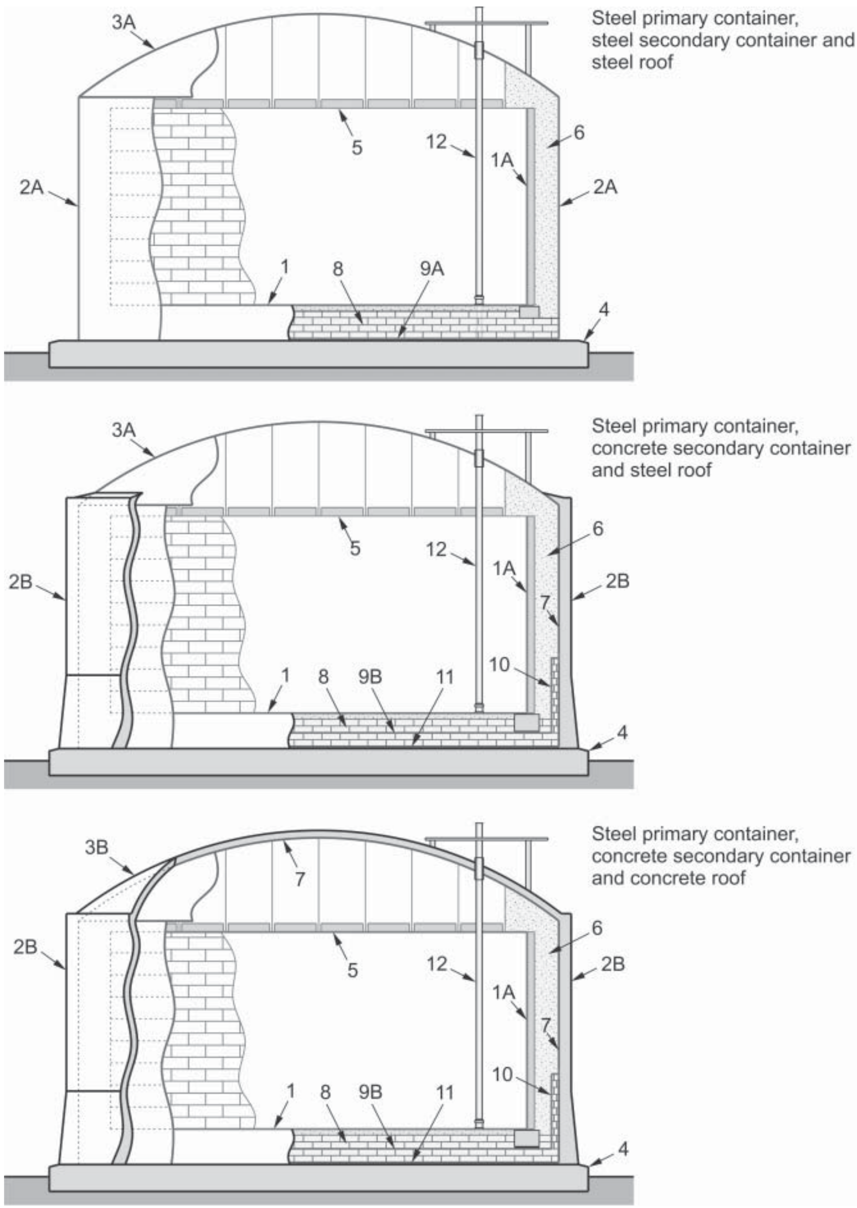


Fig. 4.3 Full containment tanks.

of the inner container. In that situation the wall is subjected to temperature gradients of up to 200°C and a temperature difference of about 100 K. With the tank diameters commonly in use, this temperature difference results in a corresponding radial shortening of the wall amounting to 4–5 cm. If additional measures are not taken, this leads to failure of the concrete cross-section at the wall/base slab junction. One remedy is to provide a transition zone at least

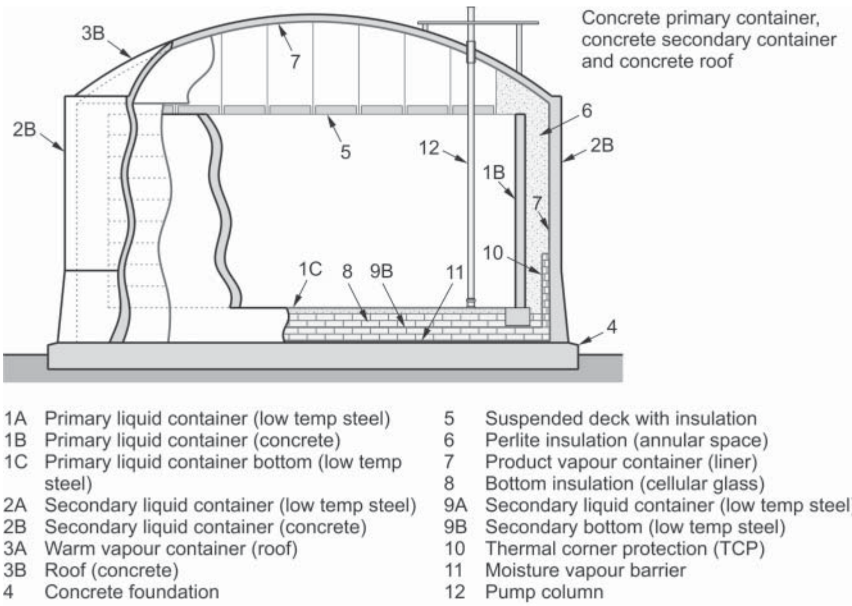
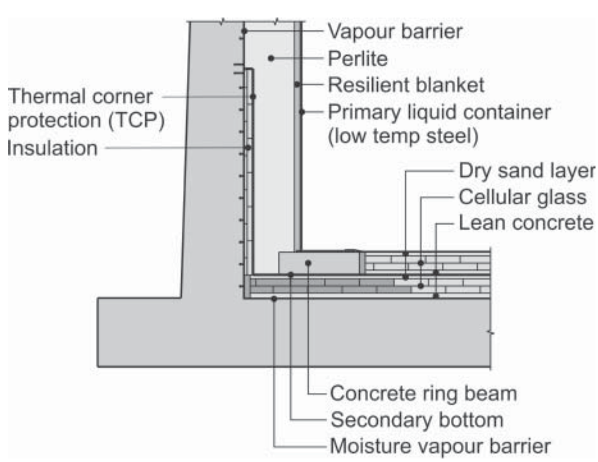


Fig. 4.3 (*Continued*)

5 m high at the base of the wall to reduce the contraction of the concrete wall to a compatible level (see Fig. 4.4). This is achieved practically by integrating a so-called secondary bottom made from 9% nickel steel in the insulation to the base slab and the wall. This secondary bottom is turned up the wall. The section with insulation and steel plates constitutes the so-called thermal corner protection (TCP). Details of the TCP are shown in Fig. 4.4; the TCP can also be seen in Fig. 4.3 (Nos. 9B and 10). This detail protects the insulation and also helps it to maintain its thermal function, thus reducing the effect of the temperature on the concrete cross-section and smoothing the deformation development.

Fig. 4.4 Detail of thermal corner protection (TCP).

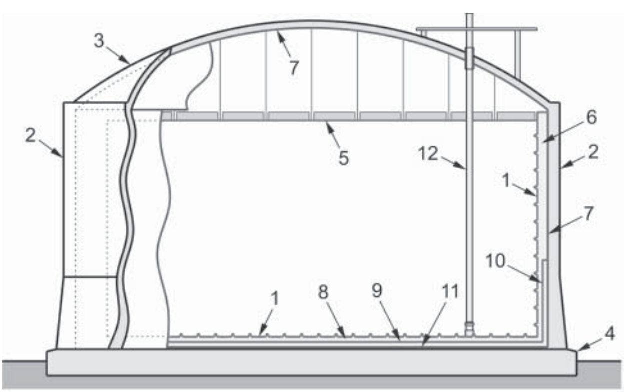


Even though experience has shown us that the risk of failure of a single containment tank (assuming it was built according to the regulations) is very low, such risks can be further reduced by introducing even stricter requirements regarding choice of materials, design, construction, inspection and testing. However, for certain hydrocarbon products, the effects of a tank failure are so serious that an even better tank system is necessary. The tank system should be chosen taking into account the location, the operating conditions and environmental criteria.

4.5 Membrane Tank System

The membrane system (Fig. 4.5) was originally developed for use on LNG tankers and only afterwards was it also employed for onshore, above-ground LNG storage tanks in a number of very different projects. Recent years have seen a great rise in the demand from operators for membrane tanks as an alternative to 9% nickel steel tanks. There are various reasons for this. The LNG industry has always been striving to reduce costs by replacing the thicker nickel steel plates by thin stainless steel ones. From an engineering viewpoint, the advantage of a membrane tank is that there is no restriction on the thickness of the tank wall plates when it comes to large-capacity tanks and good seismic performance in the event of high earthquake loads.

Many years ago, membrane tanks were only available from a few suppliers, who were in competition with the suppliers of full containment tanks. However, the membrane tank system has since been licensed. Therefore, any contractor can now purchase a license, which means that all suppliers are able to offer both full containment and membrane tanks.



1 Membrane (corrugated stainless steel)
2 Secondary liquid container (concrete)
3 Concrete roof
4 Concrete foundation
5 Suspended deck with insulation
6 Wall insulation (prefabricated elements)
7 Moisture vapour container
8 Bottom insulation (prefabricated elements)
9 Secondary liquid container (low temp steel)
10 Thermal corner protection (TCP)
11 Moisture vapour barrier
12 Pump column

Fig. 4.5 Membrane tank system.

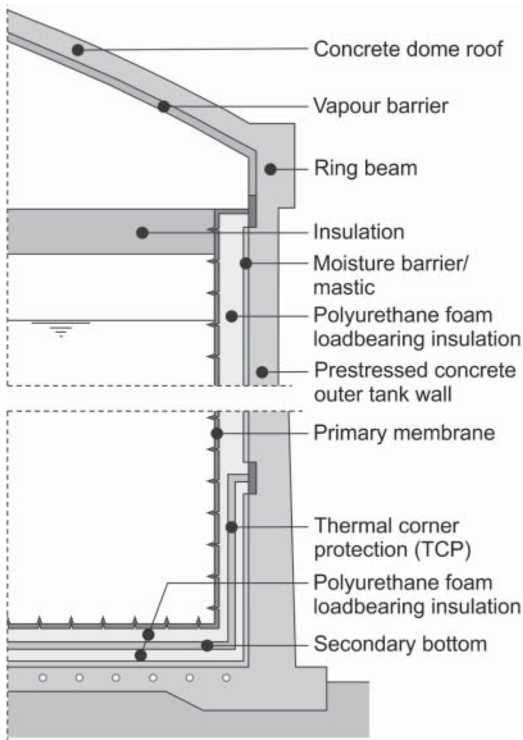


Fig. 4.6 Designations of individual membrane tank components.

According to the definitions in EN 1473 und EN 14620-1, a membrane tank system consists of a thin membrane (serving as primary container), thermal insulation and an outer concrete secondary container which together form a composite tank structure. An outer container in steel is not allowed for in the regulations. Fig. 4.6 shows the individual components of a membrane tank.

The primary container consists of a liquid- and vapour-tight stainless steel membrane, at most only about 1.5 mm thick (Fig. 4.7). The membrane does not perform any loadbearing functions. The plates are produced with corrugations at 90° to each other. Each prefabricated segment is fixed to the outer container.

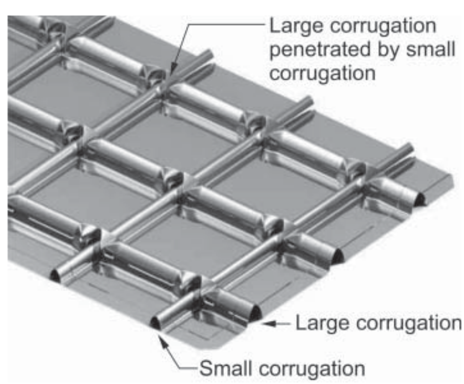


Fig. 4.7 Membrane plate with different corrugations.

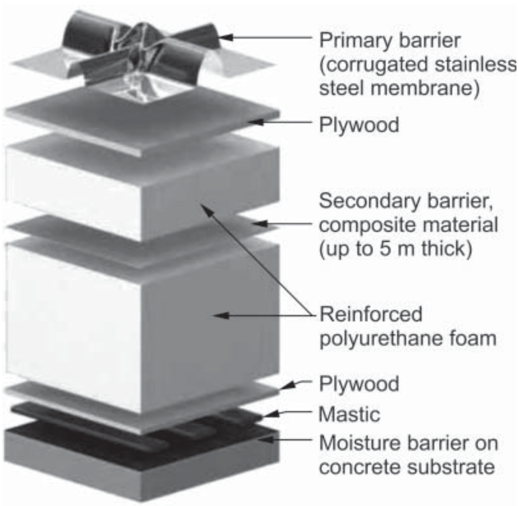


Fig. 4.8 Make-up of inner container with insulation.

This arrangement allows the membrane to deform unhindered under the effect of thermal actions without stresses and strains being induced. The separate membrane segments are overlapped and welded together. At the top of the wall the membrane is anchored to the concrete wall by a peripheral plate.

A continuous vapour barrier is attached to the inside face of the tank wall and the concrete base slab in order to prevent any water vapour or water from the concrete infiltrating the insulation space. In addition, the roof lining of plain carbon steel, the compression ring and the membrane are welded together in order to guarantee that the entire tank is gas-tight. This means that both the membrane on the inside and the vapour barrier on the outside are sealed, creating an enclosed insulation space. To make sure that the insulation continues to function and to control the infiltration of moisture or vapour, the insulation space is monitored and constantly purged with nitrogen.

Within the insulation, a secondary membrane is provided as an integral secondary bottom in the base slab and the bottom part of the wall; it is anchored about 5 m up the wall. Fig. 4.8 shows the secondary barrier between the two layers of reinforced polyurethane foam. In the event of failure of the membrane, it functions as additional protection for the rigid wall, protecting it against high restraint stresses due to thermal actions. This arrangement is known as a thermal protection system (TPS). All insulation segments are prefabricated.

The concrete outer container is the only loadbearing part of the entire system and has to resist the loads due to liquefied gas and vapour overpressure plus all the external loads. During normal operation, it is not exposed to any significant thermal loads, just like a full containment tank system. In the event of failure of the inner container, the concrete outer tank must resist the pressure of the liquid and vapour and remain liquid- and vapour-tight.

References

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3 BS 7777-1: Flat-bottomed, vertical, cylindrical storage tanks for low temperature services – Part 1: Guide to the general provisions applying for design, construction, installation and operation. BSI, London, 1993.
4 EN 1473: Installation and equipment for liquefied natural gas – Design of onshore installations, May 2016.
5 EN 14620-1: Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0°C and -165°C – Part 1: General. Beuth Verlag, Dec 2006.
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5

Performance Requirements and Design

5.1 Performance Requirements for Normal Operation

The design requirements laid down in EN 14620-3 (concrete components) are only brief, occupy only one page. Those requirements deal with specifying the partial safety factors for abnormal actions and combinations thereof. Those abnormal actions are safe shutdown earthquake (SSE), blast overpressure, external impact and leakage from the inner container (liquid spill).

Liquid-tightness of the outer container is the other point dealt with in the design requirements. A distinction is made here between containers with and without a liquid-tight steel liner or coating. The associated design scenario assumes that LNG is escaping from the inner container and the inside face of the concrete outer container in the region above the thermal corner protection (TCP) comes into direct contact with the cold liquid at a temperature of −165°C.

As already mentioned, EN 14620 does not specify any partial safety factors or combination load factors on the loads side for normal operation and normal actions; the respective tank specification has to rectify this shortcoming. Each load case for an abnormal action is superposed on the load cases due to permanent and variable actions. The partial load factors from EN 14620-3 are reproduced in Table 5.1.

The vapour- or gas-tightness of the outer container should be ensured by way of a metallic or polymer lining. This is normally achieved for LNG tanks by attaching 5 mm thick standard carbon steel plates to the inside face of the concrete structure. The vapour-tightness stipulation applies to normal operation.

The liquid-tightness requirement applies to failure of the inner container. However, a metallic lining of normal carbon steel cannot handle cryogenic temperatures and so liquid-tightness has to be ensured by the concrete cross-section. In this case the standard merely calls for a residual compression zone of 100 mm – a requirement that is much less stringent than all other comparable standards. Worldwide, the requirement is for an additional average compressive stress of 1 N/mm^2 (145 psi) in the residual compression zone, i.e. a residual compressive stress of 2 N/mm^2 at the extreme fibres. Where a liquid-tight liner or coating is used, which is the exception, crack widths must be limited according to EN 1992-1-1 and it must also be verified that the liner or coating can bridge over a gap equal to 120% of the required crack width.

Design and Construction of LNG Storage Tanks, First Edition. Josef Rötzer.

Table 5.1 Partial load factors for abnormal actions.

Load combination	Load factors for Dead loads adverse	Dead loads beneficial	Imposed loads adverse	Imposed loads beneficial	Abnormal loads	Wind loads
Normal action plus one abnormal action	1.05	1.0	1.05	0	1.0	0.3

Abnormal actions include earthquakes (safe shutdown earthquake) blast overpressure, external impact, fire and leakage from the inner container.

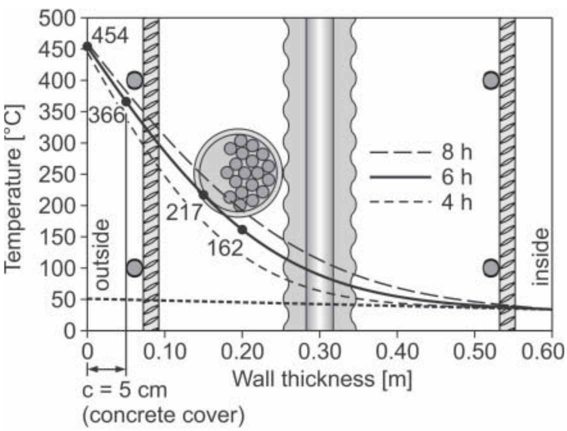

Fig. 5.1 The temperature gradient in a tank wall during a fire.

When it comes to construction details, EN 14620-3 merely points out that vertical prestressing *may* be necessary. This depends on the tank design pressure, which is very different for export and receiving terminals due to the different plant technologies needed for liquefaction and regasification. The tank design pressure for export terminals is in the region of 100 mbarg, whereas it is 300 mbarg for receiving terminals. In order to comply with the provisions regarding crack width limits, depth of compression zone and compressive stresses, receiving terminals require a vertical prestress. The scale of the technical and economic benefits when employing vertical prestressing for export terminals depends on the particular tank specification.

To protect the prestressing tendons against an external fire, EN 14620-3 recommends positioning them in the centre of the cross-section. This recommendation can be followed to a certain extent only. Where both horizontal and vertical prestressing is being employed, the vertical prestress is applied in the centre of the cross-section, the horizontal prestress outside of this, roughly at the third-point. This arrangement represents a very good compromise between structural and fire requirements. As an example, Fig. 5.1 shows the temperature gradient in a concrete wall exposed to 32 kW/m^2 radiation.

For concrete cover and minimum reinforcement, designers should comply with the provisions of EN 1992-1-1.

5.2 Thermal Design

This section on the thermal design of the tank explains the calculation of the boil-off rate of a tank during normal operation. Boil-off rate is the proportion of refrigerated liquid that evaporates per day due to heat that reaches the interior of the tank from outside. This represents one, if not the most important, criterion for assessing the serviceability of a tank.

EN 14620-1 calls for the tank to be designed for a defined boil-off rate, which is not explicitly prescribed, instead must be specified by the tank owner. In the case of large tanks, a figure of 0.05% of the tank volume is specified in the majority of cases. For smaller tanks, the figure is in the range 0.07–0.08%. In exceptional cases a value of 0.10% might be specified for sites in tropical regions. One of the few publications to include a determination of the boil-off rate is that of the Japan Gas Association [1]. It includes the following method of calculation plus details of the thermal conductivity and heat transfer of insulating materials for the entire temperature range from −170 to +50°C.

Based on steady-state temperature conditions, the heat transmission per 24 h is calculated for an idealised 12 h day/12 h night situation. The initial temperature is taken to be the maximum ambient temperature, which is laid down by the tank owner in the tank specification. The base slab temperature is given by the design of the heating. The daytime calculation has to take into account the fact that the concrete surface is heated up by solar radiation, the degree of which depends on the latitude of the site and local air pollution. Values between 700 and 900 W/m^2 are used in the calculations. Only a part of this radiation actually contributes to heating up the surface of the concrete; the other part is reflected by the surface or radiated back from the surface into the surroundings.

To calculate the boil-off rate, the thermal transmittance U is first calculated for each of the individual external surfaces with the same cross-sectional make-up. In this calculation the tank walls are assumed to be cylindrical surfaces for which Eq. (5.1) with $R_o > R_m > R_i$ is valid:

$$U = \frac{2 \cdot \Pi}{\frac{1}{R_o \cdot h_o} + \frac{1}{\lambda} \ln\left(\frac{R_o}{R_m}\right) + \frac{1}{\lambda} \ln\left(\frac{R_m}{R_i}\right) + \frac{1}{R_i \cdot h_i}} \tag{5.1}$$

Multiplying this by the associated height of the wall H and the temperature difference ΔT, we get the heat flow q in watts [W]:

$$q = \frac{2 \cdot \Pi}{\sum \frac{1}{\lambda} \cdot \ln\left(\frac{R_o}{R_i}\right) + \frac{1}{R_i \cdot h_i}} \cdot H \cdot \Delta T \tag{5.2}$$

The next step is to calculate the products of the individual heat flows and the duration of the day or night phase and add these together. This results in the energy in joules [J] which infiltrates the tank each day. Comparing this with the energy stored in the LNG (volume × density × specific heat of evaporation) gives us the boil-off rate.

Table 5.2 Hydrostatic tests for different tank types.

Stored product (tank content)	**Single containment tank**	**Double containment tank**	**Full containment tank**	**Membrane tank**
Ammonia Butane Propane Propylene	Tank (steel types II & III) FH	Inner tank (steel types I & II) FH	Inner tank (steel types I & II) FH	
		Outer tank (steel types I & II) FH	Outer tank (steel types I & II) FH	
		Outer tank (prestressed concrete) no test	Outer tank (prestressed concrete) no test	Outer tank (prestressed concrete) PH
Ethane Ethylene LNG	Tank (steel type IV) PH	Inner tank (steel type IV) PH	Inner tank (steel type IV) PH	
		Outer tank (steel type IV) PH	Outer tank (steel type IV) PH	
		Outer tank (prestressed concrete) no test	Outer tank (prestressed concrete) no test	Outer tank (prestressed concrete) PH

FH: full hydrotest (test over full height)
PH: partial hydrotest (test over reduced height)

5.3 Hydrostatic and Pneumatic Tests

Before going into service, the entire tank system with inner and outer containers, foundation, base slab, weld seams and liner, possibly also anchorages and safety valves for overpressure and partial vacuum, is tested by applying a load higher than the normal operating load in the hydrostatic and pneumatic tests. In the hydrostatic (= liquid-tightness) test, the inner container is filled with water, whereas in the pneumatic (= gas-tightness) test, the outer container is subjected to overpressure and a partial vacuum (negative pressure). The two tests can be carried out separately, but performing them together does bring benefits.

EN 14620-5 defines the level to which the inner container should be filled for the hydrostatic test, which depends on type of tank and stored product. A distinction is made between a fully filled tank (full hydrotest, FH) and a partly filled tank (partial hydrotest, PH) (see Table 5.2).

In the full hydrotest, the inner container is filled with water to its maximum design liquid level. As different liquefied gases have different densities, the factor of safety varies somewhat. In the partial hydrotest, the inner container is filled to a level corresponding to the product of 1.25 times the design liquid level and the density of the liquefied gas. For LNG with a density of 480 kg/m^3, this is achieved at 60% of the design liquid level ($0.60 \cdot 10.0/4.80 = 1.25$). A pressure relief system for overpressure and partial vacuum must be installed prior to the hydrostatic test. The pressure prevailing inside the tank must be transferred to a column of water on the outside of the tank which indicates the pressure head. The easiest way to do this is to use a hose levelling instrument marked with a scale to represent the pressure.

The deformations and settlement of the inner and outer containers should be monitored and measured at the same time. EN 14620-5 calls for eight measuring points on the outside surface of the tank, although tank specifications normally require many more. ACI 376 limits the spacing of the measuring points to 10 m. The same number of points in identical positions should be used inside and outside in order to prevent errors and simplify the test procedure. In double and full containment tank systems, measuring points should also be marked on the inner tank shell so that settlement of the inner tank can be monitored at same time as monitoring the outer tank.

The duration of the filling procedure depends on the availability of water and the state of the supply network; sometimes water has to be brought in by tankers. If several tanks are being built simultaneously, it is expedient to carry out the hydrostatic tests consecutively, reusing the water each time. According to EN 14620-5, the settlement of the tank must be measured when it is half full, three-quarters full and full; however, most tank specifications call for measurements at closer intervals.

At each filling stage, the settlement is measured at the circumferential measuring points on the inner container and the base slab for the outer container. Settlement at the centre of the tank is determined with inclinometers fitted in the base slab – two at 90° to each other to achieve redundancy. Experience has shown that it is essential to train personnel in the correct use of the inclinometers and to verify the measurements while the base slab is still accessible and can be checked with a levelling instrument.

The design calculations for an LNG tank include forecasts of the settlement behaviour during construction, the hydrostatic and pneumatic tests and the period of operation. Upper and lower bounds are specified for the characteristics of each soil stratum (see section 5.4). The settlement calculations are carried out with the upper and lower bounds, and the settlement as measured should lie between these two limits.

The pneumatic tests consist of overpressure and partial vacuum tests. A pressure of 1.25 times the tank design pressure is applied during the overpressure test. Pressure relief valves must be installed and set to this pressure prior to carrying out the test. The test pressure is maintained for 30 min, thereafter reduced to the tank design pressure. The safety valves are then set to the tank design pressure. Air is pumped into the tank to check its serviceability. The partial vacuum test must be carried out at a design negative pressure, normally 5 mbarg. A minimum holding time is not prescribed. The partial vacuum can be achieved with a pump or by lowering the water level, which is clearly the simpler method. Once the design negative pressure has been reached, the safety valves are set to this pressure and then tested. In order to counteract uplift of the steel base or the base insulation and the thermal corner protection, a sufficient amount of water should remain in the tank during the partial vacuum test.

5.4 Soil Survey, Soil Parameters and Permissible Settlement

Information on the design of the foundation can be found in section 7.1.9 of EN 14620-1 and annex B.7 of EN 14620-3. That information is not prescriptive, represents merely general advice, such as the determination of differential and total settlement and verification that all tank components can accommodate such settlement.

Therefore, permissible settlement and permissible tilt figures are given in the tank specification. Those figures normally correspond to those in the withdrawn standard BS 7777-3, which are included in section 10.3.5.2 of ACI 376. The commentary in the American standard points out that the permissible differential settlement between the edge and centre of the base slab, which reflects the dishing of the base slab, was defined to avoid damage to the cellular glass insulation. The settlement limits specified in ACI 376 are as follows:

– No limit is specified for the total settlement, provided that the pipework is designed for such a total settlement.
– Maximum permissible tilt: 1/500
– Maximum permissible differential settlement between edge of base slab and centre of tank: 1/300
– The maximum settlement around the perimeter of a tank is limited to 1/500 and may not exceed the defined permissible tilt.

EN 14620 calls for the settlement behaviour to be monitored during the various phases of the tank's life. Whereas the change in the loading over time during construction and hydrostatic testing can be predicted relatively exactly, it is not possible to predict the filling levels, and hence the loads, during normal operation. Therefore, in the case of soils whose behaviour varies over time, a sufficiently wide margin should be allowed for. No requirements are specified for the nature, scope or details of soil surveys, instead the standard refers to EN 1997-2.

ACI 376 devotes a whole chapter to foundations and defines requirements for soil surveys, ground improvement measures, shallow and deep foundations, monitoring, checking and testing. Even though some of the definitions do not differ from those for normal engineering works, defining and specifying the values considerably eases discussions with tank owners or their representatives.

The tank foundation is crucial for the serviceability of a liquefied gas tank. That is why very precise limits are specified for the settlement (see Table 5.3). Section 10.2.2 of ACI 376 covers the soil survey and defines the number, location and depth of boreholes as well as cone penetration tests (CPT) and standard penetration tests (SPT). The number depends on the footprint of the tank. Four boreholes are required up to a diameter of 100 ft (30.5 m), one in the centre and three distributed over the perimeter. One further borehole is required for each additional 10 000 ft^2 (929 m^2).

Table 5.3 Permissible settlement according to ACI 376 and BS 7777.

Nature of settlement	Permissible value
Total settlement	no limit
Tilt	1/500
Differential settlement	1/300
Settlement along perimeter of tank	1/500 and less than the permissible tilt

Table 5.4 Soil surveys required by ACI 376.

Diameter [ft]	**Diameter [m]**	**Area [ft^2]**	**Area [m^2]**	**Number of boreholes n**
100.0	30.5	7 854	730	4
150.8	46.0	17 854	1 659	5
188.3	57.4	27 854	2 588	6
219.5	66.9	37 854	3 517	7
246.8	75.2	47 854	4 446	8
271.4	82.7	57 854	5 375	9
293.9	89.6	67 854	6 304	10
314.8	96.0	77 854	7 233	11

Table 5.4 contains recommendations for tanks up to 100 m in diameter, which roughly corresponds to a tank capacity of 250 000 m^3. These figures represent minimum requirements. Additional soil survey measures will be required in the case of irregular site topography or soil stratification, areas of fill or where soil strata thicknesses differ or do not run horizontally. The usefulness of CPTs is explicitly pointed out. If these are used in combination with boreholes and sampling, they represent an efficient tool for the in situ determination of soil parameters over large areas.

Boreholes and CPTs should be arranged evenly over the tank footprint. Fig. 5.2 shows the locations that this author would specify when performing a soil survey for a tank with a 90 m diameter foundation and 200 000 m^3 capacity.

One borehole with SPT and CPT should be located at the centre of the tank and the others at radii of $0.6 \cdot R_0$ and $1.0 \cdot R_0$. The angular angle between the boreholes and CPTs along the perimeter should not exceed about 45°. Those along the inner circle should be positioned at an angle to their neighbours on the perimeter. Where soil survey reports are already available or the results of additional surveys allow clear predictions, then the number of boreholes, SPTs and CPTs may be reduced.

The depth of each borehole should not be less than the radius, i.e. 45 m in this case. The SPT number (number of blows) should be determined every 1.0 m in the boreholes. In addition to the nine boreholes for the 90 m diameter foundation, nine CPTs should also be carried out. They should be 30–45 m deep.

All these in situ investigations should be supplemented by evaluations of laboratory tests. It is essential to check the settlement behaviour during construction

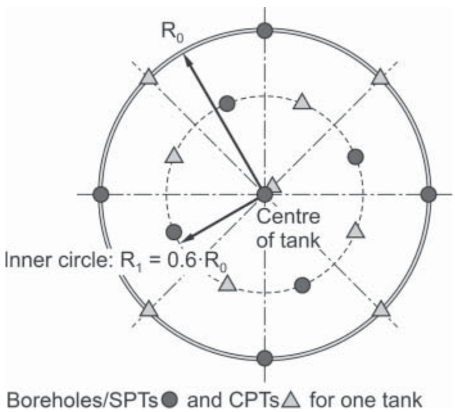

Fig. 5.2 Arrangement of boreholes/SPTs and CPTs.

and the hydrostatic and pneumatic tests and compare the measurements with the predicted values. To do this, stainless steel studs are cast into the edge of the base slab as level reference points and two inclinometers are installed in the base slab at 90° to each other. Inclinometers are unnecessary if the differential settlement is expected to be < 30 mm. If the settlement measured during construction and the hydrostatic test does not agree with the predictions, then the causes must be determined and, if possible, measures taken to prevent damage to the tank. Additional analyses and investigations are needed to evaluate the subsoil conditions and, specifically, its susceptibility to soil liquefaction. These investigations are described in section 5.5.

5.5 Susceptibility to Soil Liquefaction

Soil liquefaction is a possibility where strata extend over large areas or where there are thick lenses of loose sand below the groundwater level. Liquefaction occurs during an earthquake when the pore water pressure rises in the saturated sands and reaches the overburden pressure, at which point the intergranular pressure decreases and the shear strength of the sand stratum is reduced. The susceptibility to soil liquefaction can be determined by way of in situ SPTs or CPTs in conjunction with laboratory tests to determine the percentage of fine particles in the soil. EN 1998-5 [2], annex B, contains a diagram showing the limits to the soil liquefaction range based on $N_1(60)$ SPT values for an earthquake of magnitude 7.5 and fine particle percentages of 5, 15 and 35% (see Fig. 5.3).

According to EN 1998-5, section 4.1.4(8), the risk of soil liquefaction may be ignored when $\alpha \cdot S < 0.15$ and at least one of the following conditions is fulfilled:

– The sands have a clay content > 20% with a plasticity index PI > 10.
– The sands have a silt content > 35% and at the same time an SPT number, normalised for overburden effects and the energy ratio, $N_1(60) > 20$.
– The sands are clean and have an SPT number, normalised for overburden effects and the energy ratio, $N_1(60) > 30$.

Berhane [3] contains an evaluation of the risk of soil liquefaction and its causes plus the background to this phenomenon.

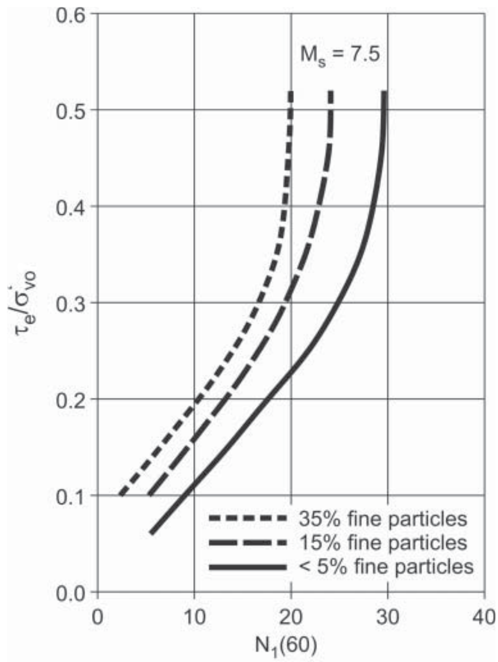

Fig. 5.3 Susceptibility to soil liquefaction and percentage of fine particles.

Verification with the Standard Penetration Test

Fig. 5.3 is based on a reference value for number of blows $N_1(60)$. The SPT numbers normally specified in soil reports, expressed as number of blows per 30 cm, therefore have to be converted to an effective overburden pressure of 100 kPa (corresponding to an atmospheric pressure of 1 bar) and to a ratio of 0.6 between impact energy and theoretical free-fall energy. Eq. (5.3) is used to carry out this conversion to the reference value $N_1(60)$:

$$N_1(60) = C_N \cdot C_{ER} \cdot C_B \cdot C_S \cdot C_R \cdot N_{30} \tag{5.3}$$

The overburden effects are corrected by multiplying the measured N_{SPT} value by a factor $(100/\sigma'_{vo})^{1/2}$, where σ'_{vo} [kPa] is the effective overburden pressure acting at that depth at the time the SPT was carried out. EN 1998-5 limits the correction factor C_N to values between 0.5 and 2.0.

$$C_N = (p_a/\sigma'_{vo})^{0.5} = (100/\sigma'_{vo})^{0.5} \tag{5.4}$$

with $0.5 < C_N < 2.0$

The following factors are also taken into account:

C_{ER} energy ratio factor
C_B borehole diameter factor
C_S sampling method factor
C_R rod length factor
N_{30} number of blows for 30 cm

Verification Based on Cone Penetration Tests

In the diagrams based on CPTs the reference value q_n/p_a is used for the horizontal axis instead of number of blows $N_1(60)$. As for SPTs, the cyclic shear stress related to the effective overburden pressure τ_c/σ'_{vo} is used for the vertical axis. However, the standard does not include a diagram for this.

$$q_n/p_a = C_Q \cdot q_c/p_a \tag{5.5}$$

$$C_Q = (p_a/\sigma'_{vo})^n = (100/\sigma'_{vo})^n \tag{5.6}$$

$n = 0.5$ for pure sands

$n = 1.0$ for clayey soils

$$q_N/p_a = C_Q \cdot q_c/p_a = (100/\sigma'_{vo})^n(q_c/p_a) \tag{5.7}$$

Calculating the Susceptibility to Soil Liquefaction

If the soil above a depth h is idealised as a beam in shear, the maximum shear stress is

$$\tau_{max} = \gamma \cdot h \cdot a_{max}/g \tag{5.8}$$

However, as the soil does not have finite shear stiffness and can deform, the shear stress will decrease. This behaviour is taken into account with a reduction factor r_d. The range of r_d is very small in the uppermost 10–15 m, but increases considerably with the depth. During an earthquake, the variation in acceleration results in a similar variation in the shear stress. This is taken into account in the analyses by using an equivalent shear stress, which is considered to be 65% of the maximum shear stress. The maximum acceleration is expressed by $a_g \cdot S$:

$$\tau_e = 0.65 \cdot a_g \cdot S/g \cdot \sigma_{vo} \cdot r_d \tag{5.9}$$

Factor r_d is not included in Eq. (4.4) of EN 1998-5.

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6

Tank Analysis

6.1 Requirements for the Analysis of the Concrete Structure

The concrete tank has to be analysed for the serviceability and ultimate limit states by means of a combination of 2D and 3D finite element models. The analysis also has to take into account heat transfer, steady-state and unsteady-state temperature gradients plus the non-linear material behaviour of the concrete. A model of the whole tank must include the soil and the foundation, with the associated effects such as soil-structure interaction, the concrete tank, inner container and roof platform. Regulations and standards call for the following aspects to be taken into account.

When analysing LNG tanks, it is customary to form upper and lower bounds for the soil parameters. Based on the soil report, best estimates for the soil parameters are specified for each soil stratum. After that, the upper and lower bounds are determined by multiplication or division ($1 + c_v$, where c_v is the uncertainty factor). The greater the uncertainties and the scatter of the soil parameters and the smaller the number of boreholes etc., the larger c_v should be. Where the soil survey complies with the stipulations of ACI 376 (see section 5.4), then c_v generally lies between 0.35 and 0.50. The procedure for taking into account scatter is that given in ASCE 4 [1], which recommends $c_v = 0.50$ when a sufficient amount of suitable soil data are available, and $c_v = 1.00$ for the case of an insufficient amount of data. As ASCE 4 covers structures for nuclear power, where the safety standards are higher than for LNG tanks, it is this author's opinion that lower standards than those given in ASCE 4 should apply for LNG tanks. Even with $c_v = 0.50$, the range is very large, and so some tank specifications permit the use of the best estimate soil parameters in the analysis of load combinations including abnormal load cases. The upper and lower bounds of the soil parameters must be considered in the load case combinations with permanent and variable loads. Until EN 14620 includes information on how to take the subsoil into account, the recommendation is to work with the stipulations of ACI 376.

Two different types of thermal analysis are required: On the one hand, the maximum and minimum temperature gradients that are established within the concrete cross-section. These temperature gradients are determined on the basis of the lowest and highest ambient temperatures with the help of steady-state

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temperature analyses. On the other hand, abnormal loads, e.g. failure of the inner container (with the escape of the cold liquid associated with such a failure), plus the actions due to various fire scenarios are taken into account by way of unsteady-state thermal analyses. In these analyses it is not always 100% clear as to the time at which the most unfavourable loads occur, so an elaborate investigation is necessary. The results of the thermal load cases must be superposed on the other permanent and variable load cases. Temperature-dependent material properties (e.g. stress-strain relationships, coefficients of thermal expansion, strength-temperature relationships) must be taken into account. Heat transfer by way of convection and radiation is modelled with the help of a film coefficient.

It is not possible to specify a damage scenario for the liquid spill load case, as the quantity and flow rate of the leaking liquefied gas cannot be predicted. In order to be able to estimate – at least approximately – the load on the concrete wall due to escaping LNG, various constant liquid levels are examined by means of an unsteady-state thermal analysis.

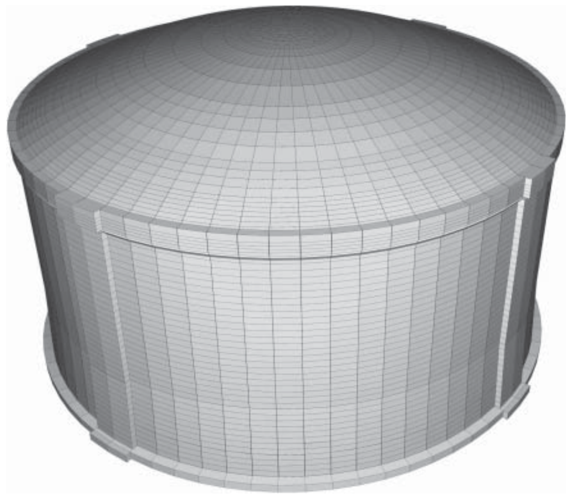
The concrete is exposed to extreme temperatures during both the liquid spill and fire load cases. The temperature gradient is so steep that large areas of the concrete cross-section crack. These non-linear cracking effects and the associated decrease in stiffness must be taken into account in the calculations.

The superposition combinations with the respective partial safety factors for the serviceability and ultimate limit states are defined in the particular tank specification; alternatively, ACI 376 offers guidance on this point. Numerous superposition options result from the combination of different variable loads with upper and lower bounds for soil and temperature. If the subsoil, or rather the foundation, has to be considered as non-rotationally symmetric, then it is necessary to consider wind and earthquake for various directions as well. Consequently, several dozen cases must be considered just for the linear-elastic analysis. In contrast to the linear-elastic analysis, in which the individual load cases can be superposed automatically, the various datasets for the load case combinations must be defined for a non-linear analysis. The relevant superpositions of the linear calculations can form the starting point for the non-linear analysis. The size and number of the elements must be such that it is possible to model the geometry and the loadbearing behaviour, but should not increase the computing time excessively.

6.2 Requirements for the Model of the Concrete Structure

The concrete tank has some loadbearing members that have to be included in the finite element model: ring beam and buttresses (Fig. 6.1). The latter are frequently used on prestressed circular tanks so that it is possible to anchor several tendons in a line along a common meridian. When using buttresses, the tendons do not have to extend around the entire circumference of the tank in one piece. Instead, they are lapped at the buttresses, with the laps of adjacent tendons being staggered. This staggering of the tendons helps to achieve a more uniform stress

Fig. 6.1 A 3D model of a concrete outer container.



distribution around the circumference of the tank. Prestressing from both ends reduces the friction losses. As a rule, even large tanks with a diameter of 90 m require four buttresses. In Korea and Japan the design of the prestressed concrete structure assumes a higher concrete compressive stress in the tank walls. The force transfer calls for a greater distance between the anchorages and hence the need for six buttresses in many cases.

The buttresses must be rigidly connected to the base slab, and so this connection must be designed and reinforced accordingly. A few tanks have been built with a joint between the base slab and the buttresses. This means that the junction between the wall and the base slab has a constant wall cross-section over the entire circumference of the tank. The intention of this design approach is to produce more consistent forces or stresses at the slab/wall junction and reduce the influence of the buttresses. Each buttress strengthens the wall in the vertical direction like a T-beam, which decreases the compressive stress and increases the crack width in the buttress. A buttress creates a fixity effect for the wall, which increases the amount of reinforcement required at the wall/buttress junction. It is therefore necessary to include the buttresses in the model of the whole tank and carry out separate calculations to verify certain details (see section 6.3).

The buttresses are about twice the thickness of the wall, and the wall thickness itself can vary considerably. On a soft subsoil, the full and empty tank load cases generate larger rotations and larger moments with changing signs, which means the wall thickness has to be increased at the junction with the base slab. The transition is achieved with two or three battered wall sections. At the top, the wall can be thinner. When employing horizontal and vertical prestressing, a wall at least 60 cm thick will be necessary in order to guarantee good conditions for installing the reinforcement and ducts and placing the concrete.

The second important loadbearing member is the ring beam, which forms the transition between roof and wall. The inside of the roof is spherical, and the radius chosen is often the same as the tank diameter. That results in an angle of 30° at the wall/roof junction. The main job of the ring beam is to resist the thrust of

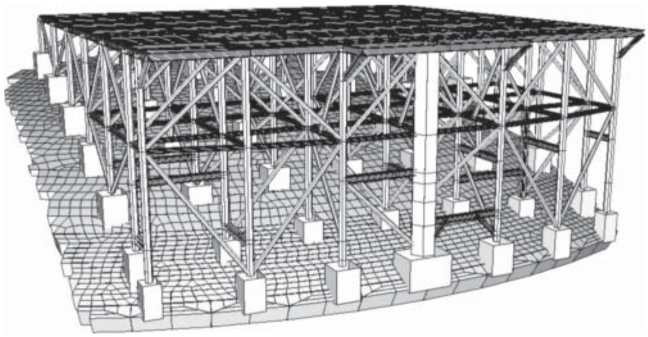


Fig. 6.2 The roof platform.

the dome roof, and that requires several tendons. Further tendons are needed to keep the ring beam and adjoining tank wall in compression over the entire cross-section.

If in addition to the structural aspects described above we also take into account the intended method of construction, primarily the concrete pours for the wall and the roof, then the result is not so many options for generating our FEM mesh. Devising a way to model the tank also includes checking the sensitivity of the mesh with respect to the ensuing internal forces. The reader is advised to consult the example of patch loads on silos by Rombach [2].

Care should be exercised when generating the mesh automatically. There are so many geometric restraint points where a load is transferred or where the amount, bar size, position or direction of reinforcement changes. Automatic mesh generation is advisable for the central area of the base slab and the irregular area of the roof platform (Fig. 6.2).

6.3 Strut-and-Tie Models for Discontinuity Regions

In certain areas, determining the internal forces and designing with FEM does not supply sufficiently accurate results. The design methods for reinforced concrete structures are generally intended for components and cross-sections with a regular stress distribution where the Bernoulli hypothesis applies (B-regions). Other areas are known as discontinuity regions (D-regions) because the form (geometric discontinuity) or loading (static discontinuity) changes. For these areas, an analysis using strut-and-tie models supplies more accurate results regarding the flow of the forces, and hence the positions and sizes of the reinforcement required, than an FEM analysis of the whole system. D-regions in LNG tanks are the ring beam with its connections to roof and wall, the buttresses, owing to the change in cross-section and the transfer of the prestressing forces, and where the depth of the base slab changes (see Fig. 6.3).

The D-regions can be modelled with a fine FEM mesh and the stress trajectories determined. After that, the tension and compression zones of the stress trajectories are combined to form the struts and ties in a strut-and-tie model (STM). The

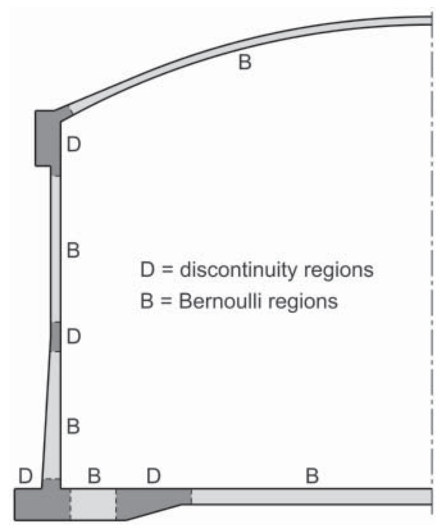


Fig. 6.3 Bernoulli regions (B-regions) and discontinuity regions (D-regions).

internal forces in the adjoining B-regions, the support reactions and the loads acting supply boundary conditions for the STM. In contrast to the very regular B-regions, a separate STM has to be developed for every D-region. However, STMs are already available for many cases which can be adapted to the actual situation of a particular tank.

The necessity for and the advantages of designing with STMs will be illustrated by way of two examples. The first of these is the change in depth of the base slab, which is dealt with in detail in [3]. The second is the force transfer from the stressing anchorage and the ensuing flow of forces in the buttress [4].

Fig. 6.4 shows an example of a change in depth of a beam, which in this case is applied to the slab/footing transition. The moments in the adjoining regions lead to tensile and compressive forces acting on the edges of the STM. Using the STM, which traces the load paths, the forces are combined on the left and right sides. What we learn from this is that different moments with different signs lead to

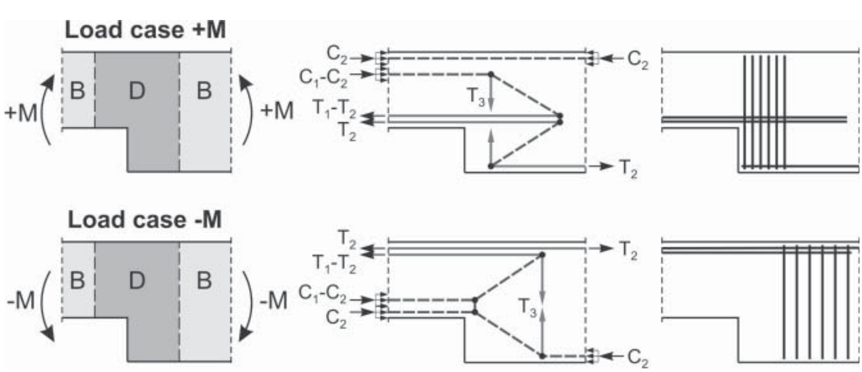


Fig. 6.4 Strut-and-tie models for change of depth in base slab.

considerable changes in the position and arrangement of the horizontal and vertical reinforcement – something that it is not apparent in the FEM calculations.

The flow of forces in the buttress depends on the geometry and stressing sequence and is therefore different for every tank. On small tanks the buttresses can be widened to avoid reverse curvature of the tendons. With larger tank diameters, on the other hand, it is hardly possible to avoid reverse curvature, as the tangential deviation from a circular arc takes place only very gradually, which would therefore make the buttress very wide and very thick. In the design of the buttress the aim should be to avoid reverse curvature, or at least to reduce it, which means that fewer stirrups (links) are required within the buttress cross-section. Additional stirrups are required in the middle of the buttress if the tendons have reverse curvature and the radial forces then act outwards. Even if the ends of the tendons are straight, the confining effect of the prestress is lacking. Furthermore, it is not difficult to provide an additional vertical U-shaped tendon in a widened buttress to produce a more uniform compressive stress state.

Whether or not transverse reinforcement is required at the edge of the buttress depends on the stressing sequence. If the tendons of one ring are tensioned one after the other, the loading situation is that shown in Fig. 6.5b. At the side of the buttress that is tensioned first, transverse reinforcement equal to about $P/6$ (P = prestressing force) will be required. If the stressing sequence is uncertain,

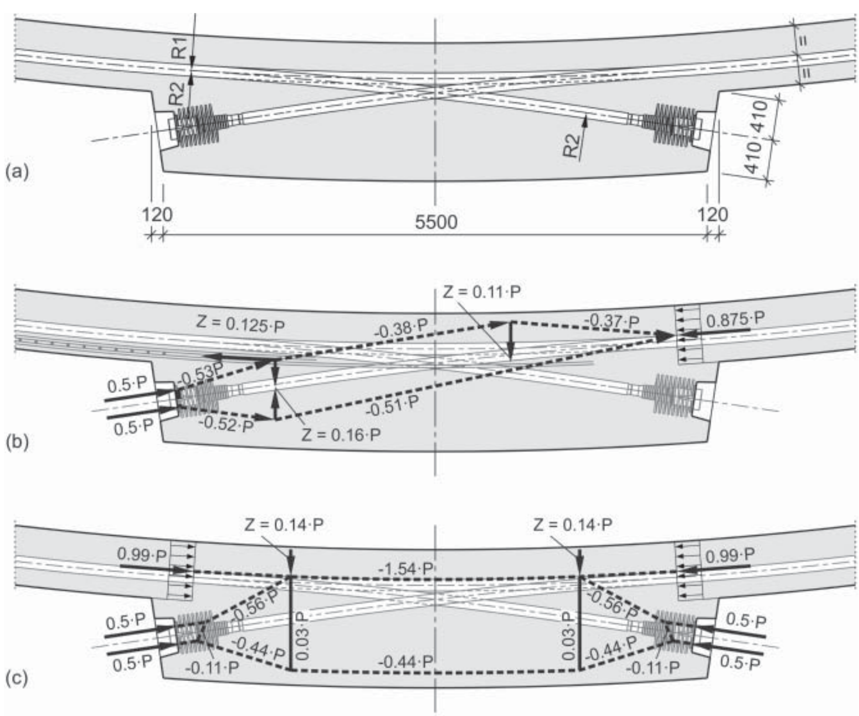


Fig. 6.5 Strut-and-tie model of buttress during stressing: a) layout of tendons, b) flow of forces when stressing from one side only, c) stressing from both sides simultaneously.

the recommendation is to provide reinforcement on both sides. Also required are bars equal to about $P/8$ laid parallel with the tendons and tying the buttress into the wall. The exact values depend on the particular STM. After the second tendon has been stressed, the flow of forces is as shown in Fig. 6.5c. If both tendons at a buttress are stressed simultaneously, the flow of forces shown in Fig. 6.5c develops immediately.

6.4 Liquid Spill

The concrete outer tank (secondary container) must be designed to accommodate the maximum amount of liquefied gas in the primary container. It is assumed that the annular space, and hence the secondary container, is filled gradually. This load case is known as "liquid spill". In addition to the entire contents of the inner container leaking out, it is also necessary to investigate the consequences of only small amounts escaping and leading to small patches with a very low temperature. This is the "cold spot" load case.

There are no stipulations regarding this scenario or the flow rate of the escaping liquefied gas. If a leak develops in the primary container and LNG escapes, it flows into the annular space between the inner and outer tanks. At the start of this process, the LNG remains in the region of the thermal corner protection (TCP). Once the level reaches the upper edge of the TCP, this represents the maximum load on the anchorage of the TCP in the concrete wall, as the cast-in steel cools and contracts while the concrete wall maintains its temperature and thus does not contract. The ACI 376 Committee is currently working on a "Code for Thermal Protection", which should include clear stipulations concerning the cast-in steel and its anchorage in the concrete wall. It must be ensured that the steel does not become detached from the wall, thus allowing LNG to flow behind the TCP. The 2011 edition of ACI 376 called for a maximum crack width of 0.20 mm in an area above the TCP anchorage equal to at least twice the wall thickness (see Fig. 6.6).

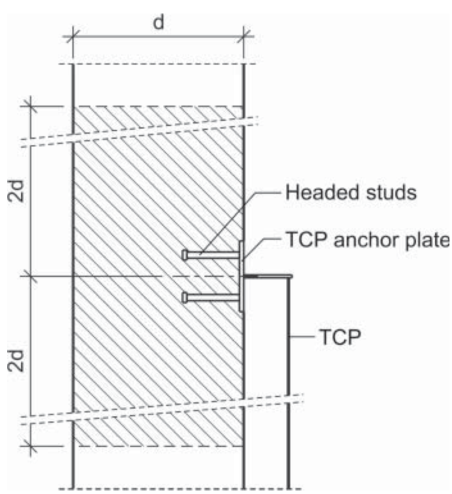


Fig. 6.6 Fixing of thermal corner protection (TCP).

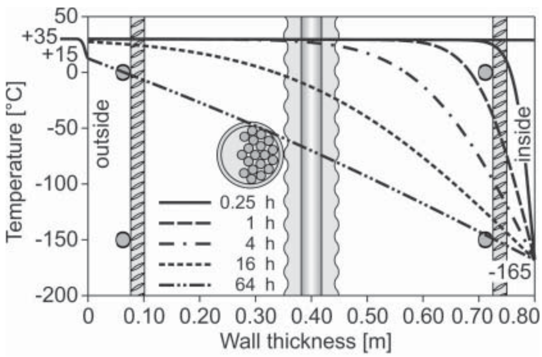


Fig. 6.7 Temperature gradient in concrete wall, development over time.

If liquefied gas continues to leak out, the level rises further. The LNG comes into direct contact with the concrete wall, which starts to cool as a result. It takes almost three days for a linear temperature gradient to become established (see Fig. 6.7).

The change in temperature in the wall over time cannot be known because the flow rate of the leaking LNG, and hence the rise in the level, is unknown. At the same time as the cooling effect infiltrates the concrete, so the level of the liquid rises, cooling the surface of the concrete higher up the wall – the two effects are superimposed. In order to be able to approximate the temperature-time relationship, various constant liquid levels are examined and the ensuing internal forces plotted as an envelope. Four or five levels are usually prescribed in tank specifications: the maximum possible level, one level above the TCP anchorage and several other levels in between.

The effects of a changing level on a wall of constant thickness have been investigated in [5]. It was shown that the level of the liquid has a direct influence on the bending moment. In the upper half of the tank the maximum bending moment is affected only marginally by the level of the LNG. The maximum bending moments are found in the region directly above the TCP anchorage and occur with a liquid level just a few metres above the TCP anchorage, because this creates curvature in two opposing directions which almost coincide (see Fig. 6.8).

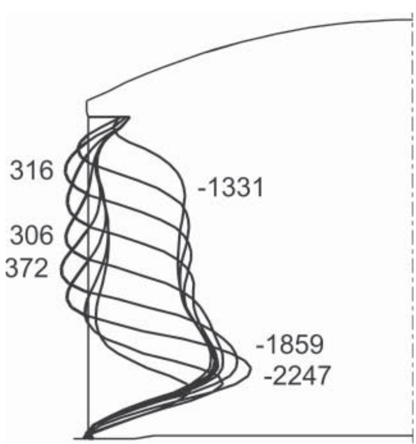


Fig. 6.8 Bending moments for different liquid levels.

The concrete tank must hold the liquid and resist the pressure of the gas if the inner container fails. The liquid-tightness requirement must be satisfied by the concrete cross-section. To achieve this, EN 14620 merely calls for a compression zone of 100 mm within the concrete cross-section. ACI 376 and the majority of tank specifications require a compression zone of at least 90 mm (some specifications require 100 mm) or 10% of the wall cross-section, whichever is the greater, plus a concrete compressive stress of min. 1 N/mm^2 in this residual compression zone. In the opinion of this author, the combined requirement for a residual compression zone and a residual compressive stress of 1 N/mm^2 should be regarded as a minimum.

Stricter requirements are placed on the reinforcement in those areas where the temperature of the reinforcement during normal operation or an emergency situation drops below -20°C – a limit that was stated in the now withdrawn BS 7777 and has been included in EN 14620-3 unchanged. However, annex C.1 of EN 1992-1-1 defines a lower limit of -40°C below which the properties of the reinforcing steel are no longer valid. This difference has no effect for LNG tanks, but does play a role when a tank contains propane or ammonia.

The unsteady-state temperature analysis shows us the course of the isotherms. The -20°C isotherm indicates the area in which cryogenic reinforcement is necessary. Fig. 6.9 illustrates the isotherms around the TCP and just above it; in this example the TCP anchorage is at a level of 5.0 m and the wall 80 cm thick. The area requiring cryogenic reinforcement extends below the TCP anchorage for a distance equal to roughly twice the thickness of the wall and hence into the first wall ring (i.e. first concrete lift). Cryogenic reinforcement must therefore be specified for the vertical reinforcement on the inside face for the first wall ring, whereas the cryogenic reinforcement in the horizontal direction can be matched exactly to the temperature. The same is true for the area above the level of the liquefied gas. Calculations have shown that the -20°C isotherm lies about one wall thickness above the level of the liquid.

The additional requirements regarding material properties at low temperatures are defined in EN 14620-3, annex A. Two options are specified in ACI 376: calculating with reduced stresses or using a material with better toughness properties.

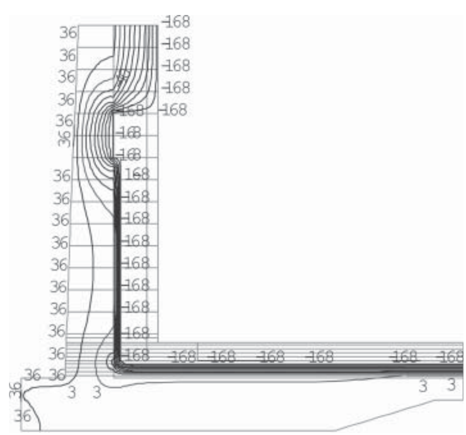


Fig. 6.9 Isotherms around TCP.

Table 6.1 Mechanical properties of cryogenic reinforcement.

Property	KRYBAR-165		KRYBAR-620	
@ room temperature		**EN 1992-1-1**		**EN 1992-1-1**
Yield strength f_y	> 500 N/mm^2	> 500 N/mm^2	> 500 N/mm^2	> 500 N/mm^2
Tensile strength	> 1.10 f_y	> 1.08 f_y	> 1.08 f_y	> 1.08 f_y
Elongation 5d	> 15%	–	> 14%	–
Total elongation A_{gt}	> 6%	> 5%	> 5%	> 5%
@ −165°C		**BS 7777-3**		**EN 14620-3**
Total elongation A_{gt} of unnotched bars	> 4%	> 3%	> 3%	> 3%
Total elongation A_{gt} of notched bars	> 1%	> 1%	–	–
Yield strength	–	–	> 1.15 f_y	> 1.15 f_y
Notch sensitivity ratio (NSR)	> 1.0	≥ 1.0	≥ 1.0	≥ 1.0

However, the American code does not specify any figures, merely refers to EN 14620-3. Where only small amounts of reinforcement or small bar diameters are involved, it is sometimes not worth using cryogenic reinforcement as the costs would be disproportionately high. In such cases a much lower steel stress is assumed in the calculations. BS 7777 (withdrawn) specifies 68.9 N/mm^2 for diameters $12 < d \leq 25$ mm and 55.2 N/mm^2 for $d > 25$ mm; NFPA 59A is the same except that the figure of 55.2 N/mm^2 applies to bars 25 mm and larger.

The requirements given in annex A.3 of EN 14620-3 relate to the plastic elongation and yield strength. Furthermore, a notch sensitivity ratio (NSR) ≥ 1 is required for notched specimens. The NSR is defined as the ratio of the tensile strength of a notched bar to the 0.2% proof stress of an unnotched bar. Accordingly, cryogenic reinforcement is manufactured according to EN 14620-3 and the withdrawn BS 7777. The data of one manufacturer are listed in Table 6.1.

When it comes to prestressing steel, it is necessary to verify that the strands and anchorages are suitable for the temperatures that will arise. Couplers are regarded as suitable when the results of tensile strength and toughness tests at the design temperature deviate by only 5% from the figures for ambient temperature.

6.5 Fire Load Cases

All fire scenarios that must be examined and analysed result from exceptional or fault-related incidents, i.e. are always abnormal actions. Therefore, non-linear calculations are required for load case combinations with fire load cases. The description of the actions due to a fire are not obtained via a defined design fire, fire curve or time-temperature curve, instead via the definition of the maximum permissible heat radiation intensity at the surface of the component. According to EN 1473 [6], this is 32 kW/m^2 for the external concrete surfaces of storage tanks and 15 kW/m^2 for steel surfaces.

The fire scenarios to be investigated are either defined during the approval phase and the front end engineering design (FEED) phase or the tank designer must define and check potential scenarios. The procedure can be divided into three steps:

– idealising a radiation, or rather fire, scenario,
– calculating the temperature development in the concrete components (unsteady-state heat transfer calculation), and
– calculating the loadbearing behaviour of the concrete structure when exposed to fire (non-linear structural calculation).

The fire scenario depends on the layout of the plant. It takes into account neighbouring tanks, impounding basin and other sources of fire. Rötzer and Salvatore [7, 8] have presented the procedure for a fire at an emergency relief valve; Fig. 6.10 illustrates this scenario.

The emergency relief valve must be positioned so high above the concrete roof that the heat radiation to which the roof is exposed does not exceed 32 kW/m^2 – for all wind directions and wind speeds. The maximum radiation value of 32 kW/m^2 applies only at the point at which the radiation acts perpendicular to the roof dome. Either side of this, the radiation decreases due to the increasing distance and angle. Rötzer and Salvatore [7] show that a linear distribution of the radiation represents a good approximation of the reality.

The next step involves calculating the temperature development within the concrete. At the surface of the concrete, a heat exchange takes place with the surrounding air. A convective heat transfer coefficient is included to take account of this effect. The coefficient depends mainly on the movement of the air in contact with the concrete surface, rising with the increase in movement. EN 1473 prescribes different wind speeds for different situations – from 1.5 m/s for determining the boil-off rate to 10 m/s for heat radiation. The heat transfer coefficient is calculated independently of this, usually with the equation $h_c = 5.7 + 3.8 \cdot v$ [9]. Applying a common wind speed $v = 4$ m/s results in a heat transfer coefficient of 20.9 W/m^{2}K ($\alpha = 1/h_c = 0.048$ m^{2}K/W).

The material parameters required for calculating this unsteady-state heat transfer are the thermal conductivity λ_c [W/mK] and the specific heat c_p [kJ/kgK] of concrete depending on the temperature. EN 1992-1-2 specifies values for both of these parameters; these are shown in Figs. 6.11 and 6.12 and are included in

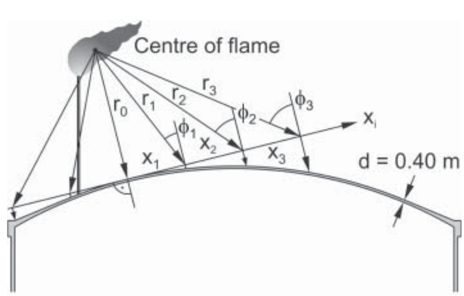


Fig. 6.10 Idealised heat radiation scenario.

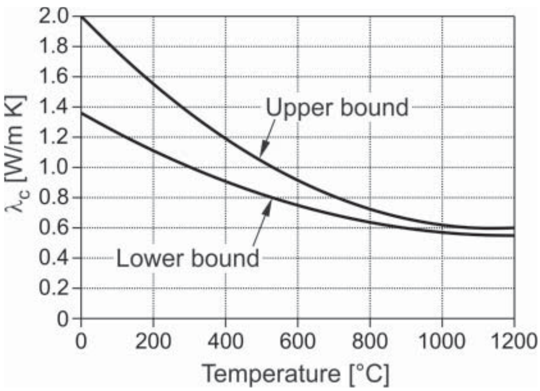


Fig. 6.11 Upper and lower bounds of thermal conductivity.

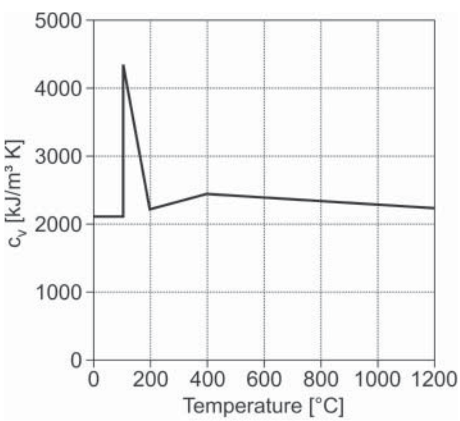


Fig. 6.12 Relationship between specific heat and temperature.

many FEM programs. If the specific heat is not as shown in Fig. 6.12, instead is assumed to be constant, the results differ only marginally.

Over time, the temperature gradient migrates further into the concrete cross-section. Fig. 6.13 shows how the temperature gradient within a cross-section changes over time for a 40 cm thick tank dome.

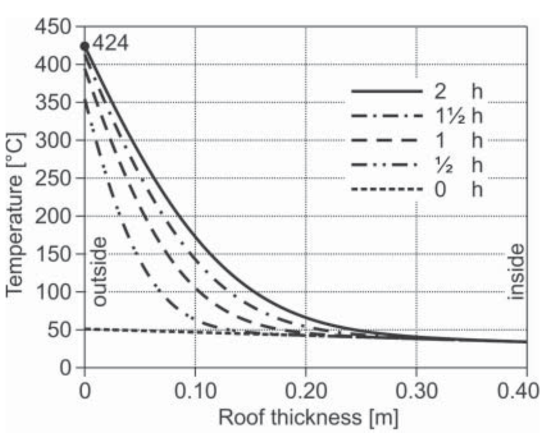


Fig. 6.13 Temperature gradient in tank roof due to heat radiation.

The fire load case must be combined with other standard load cases. The partial safety factors were discussed in section 5.1. A non-linear analysis must be performed for all load combinations that include abnormal loads. Where it is unclear as to which load combination is critical, a linear-elastic calculation must be carried out first to clarify this issue.

The amount of reinforcement required for an initial non-linear calculation results from the envelope embracing minimum reinforcement, the reinforcement required during construction phases and the reinforcement due to linear superposition. The reinforcement is adjusted iteratively until the required limits for concrete compressive strain and steel elongation are complied with. The figures used should take into account the arrangement of the reinforcement, e.g. any curtailment corresponding to the individual wall rings. Fig. 6.14 shows an example of the hoop reinforcement in the outside face.

The next issue that has to be addressed is that of the strength figures to be used in the calculations. The temperature gradient within the cross-section has to be considered here. After 2 h exposure to heat radiation, the temperature of the concrete surface is below 500°C (see Fig. 6.15). Compression prevails on the outside of the dome, tension on the cold inside face. The simplified calculation method according to EN 1992-1-2, section 4.2, prescribes reducing the concrete compressive strength only for temperatures > 500°C. So in this case it is not necessary to work with a lower compressive strength.

The stresses and strains must be evaluated and checked. If necessary, the calculation must be repeated with reduced strength figures or, as an approximation, a reduced concrete cross-section.

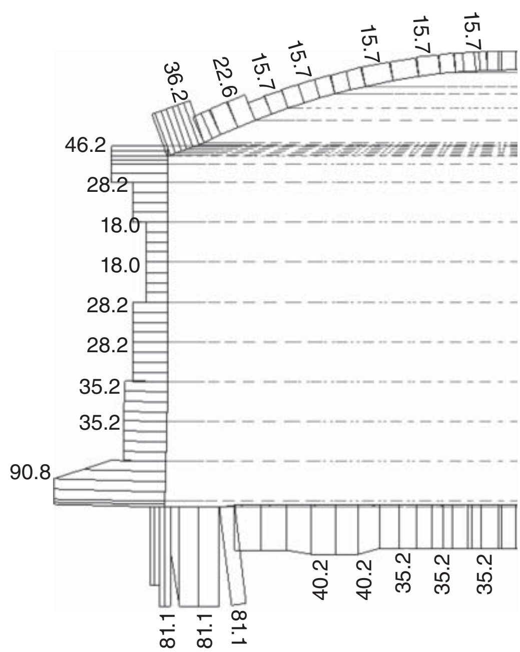


Fig. 6.14 Example of hoop reinforcement in outside face of tank wall.

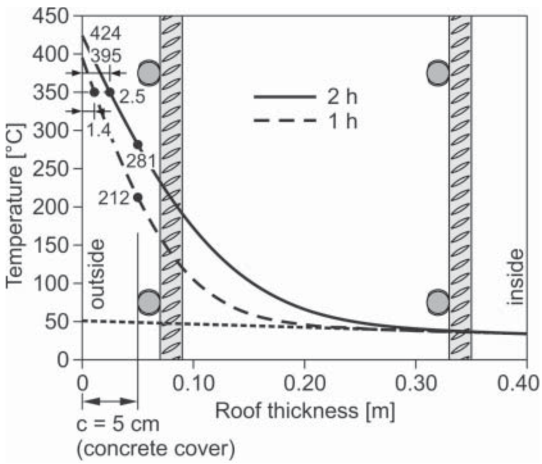


Fig. 6.15 Temperature gradient in roof after 1 h and 2 h.

6.6 Explosion and Impact

Like so many recommendations, the definitions of loads for explosion and impact are based on EEMUA 147 [10]. The starting point for the calculation is an explosion in the vicinity of a tank. Such an explosion results in two actions on the concrete tank: a pressure wave acting on the entire tank structure and flying debris that strikes the surface of the tank and causes local stresses.

An explosion in the vicinity of a tank generates a pressure wave that is reflected by the wall and roof of the tank. Such a pressure wave is of short duration and therefore the dynamic behaviour of the entire tank structure, including its foundation, should be taken into account in the analysis [10, 11].

The course of the pressure over time must be determined in a risk analysis and specified by the tank owner. One possible calculation procedure is outlined in the ASCE publication *Design of Blast-Resistant Buildings in Petrochemical Facilities* [12]. The usual approach is to convert the short-duration action into an equivalent static load and design the reinforced concrete for a quasi-static load in the normal way. The course of the pressure wave is normally represented by a simple isosceles triangle, but in some cases is defined as a pressure phase followed by a suction phase. Fig. 6.16 shows a typical pressure wave diagram.

The second abnormal load scenario requires the tank to be designed to resist “flying objects”, i.e. the impact of debris thrown into the air by an explosion. As a

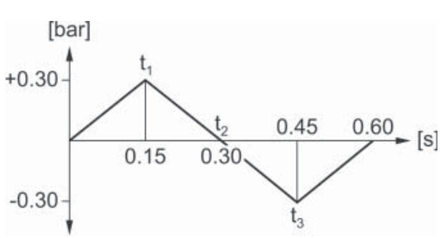


Fig. 6.16 Example of a blast pressure wave development.

starting point, EEMUA 147 specifies a projectile diameter of 4 in. (100 mm) and a velocity of 160 km/h (45 m/s), whereas the old BS 7777-1 [11] states in a note that it is reasonable to consider a valve with a weight of 50 kg travelling at a speed of 45 m/s. EN 14620 does not contain any information on or definitions of impact actions.

Likewise, although section 5.1.14 of ACI 376 [13] is entitled "Explosion and Impact", and thus illustrates how these two load cases should be considered together, no information is provided regarding the magnitude of the actions. However, section 8.5 of the American code does include an empirical formula for determining the required thickness of the concrete section. This equation originally appeared in ACI 349.

$$v^2 = C \cdot f'_c \cdot w^{\frac{1}{3}} \cdot \left[D_p \frac{h^2}{m_p}\right]^{\frac{4}{3}} \tag{6.1}$$

where:

C empirical value (= 1.89)
w concrete density [kg/m^3]
f_c cylinder compressive strength [N/m^2]
h cross-section thickness [m]
D_p projectile diameter [m]
m_p mass [kg]

In Europe the following equation is normally used. It comes from CEB 187 [14] and is based on more than 300 tests that were conducted in the UK. It also takes account of the amount of reinforcement.

$$v_c = 1.3 \cdot \rho^{\frac{1}{6}} \cdot \sqrt{f_{cyl}} \cdot \left[\frac{p \cdot e^2}{\Pi \cdot m}\right]^{\frac{2}{3}} \cdot \sqrt{r + 0.3} \tag{6.2}$$

where:

ρ concrete density [kg/m^3]
f_{cyl} cylinder compressive strength [N/m^2]
e cross-section thickness [m]

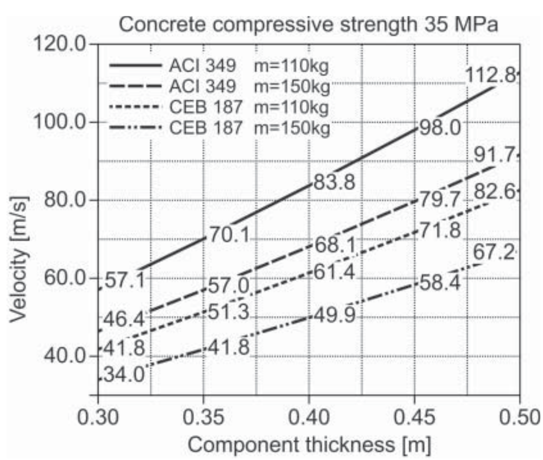


Fig. 6.17 Comparison of results obtained with CEB 187 and ACI 349.

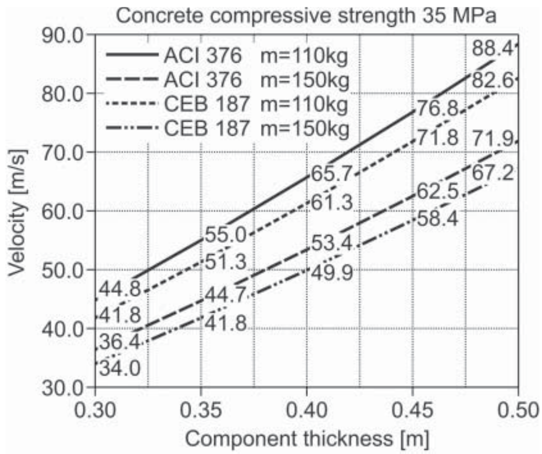


Fig. 6.18 Comparison of results obtained with CEB 187 and ACI 376.

p projectile diameter [m]
m mass [kg]
r amount of reinforcement [%]

Compared with Eq. (6.2) (CEB 187), using Eq. (6.1) (ACI 349) results in the assumption of faster speeds, or rather, thinner cross-sections. In order to obtain better agreement between the results of these two equations, it was decided to specify an additional cross-section reserve of 20% when drawing up ACI 376. Fig. 6.17 compares the results according to CEB 187 and ACI 349, and Fig. 6.18 shows the adjusted figures taking into account the modified equation in ACI 376.

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7

Dynamic Analysis

7.1 Theory of Sloshing Fluid

Regulations and standards exist for analysing tanks subjected to seismic actions. However, carrying out the right, appropriate analysis is relatively difficult and presumes that the designer is experienced and fully familiar with the procedures. This has partly to do with the complicated theory and partly with the complex mathematics. To help designers gain a better understanding of this topic, the principles of the theory of sloshing fluid and its history will be briefly outlined here.

During an earthquake, the tank structure and its liquid contents undergo acceleration. A hydrodynamic pressure acts on the tank structure and changes its deformation, which in turn has an influence on the pressure. There is no self-contained analytical solution for the ensuing fluid-structure interaction. The formulation of these relationships is demanding and time-consuming. Therefore, approximation methods were developed for practical applications at a very early stage. Fundamental work on this was carried out by Housner in the late 1950s and by Veletsos 20 years later.

Annex A of EN 1998-4 is explained after describing the Housner and Veletsos methods. The Eurocode method requires the designer to determine series expansions of the Bessel function in order to calculate the changing hydrodynamic pressure. To simplify this, evaluations have been carried out for LNG tanks with customary slenderness ratios ($\gamma = H/R$) between 0.60 and 1.20 and the results presented in tabular and graphical form as prefactors.

The hydrodynamic pressure can be divided into three parts that are calculated separately and subsequently superimposed. Fig. 7.1 is a qualitative presentation of the pressures acting on a tank wall:

- rigid impulsive pressure $p_{i,r}$,
- flexible impulsive pressure $p_{i,f}$, and
- convective pressure p_c.

If a tank filled with a liquid is excited horizontally, the inertia of the liquid has an influence on the deformation of the tank wall and, as a result, generates the so-called impulsive pressure p_i. Owing to the horizontal excitation, the surface of the liquid sloshes around. This change in the level of the liquid generates the so-called convective (sloshing) pressure p_c [1]. The impulsive pressure p_i is

Design and Construction of LNG Storage Tanks, First Edition. Josef Rötzer.

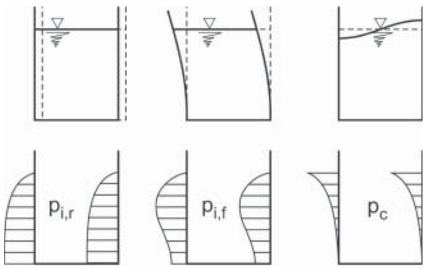

Fig. 7.1 Qualitative presentation of rigid impulsive, flexible impulsive and convective pressure distributions on a tank wall after [1].

divided into two parts: the pressure on a rigid wall $p_{\mathrm{i,r}}$ and the pressure $p_{\mathrm{i,f}}$ that ensues additionally in a flexible, deforming wall.

Tank oscillation periods usually lie between 0.4 and 0.6 s, whereas the oscillation period of the sloshing motion is about 10 s. As these two periods are very different, the fluid-structure interaction can be neglected when considering the convective pressure and the designer can assume a rigid tank.

The first theories describing the behaviour of the liquid were published by Housner in the late 1950s [2, 3]. Housner also investigated flexible tank walls in [2]. The article [4] and specification [5] based on his theory only make use of the results for rigid tanks. Despite this limitation, the significance of dividing the mass into impulsive and sloshing components and the ensuing base shear is considerable.

The work of Veletsos at Rice University Houston brought further developments more than 10 years later [6–8], see Fig. 7.2. Veletsos derived the relationships using a vertical, cylindrical tank, and employed a system of cylindrical coordinates for his equations. In order to consider the deformation of the tank walls, he therefore used three functions:

– $\psi_{\mathrm{A}}(z) = \sin(\pi/2 \cdot z/H)$,
– $\psi_{\mathrm{B}}(z) = (z/H)$, and
– $\psi_{\mathrm{C}}(z) = 1 - \cos(\pi/2 \cdot z/H)$.

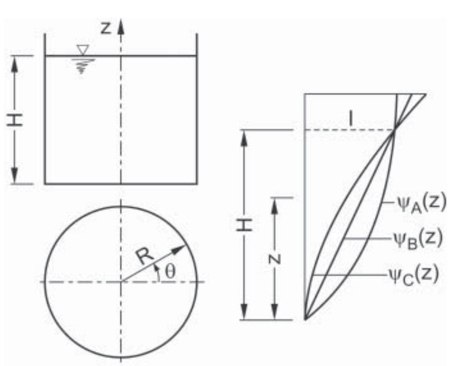

Fig. 7.2 Veletsos' deformation assumptions.

7.2 Housner's Method

Housner examined tanks with two-fold symmetry and a horizontal base and rigid wall. He developed an approximation method for calculating the hydrodynamic pressure for a container filled with a liquid which undergoes a horizontal acceleration. To do this, he derived relationships to describe the impulsive and convective components of the liquid. Figs. 7.3 and 7.4 and the designations used in those figures have been taken from TID 7024, appendix F, where Housner describes his theory. TID 7024 [9] is available online.

In TID 7024 he expanded the work of his earlier publications and corrected typos in a number of expressions. The impulsive and convective pressures are examined separately. Impulsive pressure occurs due to the inertia forces generated by the accelerations of the container walls. The forces are directly proportional to the accelerations. Convective pressure, on the other hand, is generated by the oscillations of the liquid. The liquid is assumed to be incompressible and the displacements of the liquid small.

Housner's work was based on the findings of Jacobsen [10], who proved that for rectangular and circular tanks, the accelerations in a vertical plane of symmetry (x-y plane) do not cause any accelerations in a direction perpendicular to that vertical plane (z plane). In his model of the impulsive component, he idealised the fluid as several independent, rigid, vertical membranes. He set up the equilibrium condition for an element cut out from one membrane and in doing so took into

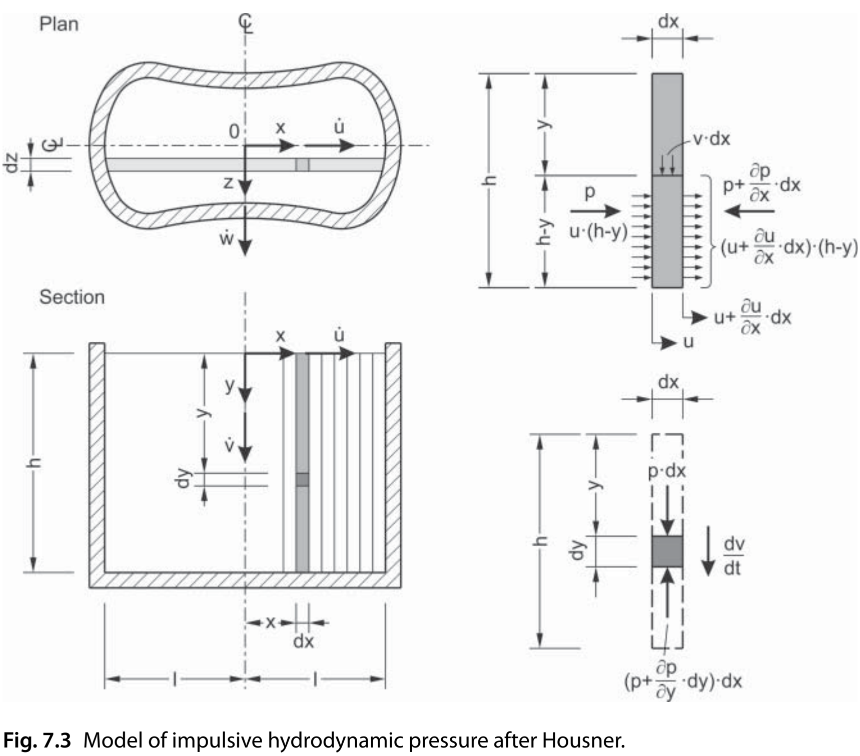

Fig. 7.3 Model of impulsive hydrodynamic pressure after Housner.

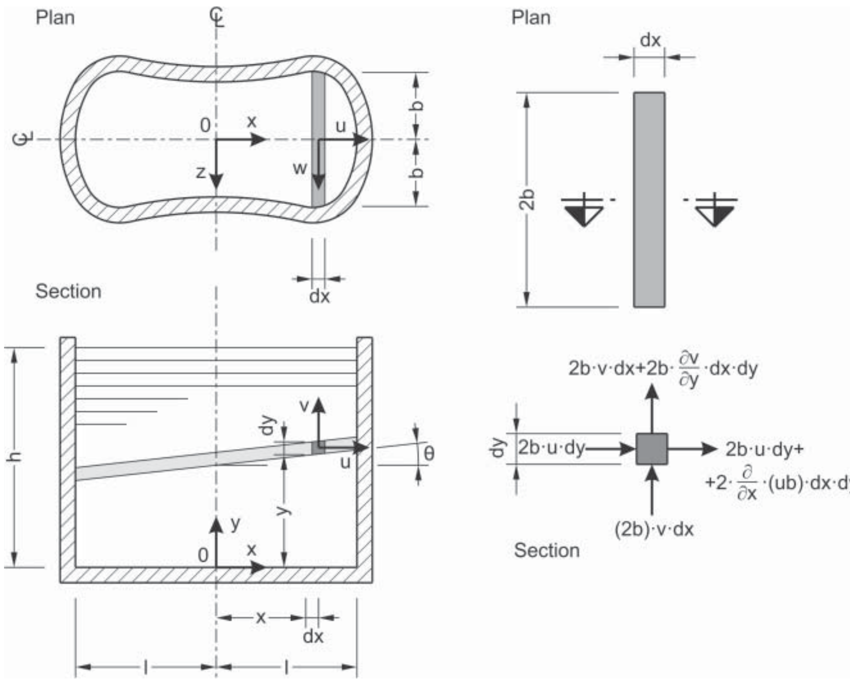

Fig. 7.4 Model of convective hydrodynamic pressure after Housner.

account the inertia forces. He used the following approach to solve the ensuing differential equation:

$$u' = C_1 \sin h\sqrt{3}\frac{x}{h} + C_2 \cos\ h\sqrt{3}\frac{x}{h} \tag{7.1}$$

Oscillations are caused in a liquid itself if the walls of its containment structure are excited by accelerations. It is this motion that generates pressure on the walls and base of the containment structure.

To derive the equations for the convective component, he considered the first eigenmode of the sloshing motion. He idealised the fluid as a horizontal, rigid membrane that can rotate about its horizontal axis, then determined the basic equations by examining the mass and continuity relationships on an element $dx \cdot dy$ cut out of the whole. These were integrated and solved for the displacements u and w plus rotation θ by means of the boundary conditions.

The equations were derived for a general tank cross-section with two-fold symmetry and subsequently simplified for circular and rectangular cross-sections.

Housner's method resulted in the following equations for the impulsive and convective hydrodynamic pressure on the wall and the mass relationships:

$$p_{w,i} = \rho \dot{u}_0 h \left[\frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2\right] \sqrt{3} \tan\ h\left(\sqrt{3}\frac{R}{h}\cos\phi\right) \tag{7.2}$$

$$\frac{m_i}{m} = \frac{\tan\ h\left(\sqrt{3}\frac{R}{h}\right)}{\sqrt{3}\frac{R}{h}} \tag{7.3}$$

$$p_{w,c} = \sqrt{\frac{3}{8}}\rho R^2 \Theta_h \left[\frac{x}{R} - \frac{1}{3}\left(\frac{x}{R}\right)^3 - \frac{1}{2}\frac{x}{R}\left(\frac{z}{R}\right)^2\right] \frac{\cos h\sqrt{\frac{27}{8}\frac{y}{R}}}{\sin h\sqrt{\frac{27}{8}\frac{h}{R}}} \tag{7.4}$$

$$\frac{m_c}{m} = 0.318\frac{R}{h} \cdot \tan h\sqrt{\frac{27}{8}\frac{h}{R}} \tag{7.5}$$

Housner's results for the hydrodynamic pressure for the impulsive, rigid and convective components are used in many American standards, e.g. API 620, API 650 and ACI 350. The factor in the equation for determining the convective mass (7.5) component has been changed later from 0.318 to 0.420.

7.3 Veletsos' Method

Veletsos' approach uses two systems of coordinates whose origins lie at the centre of the base of the tank. The first system is a global Cartesian system of coordinates with its x-y plane in the base of the tank and the z axis in the upward plane of symmetry. It is used to describe the seismic action and the rigid-body displacement. The tank and its relative displacements are described by a system of cylindrical coordinates, likewise with its origin at the centre of the tank base and the z axis extending upwards (Fig. 7.5). Two different relationships are used in this system of cylindrical coordinates, one with absolute magnitudes and another with cylindrical unit coordinates where $\zeta = z/H$ and $\xi = r/R$.

Veletsos derived the hydrodynamic pressure from the potential flow of the liquid taking into account the boundary conditions. Eq. (7.6) expresses the motion

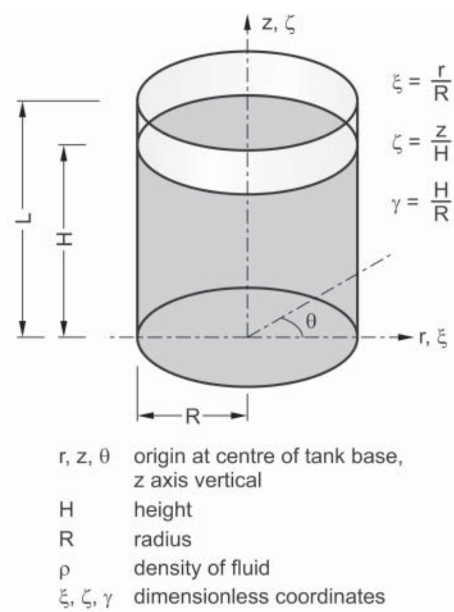

Fig. 7.5 Veletsos' model and designations.

of the liquid in terms of global cylindrical coordinates. An incompressible, frictionless liquid is presumed.

$$\frac{\partial^2\Phi}{\partial r^2} + \frac{1}{r}\frac{\partial\Phi}{\partial r^2} + \frac{1}{r^2}\frac{\partial^2\Phi}{\partial\theta} + \frac{\partial^2\Phi}{\partial z^2} = 0 \tag{7.6}$$

where:

Φ velocity potential
t time
r, θ, z local coordinates

To solve the differential equation for the potential flow, it is necessary to define the boundary conditions and hence solve the equation. The boundary conditions result from the accordance between the velocities at the boundaries of the container, or rather, liquid. In order to be able to solve the differential equation, the velocity potential is divided into three sub-potentials and solved for each of the specific boundary conditions. The sub-potentials represent a rigid displacement of the tank (impulsive pressure component), a flexible deformation of the tank wall (flexible impulsive pressure component) and a rigid displacement of the tank with deformation of the surface of the liquid (convective pressure component).

$$\Phi = \Phi_1 + \Phi_2 + \Phi_3 \tag{7.7}$$

Besides the division into three sub-potentials, a separable partial differential equation was chosen to describe the potential so that each factor is then dependent on a single variable only.

$$\Phi = R(r) \cdot Z(z) \cdot \Theta(\theta) \cdot T(t)$$

or

$$\Phi = R(\xi) \cdot Z(\zeta) \cdot \Theta(\theta) \cdot T(t) \tag{7.8}$$

It is also assumed that the velocity potential is symmetrical and there are no deviations from a perfectly cylindrical shell, which simplifies the term $\Theta(\theta)$. Two ordinary decoupled differential equations remain for $Z(\zeta)$ and $R(\xi)$. The reader should consult [11] and [1] for more detailed information.

$$\frac{1}{Z(\zeta)}\frac{d^2Z(\zeta)}{d\zeta^2} = \lambda^2 \tag{7.9}$$

$$\xi^2\frac{d^2R(\xi)}{d\xi^2} + \xi\frac{dR(\xi)}{d\xi} + (\lambda^2\xi^2 - m^2)\,P(\xi) = 0 \tag{7.10}$$

Eq. (7.10) is called Bessel's differential equation. The solutions to this equation are called Bessel functions or cylinder functions. Note that I_1, I'_1 denote the Bessel function of the first kind and its derivative, and J_1, J'_1 the modified Bessel function of the first kind and its derivative.

7.4 Provisions in EN 1998-4, Annex A

Annex A of EN 1998-4 supplies two important details. Firstly, the pressure acting on the tank wall and tank base – values that are required to design the steel inner container. Secondly, the division of the mass of the liquid into impulsive and sloshing components and the associated lever arms for the calculations performed on an equivalent dynamic system. The calculation methods for tank design are not presented sufficiently well and using annex A is time-consuming and prone to errors. One reason for this is the erroneous designations, another the challenging mathematical expressions, which are based on Veletsos' theory and require a series expansion with Bessel functions in order to solve the differential equations. The term in the equation that contains this series expansion has been condensed in the factor C and evaluated for various geometries.

7.4.1 Hydrodynamic Pressure on Tank

The rigid impulsive pressure component is calculated with Eq. (7.11).

$$p_i(\xi, \zeta, \Theta, t) = C_i(\xi, \zeta)\rho R\ \cos\theta\, A_g(t) \tag{7.11}$$

where:

ρ density of liquid
θ circumferential angle in system of cylindrical coordinates
$A_g(t)$ maximum value of free-field acceleration

$$C_i(\xi, \zeta) = 2\,\gamma \sum_{n=0}^{\infty} \frac{(-1)^n}{\nu_n^2\ I_1'\ (\nu_n/\gamma)} \cdot \cos\left(\nu_n\ \zeta\right) \cdot I_1\ \left(\frac{\nu_n}{\gamma}\ \xi\right) \tag{7.12}$$

$$\nu_n = \frac{2n+1}{2}\,\Pi \qquad\qquad \gamma = \frac{H}{R} \tag{7.13}$$

In annex A of EN 1998-4 the equation for p_i includes the height H ($H = \gamma \cdot R$). In most German-language publications, and in this text, too, the equation includes the radius R. The missing tank slenderness ratio γ is included in factor C_i. The equation for C_i supplies the pressure coefficients for the tank base for $\zeta = 0$ ($z = 0$) and the values at the tank wall for $\xi = 1$ ($r = R$).

The convective pressure component for the first eigenmode is given by Eq. (7.14):

$$p_{c,1}(\xi, \zeta, \Theta, t) = C_{c,1}(\xi, \zeta)\rho R\ \cos\Theta\ A_{c,1} \tag{7.14}$$

$$C_{c,1}(\xi, \zeta) = \frac{2\cos h(\lambda_1\gamma\zeta)\, J_1(\lambda_1\xi)}{(\lambda_1^2 - 1)\ \cos h(\lambda_1\gamma) J_1(\lambda_1)} \tag{7.15}$$

If $i = 1$ is used for the first eigenmode and $r = R$ ($\xi = 1$) for the tank wall, the equation simplifies to

$$C_{c,1}(\xi = 1, \zeta) = 0.837 \frac{\cos h(\lambda_1\gamma\zeta)}{\cos h(\lambda_1\gamma)} \tag{7.16}$$

In Eq. (7.14) S_a is the spectral acceleration, i.e. the acceleration that results from the spectrum for the respective period of natural oscillation. The period of natural oscillation is given by the following equation:

$$T_{cn} = \frac{2\Pi}{\sqrt{g\frac{\lambda_n}{R}\tan\ h(\lambda_n\gamma)}} \tag{7.17}$$

Normally, only the first eigenmode is considered. Therefore, with $n = 1$ and $\lambda_1 = 1.8412$, the equation simplifies to

$$T_{c1} = 1.4784\sqrt{\frac{R}{\tan h(\lambda_1 \gamma)}} \tag{7.18}$$

The hydrodynamic pressures on the tank wall have been evaluated for the customary geometries of LNG storage tanks, and these are given in Tables 7.1 and 7.2 as well as Figs. 7.6 and 7.7.

Table 7.1 Tables of C_i coefficients for rigid impulsive pressure.

a) on tank wall

ζ = z/H	γ = H/R						
	0,60	**0,70**	**0,80**	**0,90**	**1,00**	**1,10**	**1,20**
1,00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.90	0.1460	0.1695	0.1917	0.2130	0.2330	0.2519	0.2698
0.80	0.2368	0.2744	0.3099	0.3425	0.3731	0.4014	0.4277
0.70	0.3064	0.3543	0.3987	0.4397	0.4770	0.5112	0.5423
0.60	0.3609	0.4167	0.4680	0.5145	0.5565	0.5942	0.6280
0.50	0.4041	0.4659	0.5223	0.5728	0.6178	0.6576	0.6928
0.40	0.4377	0.5040	0.5640	0.6174	0.6644	0.7054	0.7412
0.30	0.4628	0.5324	0.5950	0.6503	0.6986	0.7403	0.7762
0.20	0.4803	0.5522	0.6166	0.6731	0.7221	0.7642	0.8000
0.10	0.4906	0.5638	0.6292	0.6864	0.7358	0.7779	0.8137
0.00	0.4940	0.5677	0.6334	0.6908	0.7403	0.7825	0.8182

b) on base slab

ξ = r/R	γ = H/R						
	0.60	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
1.00	0.4940	0.5677	0.6334	0.6908	0.7403	0.7825	0.8182
0.90	0.4010	0.4733	0.5380	0.5947	0.6435	0.6852	0.7204
0.80	0.3216	0.3903	0.4520	0.5062	0.5531	0.5931	0.6271
0.70	0.2549	0.3178	0.3748	0.4252	0.4688	0.5062	0.5379
0.60	0.1994	0.2549	0.3058	0.3510	0.3903	0.4241	0.4529
0.50	0.1531	0.2002	0.2438	0.2829	0.3170	0.3464	0.3714
0.40	0.1142	0.1523	0.1879	0.2200	0.2481	0.2724	0.2931
0.30	0.0810	0.1097	0.1368	0.1613	0.1828	0.2015	0.2175
0.20	0.0518	0.0710	0.0892	0.1057	0.1204	0.1330	0.1438
0.10	0.0253	0.0349	0.0440	0.0523	0.0597	0.0661	0.0716
0.00	0.000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Table 7.2 Tables of C_c coefficients for convective pressure.

a) on tank wall

$\zeta = z/H$	$\gamma = H/R$						
	0.60	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
1.00	0.8368	0.8368	0.8368	0.8368	0.8368	0.8368	0.8367
0.90	0.7676	0.7509	0.7346	0.7188	0.7037	0.6893	0.6754
0.80	0.7078	0.6775	0.6483	0.6206	0.5945	0.5700	0.5470
0.70	0.6566	0.6153	0.5761	0.5394	0.5055	0.4743	0.4455
0.60	0.6135	0.5634	0.5164	0.4731	0.4337	0.3981	0.3658
0.50	0.5778	0.5208	0.4679	0.4199	0.3767	0.3382	0.3040
0.40	0.5492	0.4869	0.4297	0.3781	0.3324	0.2923	0.2571
0.30	0.5273	0.4611	0.4007	0.3468	0.2992	0.2584	0.2229
0.20	0.5119	0.4430	0.3805	0.3250	0.2767	0.2351	0.1995
0.10	0.5027	0.4323	0.3685	0.3122	0.2634	0.2215	0.1860
0.00	0.4997	0.4287	0.3645	0.3080	0.2590	0.2171	0.1815

b) on base slab

$\xi = r/R$	$\gamma = H/R$						
	0.60	**0.70**	**0.80**	**0.90**	**1.00**	**1.10**	**1.20**
1.00	0.4997	0.4287	0.3645	0.3080	0.2590	0.2171	0.1815
0.90	0.4937	0.4236	0.3602	0.3043	0.2559	0.2145	0.1793
0.80	0.4757	0.4082	0.3471	0.2932	0.2466	0.2067	0.1728
0.70	0.4462	0.3828	0.3255	0.2750	0.2312	0.1938	0.1621
0.60	0.4055	0.3479	0.2959	0.2500	0.2102	0.1762	0.1473
0.50	0.3548	0.3044	0.2589	0.2187	0.1839	0.1542	0.1289
0.40	0.2952	0.2533	0.2154	0.1820	0.1530	0.1283	0.1073
0.30	0.2282	0.1958	0.1665	0.1407	0.1183	0.0991	0.0829
0.20	0.1554	0.1334	0.1134	0.0958	0.0806	0.0675	0.0565
0.10	0.0787	0.0675	0.0574	0.0485	0.0408	0.0342	0.0286
0.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

To illustrate the influence of the flexible pressure component, the pressures have been evaluated for the assumed linear deformation of the tank wall ($\psi_B(z) = z/H$), and the pressure coefficients $C_{i,f}$ are shown in Fig. 7.8.

7.4.2 Masses and Associated Lever Arms

In order to analyse the whole system, it is necessary to determine the impulsive and sloshing mass components and their associated lever arms. The internal forces below the tank foundation are designated base shear and base moment.

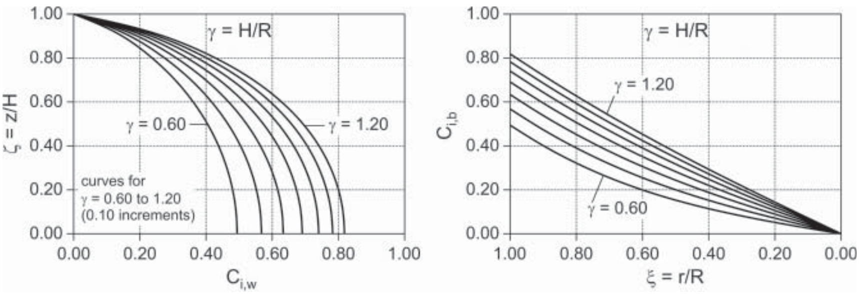

Fig. 7.6 Rigid impulsive pressure component acting on tank wall and base slab.

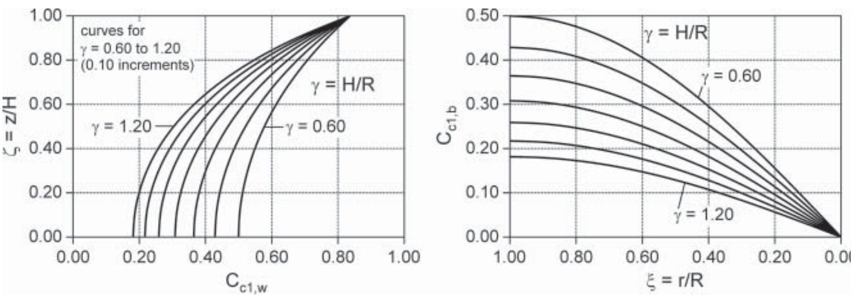

Fig. 7.7 Convective pressure component acting on tank wall and base slab.

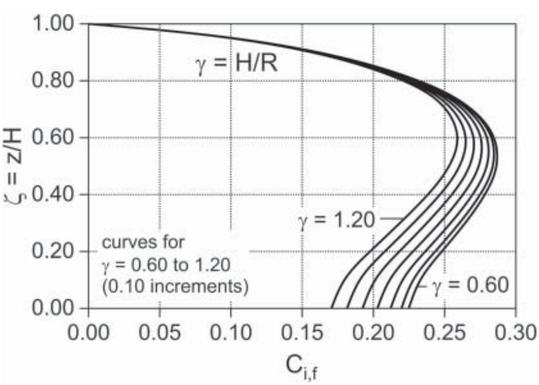

Fig. 7.8 Flexible impulsive pressure component acting on tank wall.

The former is the product of impulsive or convective mass and the free-field or spectral acceleration, the latter obtained by multiplying the base shear by the associated lever arm.

The equations for determining the masses and lever arms are given in annex A of EN 1998-4. Eqs. (7.19) and (7.20) are used to calculate the masses. Fig. 7.9 compares the results with the masses calculated according to the API publication [4]. There is very good agreement between the results for the slenderness ratios of 0.5 to 1.2 customary for LNG storage tanks.

$$\frac{m_i}{m} = 2\,\gamma \sum_{n=0}^{\infty} \frac{I_1\,(\nu_n/\gamma)}{\nu_n^3\, I_1'\,(\nu_n/\gamma)} \tag{7.19}$$

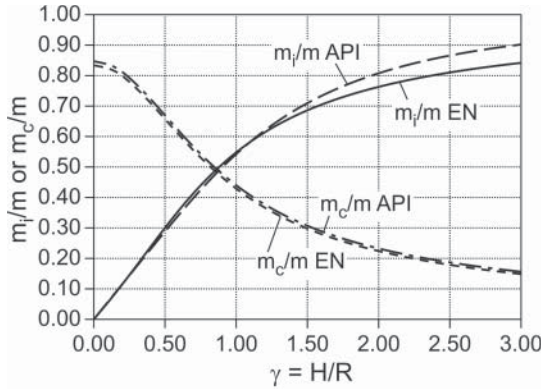

Fig. 7.9 Graphic presentation of impulsive and convective mass components according to EN 1998-4 and API.

$$\frac{m_{c1}}{m} = \frac{2\tan h(\lambda_1\gamma)}{\gamma\lambda_1(\lambda_1^2 - 1)} \tag{7.20}$$

The impulsive and convective lever arms are calculated with Eqs. (7.21) and (7.22):

$$\frac{h_i'}{H} = \frac{\frac{1}{2} + 2\,\gamma \displaystyle\sum_{n=0}^{\infty} \frac{\nu_n + 2\ (-1)^{n+1}}{\nu_n^4} \cdot \frac{I_1\ (\nu_n/\gamma)}{I_1'\ (\nu_n/\gamma)}}{2\,\gamma \displaystyle\sum_{n=0}^{\infty} \frac{I_1\ (\nu_n/\gamma)}{\nu_n^3 \cdot I_1'\ (\nu_n/\gamma)}} \tag{7.21}$$

$$\frac{h_{c1}'}{H} = 1 + \frac{2 - \cos h(\lambda_1\gamma)}{\lambda_1\gamma \sin h(\lambda_1\gamma)} \tag{7.22}$$

Again, Fig. 7.10 compares the results with the lever arms according to API. The results for the convective lever arm according EN and API codes are almost identical. While the results for the impulsive lever arm above base plate are close together, they differ from each other for lever arms below base plate. For a H/R ratio of 1.0, the deviation amounts to 10%.

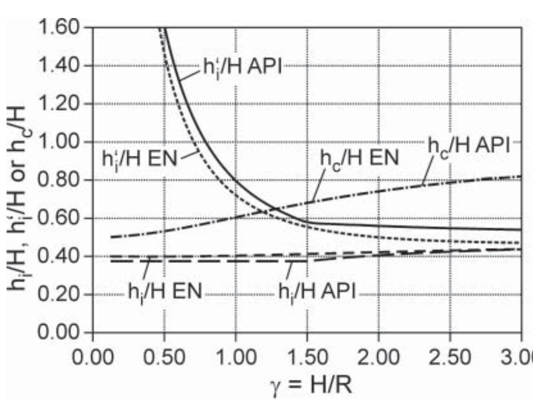

Fig. 7.10 Graphic presentation of impulsive and convective lever arms according to EN 1998-4 and API.

7.5 Seismic Design of LNG Tanks

The provisions in the EN 1998 series apply to engineered structures in earthquake regions. In the event of an earthquake, lives must be protected and damage limited. In addition, for important structures (e.g. hospitals), it is also necessary to ensure that their functionality is still guaranteed. Special structures that pose potential risks during or after an earthquake (e.g. nuclear power stations, offshore structures or large dams) are not covered by EN 1998. In terms of dynamics, liquid-retaining structures are very different to other engineered structures. Therefore, a number of aspects must be considered in their design. During an earthquake, the liquid contents represent a huge mass that is excited, and even if the accelerations are only moderate, huge forces still ensue. Owing to the properties of the stored products and the potential risks associated with them, it is necessary to comply with regulations that go beyond EN 1998-1 and EN 1998-4 when designing tanks for refrigerated liquefied gases. The actions are defined in EN 14620-1, section 7.3. Nevertheless, the provisions and methods of calculation given in EN 1998-1 still apply (Table 7.3).

EN 1998-1 distinguishes between requirements concerning stability (no-collapse requirement, NCR) and requirements concerning limits to damage (damage limitation requirement, DLR).

Table 7.3 Definitions of various design earthquakes.

Standard	Earthquake designation	Abbreviation	Reference return period T_L [years]	Probability of exceedance p_R [%]	Mean return period T_R [years]	Importance factor γ_I
EN 1998-4	damage limitation requirement	DLR	10	10	95	0.8
	no-collapse requirement	NCR	50	10	475	1.0
EN 14620-1	operating basis earthquake	OBE	50	10	475	1.0
	safe shutdown earthquake	SSE	50	1	4975	2.2
NFPA 59A	operating basis earthquake	OBE	50	10	475	1.0
	safe shutdown earthquake	SSE	50	2	2475	1.7

The spectra given in EN 1998-1 are based on the NCR and include a return period of 475 years. In order to be able to take into account the different significance of different structures, the standard defines importance classes I to IV and assigns them importance factors of 0.80 to 1.40.

For the damage limitation state, applying a 10% probability of being exceeded and a reference return period of 10 years results in a return period T_{DLR} = 95 years. For the NCR, applying a 10% probability of being exceeded and a reference return period of 50 years results in a return period T_{NCR} = 475 years. This corresponds to importance class II in EN 1998-1 and an importance factor of 1.0.

The equation given in section 2.1(4) of EN 1998-1, which is based on a Poisson distribution, should be used to convert to other probabilities of exceedance.

$$p_R = 1 - e^{-T_L/T_R} \tag{7.23}$$

$$\gamma_I = \frac{1}{(P_L/P_{LR})^{1/3}} \tag{7.24}$$

where:

P_R probability of exceedance in T_L years
T_L years with a certain level
T_R mean return period
T_{LR} reference return period
k exponent (to EN 1998-1, $k = 3$)

which results in

$$\gamma_{I,4975} = \frac{1}{(475/4975)^{1/3}} = 2.2 \tag{7.25}$$

As already mentioned, the seismic design of LNG tanks should include a much higher level of safety than for customary structures designed according to EN 1998-1. All the standards worldwide take into account two different earthquakes:

The operating basis earthquake (OBE) is defined as the "maximum earthquake event for which no damage is sustained and restart and safe operation can continue" [12]. The definition of the OBE – assuming 5% damped response spectra and a 10% probability of being exceeded within a 50-year period – corresponds to a return period of 475 years.

The safe shutdown earthquake (SSE) is defined as the "maximum earthquake event for which the essential fail-safe functions and mechanisms are designed to be preserved". Permanent damage is acceptable, provided the containment is not affected [12]. In Europe the SSE – assuming 5% damped response spectra and a 1% probability of being exceeded within a 50-year period – corresponds to a return period of 4975 years, which in turn, according to Eq. (7.25), corresponds to an importance factor of 2.2. The American codes define a 2% probability of being exceeded and hence a return period of 2475 years.

When analysing liquid-retaining structures for seismic actions, it is essential to consider the interaction between the liquid and the containment structure as well as the interaction between the tank foundation and the subsoil. Discrete modelling of the liquid is inappropriate for practical requirements. The simplest approximation methods are equivalent mechanical models that include only one mass element to take account of the liquid (compared with the mass of the liquid, the mass of the steel tank can be neglected) and one further mass element to take account of the tank and the subsoil. The fluid-structure-soil interaction is modelled with spring and damping elements. These two-degree-of-freedom oscillators represent one way of ascertaining the dynamic system behaviour with only a small amount of work. They are sufficiently accurate to specify the tank geometry and freeboard for preliminary design purposes.

EN 1998-1 calls for the distribution of stiffness and mass to be included when modelling the structure, also the strength distribution when using non-linear calculations, and to take into account cracking when determining the stiffness of a reinforced concrete structure. The standard specifies four methods of analysis for its scope of application which are repeated in EN 1998-4 (silos, tanks and pipelines).

Those four methods of analysis comprise two linear-elastic methods – the simplified response spectrum method and the multi-modal response spectrum method – and two non-linear methods – non-linear static (pushover) analysis and non-linear time history (dynamic) analysis. The simplified response spectrum method uses the first eigenmode only. It can be used for those structures in which the response to seismic action of every component can be described approximately by a single-degree-of-freedom oscillator. In the multi-modal response spectrum method, the analysis is based on several eigenmodes. All crucial modal forms are deemed to have been included when all eigenmodes with a modal mass > 5% of the total mass have been considered or when the sum of the modal masses considered contains at least 90% of the total mass of the structure.

When it comes to the final calculations, it will be necessary to carry out more detailed modelling that reflects the individual masses and stiffnesses. The magnitudes and distributions of the hydrodynamic pressures can be calculated according to EN 1998-4, annex A. Fig. 7.11 shows one example of such a model. When using this model, 99% of the total mass is considered. The subsoil springs are determined with dynamic subsoil parameters. Table 7.4 lists the equivalent spring constants [13].

The dynamic loading is defined by means of spectra for OBE and SSE, which must be specified by the tank owner. The modal damping ratios can be determined through a weighted superposition of the damping ratios of the individual elements. The ratio of the deformation energy stored in the respective element (subsoil springs, sloshing spring, inner and outer containers) to the deformation energy of the total system can be used as the weighting factor. The damping ratios can therefore be calculated for each eigenmode with the following equation [14]:

$$\xi_i = \sum \frac{E_{k,i}}{E_{tot,i}} \xi_{e,k} = \frac{\sum E_{k,i} \cdot \xi_{e,k}}{E_{tot,i}} = \frac{\sum \phi_{k,i}^T \cdot K_e \cdot \phi_{k,i}}{\phi_i^T \cdot K \cdot \phi_i} \tag{7.26}$$

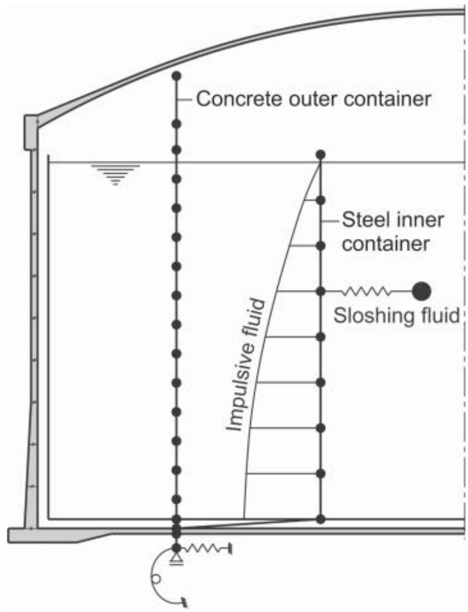

Fig. 7.11 Dynamic system for horizontal excitation.

Table 7.4 Equivalent spring constants.

Type	Vertical oscillation	Horizontal oscillation	Rocking oscillation	Torsional oscillation
Stiffness k	$\frac{4\,\mathrm{Gr}}{1-\nu}$	$\frac{8\,\mathrm{Gr}}{2-\nu}$	$\frac{8\,\mathrm{Gr}^3}{3(1-\nu)}$	$\frac{16\,\mathrm{Gr}^3}{3}$

Note: G = shear modulus; r = radius of base slab; ν = Poisson's ratio of soil

where:

ξ_i damping ratio for eigenmode i
$E_{k,i}$ deformation energy in element k
$E_{tot,i}$ deformation energy in total system
$\zeta_{e,k}$ damping ratio for element k
K system stiffness matrix
K_e stiffness matrix for element k
ϕ_i displacement of total system in eigenmode i
$\phi_{k,i}$ displacement of element k in eigenmode i

The damping ratios calculated in this way are limited for the remainder of the application in order to avoid deviating too much from the eigenmodes and eigenfrequencies of the undamped system. This is achieved by specifying a maximum damping increment of 0.70 for OBE and 0.63 for SSE, which correspond to damping ratios of 15% and 20% respectively (see Table 7.5).

Table 7.5 Damping ratios.

Component/Material	Damping OBE [%]	Damping SSE [%]
Sloshing fluid	0.5	0.5
Impulsive fluid	2.0	4.0–5.0
Steel tank	2.0	4.0–5.0
Concrete tank	2.0	5.0
Soil	< 15	< 20

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8

Construction

8.1 Construction Phases and Procedures

The steel inner and concrete outer containers are constructed by highly specialised, but usually different, contractors. Various construction options and approaches are possible for the steel and concrete containers. One of the consequences of this is that coordinating the operations on site between the two contractors becomes particularly significant, and involves much more work in the case of a full containment tank than for the other, simpler tank systems. The commissioning date for the plant is laid down many years in advance and agreed with gas suppliers, gas customers and shipping companies. Penalty clauses involving heavy fines for late completion are usually included in the contracts and this is why the timetable is normally given maximum priority.

8.1.1 Base Slab

Depending on the type of foundation, between 4000 and 7000 m^3 of concrete will be required for the base slab of a large LNG storage tank with a capacity of 180 000–200 000 m^3. Irrespective of whether the tank is supported on piles, with a more or less constant base slab depth, or on a raft foundation with a varying slab depth, the base slab is divided into inner and outer areas. Each of these two areas is further divided into several concrete pours. The layout of the construction joints is of course taken into account when planning the work on site, but also earlier, at the tendering stage and when drawing up the timetable. The layout and sizes of the concrete pours depend on the amount of concrete that can be placed in a day, the intended sequence of operations, optimisation of formwork and reinforcement and also structural considerations. With a batching plant on site that can produce up to 70 m^3/h, it is possible to place up to 1000 m^3 of concrete each day. On confined sites, the inner area is cast first so that it can be used afterwards for preparing and assembling the formwork. Also from the point of view of reducing the restraint stresses in the base slab, it is better to cast the inner area first.

However, in order to shorten the time needed to construct the outer container and be able to begin building the tank wall sooner, casting the perimeter of the base slab first is advantageous. Regardless of the sequence of concreting operations, it is expedient when all the reinforcement across the whole slab

Design and Construction of LNG Storage Tanks, First Edition. Josef Rötzer.

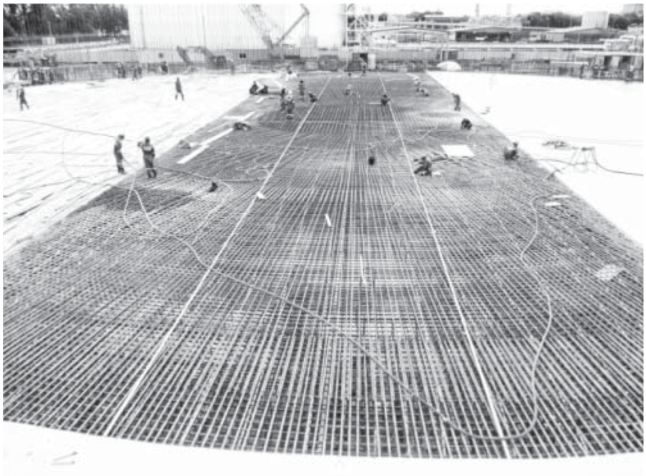

Fig. 8.1 Concrete pours for a tank base slab.

is laid prior to concreting (see Fig. 8.1). Most tank specifications call for construction joints in the wall to be avoided as far as possible. Apart from around temporary openings, vertical wall joints are explicitly not permitted, i.e. when using climbing formwork systems, each wall segment must be concreted in one pour as a complete ring.

8.1.2 Tank Wall

Climbing formwork and slipforming techniques are used to construct the tank wall. The decision regarding which method to use depends on the particular situation and must be weighed up very carefully.

Climbing formwork requires fewer preparations and a far less specialised, less experienced site crew. The work can be interrupted during unsuitable weather conditions or for other reasons. Cast-in parts, of which there are many for fixing the liner and other components, can be relatively easily and accurately fixed to the formwork. In terms of site operations, climbing formwork allows the concrete to be cast in segments, normally complete rings, and the use of prefabricated reinforcement meshes or complete reinforcement cages (Fig. 8.2). Such cages can be assembled with normal reinforcement on the outside, cryogenic reinforcement on the inside, shear stirrups and ducts for prestressing tendons. Climbing formwork is less sensitive to construction errors and tolerance discrepancies. After each concrete pour, the formwork can be realigned and adjusted. The concrete technology does not have to comply with any additional measures or requirements when using climbing formwork.

Slipforming, on the other hand, requires much more preparatory work and must be planned in far more detail. Furthermore, a specialised, experienced site crew is essential. No significant interruptions to the slipforming process are permitted, and the use of pre-assembled reinforcement is also impossible. This method calls for a greater quantity of reinforcement as moderate bond is

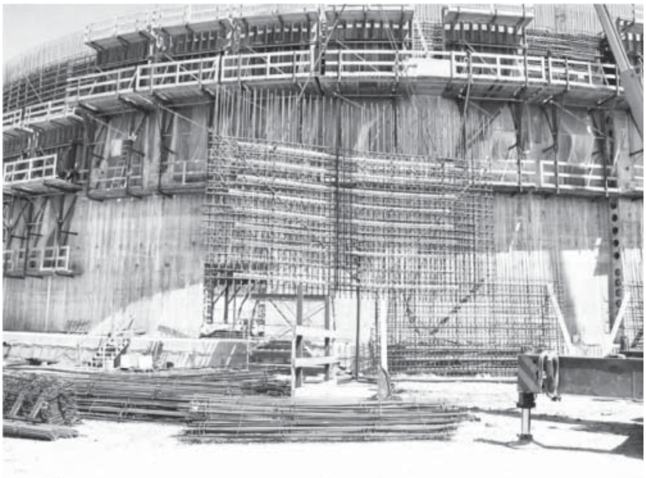

Fig. 8.2 A pre-assembled reinforcement cage being installed.

assumed and because – to simplify installing the reinforcement – the bars are shorter, which increases the number of bar laps. Slipforming is more sensitive to construction errors and tolerance discrepancies.

The formwork sheeting for slipforming is 1.0–1.30 m high. To ease the sliding motion and reduce the friction forces between the concrete and the sheeting, the sheeting widens out slightly in the direction of the set concrete. According to [1], the deviation compared with the intended wall thickness is −1 mm at the top and +3 mm at the bottom. Furthermore, a maximum aggregate size of 32 mm should be chosen in order to reduce the cement content and hence the adhesion tendency of the concrete. Sliding takes place at regular intervals in separate strokes of 20–25 mm each time. The concrete is cast in 20–25 cm layers over the entire circumference and compacted with poker vibrators. To simplify fixing the reinforcement, the horizontal reinforcing bars should be pre-bent, no longer than 10 m and no heavier than 25 kg. Vertical reinforcing bars should not be more than 5.50 m long [1].

The concrete mix and the setting behaviour must be adapted to the ambient temperature and the rate of slipforming. Suitable concrete mixes should be designed for different temperature ranges and different slipforming rates. At least one concrete mix should be specified for each of the day and night shifts as well as the thicker bottom and thinner top sections of the wall.

The rate at which the reinforcement can be fixed determines the speed of the slipforming operation. Progress depends on the amount of reinforcement required and, primarily, the proficiency of the steel-fixing crew, given the respective regional and climatic conditions. The following example serves to illustrate the interdependencies. In the example, the top and bottom parts of the wall, including the buttresses, are considered. The main parameters for this example are a tank diameter of 90 m and a steel-fixing crew of 60 (one person every 5 m) able to fix reinforcement at a rate of 50 kg/h (20 h/t).

	bottom	top
wall circumference	270 m	270 m
amount of concrete	310 m^3/m	200 m^3/m
reinforcement per unit volume	230 kg/m^3	150 kg/m^3
amount of reinforcement	72 t/m	30 t/m
steel-fixing rate	20 h/t	20 h/t
total time for fixing reinforcement	1440 h/m	600 h/t
total time per steel-fixer	24 h/m	10 h/m
slipforming rate	1.0 m/day	2.4 m/day

Based on the slipforming rate calculated, for a 40 m high wall, it will take about three weeks to complete the actual slipforming work, and the preparatory measures will occupy a further four to five weeks. When using climbing formwork, it will take about three weeks to cast the first complete wall ring (first concrete lift), albeit with much smaller crews for fixing the reinforcement and concreting. This is due to the greater quantity of reinforcement, the starter bars (sometimes in several layers), the more complicated fixing of reinforcement and a site crew that is not yet operating well together. Completing the 40 m high wall in 10 lifts will take five to six months (e.g. $3 + 3 + 3 + 7 \times 2$ weeks). In this example 60 operatives are required just to fix the formwork. In addition, about 15 others are needed to place and compact the concrete, and further personnel are required for cranes, concrete pumps and finishing work plus at least five specialists for the slipforming operation itself. So, in total, a workforce of at least 100 is required for each of two 12 h shifts. This example is intended to illustrate the limits to the possible applications for slipforming LNG tanks. The relationships with respect to geometry and amount of concrete and reinforcement are different to those that apply to silos, vessels and towers, for which slipforming is highly suitable.

The decision to use slipforming to build an LNG tank depends on each particular situation. The governing criterion is the rate at which the reinforcement can be installed, which, experience shows, can lie between 15 and 75 kg/h depending on the location of the site. The lower value seems very low, but it should be remembered that this is not really the performance per hour, but rather an average value taken over weeks and working a 12 h shift each day, and in some cases working in high ambient temperatures. The rate of 50 kg/h that applies in the above example tends to reflect European conditions. If the rate of fixing reinforcement falls too far below this figure, the boundary conditions for slipforming are no longer favourable.

The advantages of using climbing formwork are the higher quality of construction and the smaller tolerances and deformations. The advantages of slipforming, apart from the quicker completion, are the avoidance of all construction joints and foreign bodies or formwork in the wall. Its greatest advantage is the much shorter time taken to construct the tank wall. Some situations, e.g. cold regions with only a few months of moderate temperatures, can tip the decision in favour of slipforming.

8.1.3 Ring Beam

The ring beam or eaves beam at the top of the wall includes a cast-in intermediate steel element fitted between the cylindrical wall and the domed roof (see Fig. 8.3). This steel element (the compression ring), made from plates 20–30 mm thick, is prefabricated in segments welded together to form a ring. The steel roof is first assembled just above the base slab and then raised pneumatically by pumping air into the tank. Once in position, the roof is welded to the compression ring. Afterwards, the ring beam is cast with the necessary starter bars for the concrete of the tank roof.

8.1.4 Tank Roof

A concrete tank roof has to comply with a whole series of very diverse requirements. The design for abnormal external actions, i.e. blast pressure wave, fire and impact, call for a concrete dome instead of a steel roof. However, the full containment tank concept requires a vapour barrier on the underside of the concrete roof. In addition, the suspended aluminium deck of the inner container also has to be supported from the roof. A concrete outer container with an 80 m inside diameter and a roof thickness of 0.40 m in the middle requires more than 2500 m^3 of concrete for the roof, which means a self-weight of 6000 t that has to be

Fig. 8.3 Installing the compression ring.

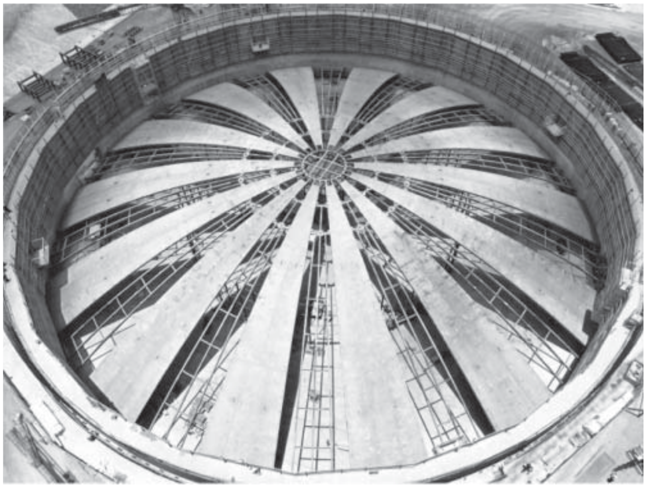

Fig. 8.4 Steel roof during construction – view from above.

supported. Searching for a method of construction to suit the above boundary conditions, concreting the tank roof on a steel roof supported pneumatically has proved to be an economic, time-saving method. The steel roof consists of radial and circumferential steel beams and steel plates at least 5 mm thick on top of those beams (Fig. 8.4). All the steel plates are welded together so the whole functions as a membrane when raising the roof pneumatically ("roof air raising") and, in its final condition, as a vapour barrier for the roof. The steel beams are attached to the concrete roof by means of headed shear studs, although the roof is rarely considered to be a composite structure. The steel beams must be fabricated accurately and include structural connections for transferring the loads in order to achieve and retain the desired dome shape. It should be remembered that the deformation of the steel structure that takes place during concreting is transferred to all valves and other fittings, the suspended pump column, the plinths for the roof platform and the fixings for the suspended insulation deck, and that all these connections will have to be adjusted and redone if the deformations are excessive.

On smaller tanks, triangular steel roof sectors are prefabricated, lifted into their final position and joined together there. The aim here is to keep the number of sectors to a minimum. Their maximum size and weight depend on the cranes available for lifting. The sectors are supported on a temporary scaffold tower erected in the middle of the tank for this purpose and on the peripheral compression ring. With larger tank diameters, the steel structure is assembled on the base slab. Again, a central scaffold tower provides temporary support, but in this case it is much shorter.

Corbels to support each individual steel beam are built into the concrete wall below the anchorage for the thermal corner protection (TCP) about 5 m above the base slab. To limit deflections during assembly, and hence minimise the deformations in the finished state, a ring of additional temporary supports is provided between the corbels and the scaffold tower (see Fig. 8.5).

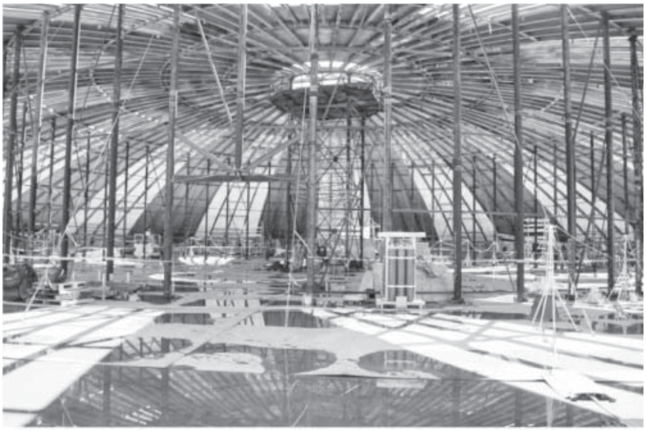

Fig. 8.5 Steel roof during construction – view from below.

This method of construction enables the steel roof to be assembled complete with the suspended insulation deck, insulation and all steel roof penetrations, e.g. access openings, pumps, valves and other fittings. A flexible membrane is fitted to seal the gap between the steel roof and the concrete tank wall. The seal includes temporary drainage pipes that drain precipitation downwards within the tank wall. Steel pipes installed in the base slab drain the water to the outside. After completing the roof, the drainage pipes are filled with concrete. This method ensures that work on the vapour barrier and the base insulation (which must be kept dry) inside the tank can begin while still building the concrete tank wall. After completing the concrete wall and the bottom part of the ring beam above it plus the steel roof with all its fittings, the steel roof structure can be raised pneumatically (Fig. 8.6). To do this, the two openings in the wall are closed off temporarily with steel plates.

Fig. 8.6 "Roof air raising".

Air blowers are set up at the larger opening to pump air into the tank. At least one compressor should be kept in reserve. The pressure required to raise the roof must compensate for the self-weight of the steel structure plus the suspended insulation deck and is in the order of magnitude of 1 kN/m^2 (100 mbarg). During the raising procedure, the steel roof is guided and held by wire ropes to prevent it twisting or rotating, to ensure that it arrives at the top of the tank in its intended orientation.

8.1.5 Concrete Roof

The tank roof must be designed for a uniformly distributed imposed load of 1.2 kN/m^2. The self-weight of a 40 cm thick concrete roof is 10 kN/m^2 and therefore about eight times the imposed load. It is the construction phase, and not the operating condition, that governs the design of the concrete roof. In order to remain economical, concreting the roof while still applying pressure from below has become the established method. Internal pressure in the order of magnitude of about 10 kN/m^2 (900–1200 mbarg) counteracts the self-weight of the steel roof and the load of the fresh concrete and thus significantly reduces the resulting forces and stresses. This method is in widespread use, whereas the construction details are often different and remain the intellectual property of the contractors. The overpressure must be maintained until the concrete has reached a specified strength (which is less than its design strength).

The concrete is placed in rings or in two layers in order to reduce the amount of concrete and duration of the concreting works. Concreting in layers reduces the weight of the fresh concrete considerably and leads to a more uniform loading, but does require a large amount of concrete and creates a large joint surface. This method requires the joints to be prepared afterwards and it might even be necessary to include shear reinforcement.

Concreting in rings (Fig. 8.7) over the full thickness of the roof can increase the thickness of the steel liner, but should be regarded as the better method in terms of quality because it results in a monolithic concrete structure whose only joints are perpendicular to the plane of the shell and flow of forces. However, this approach can lead to somewhat greater deformations and deviations from the theoretical form of the roof. After concreting one ring, it takes one day to prepare the construction joint. Whichever concreting method is chosen, the load of the concrete should be applied as circumferentially as possible in order to generate an approximately rotationally symmetric loading condition. To achieve this, at least three cranes or three concrete pumps will be required per tank.

Once the roof has been cast, most of the work on the concrete outer container is then over. At this point the laying of the base insulation and the tank base are also finished and work on the bottom segments of the steel inner container will have started. On opposite sides of the concrete wall there are two temporary openings measuring about 3.0–3.5 m high x 6.0–8.0 wide (Fig. 8.8). During construction these openings provide access for personnel and materials and also function as a separate means of escape.

The levels of the threshold and head of each wall openings are restricted. The threshold is not flush with the top of the concrete base slab, instead is aligned

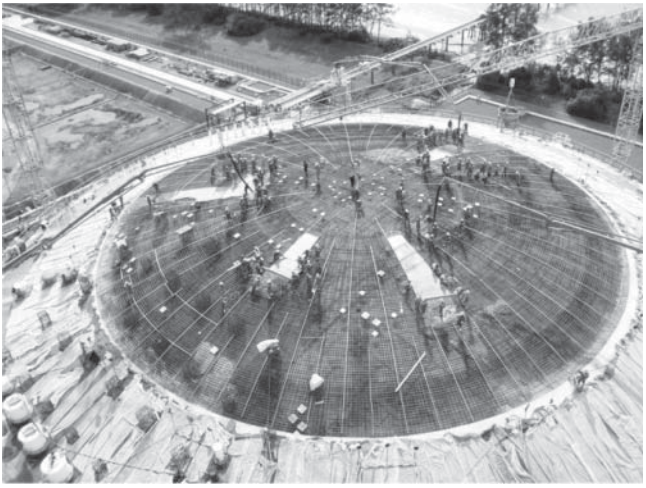

Fig. 8.7 Concreting the roof in circumferential rings.

Fig. 8.8 Temporary opening.

with the level of the base of the inner container. And the head should not be too close to the cast-in items anchoring the TCP. To avoid any potential clashes, many tank specifications specify a minimum distance of 1 m between the head of the opening and the TCP anchorage. When positioning the openings and the support corbels, it is also necessary to ensure a sufficiently high working space beneath the steel dome.

Prior to commissioning, the tanks must pass their hydrostatic and pneumatic tests. After testing, the inside of the 9% nickel steel tank must be cleaned following the contact with water, the temporary openings reinforced and concreted and the prestressing completed for these areas. A ring of nozzles in the roof above the wall insulation level enables perlite insulation to be blown into the annular space.

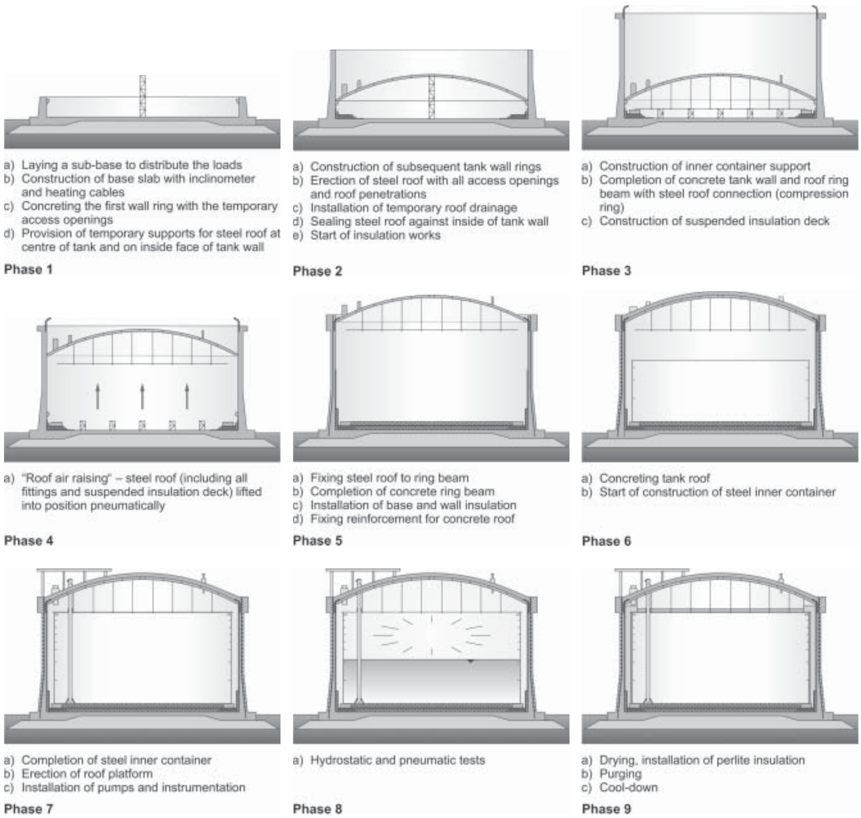

Fig. 8.9 Idealised construction phases.

Neglecting more complex foundation or ground improvement works, an LNG tank takes between 30 and 36 months to be built. The constraints placed on budgets and timetables have led to various forms of construction being developed and becoming established. For the operators of liquefaction plants, the returns that can be gained from shorter construction times and earlier commissioning are often greater than the costs of the measures needed to achieve that. "Roof air raising" is a typical example of this and the ingenuity of the engineers. Fig. 8.9 shows the general construction procedure for a 9% nickel steel full containment tank [2].

8.2 Wall Formwork

Just 10 to 15 years ago, it was normal to include continuous vertical cast-in elements in the concrete wall to which the steel liner could be welded. The number of cast-in plates usually matched the number of roof rafters, or was at least coordinated with them, and a multiple of four or eight. This regular grid was reflected in the number of formwork elements and reinforcement meshes (Fig. 8.10). The

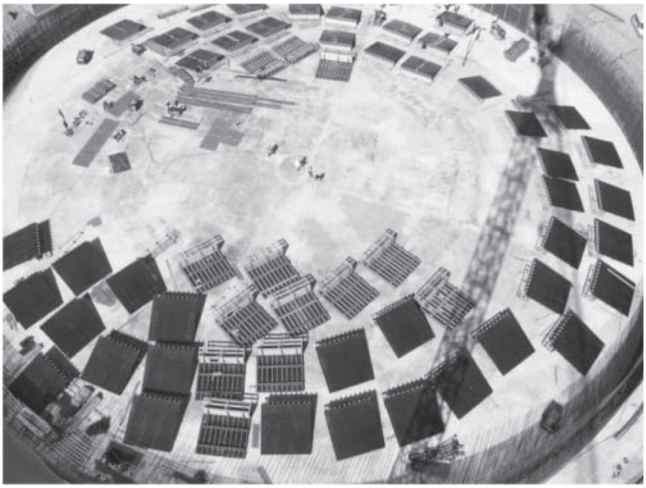

Fig. 8.10 Preparing the wall formwork elements.

spacing of the plates was 2.0–2.5 m, the width of the formwork elements twice that, i.e. 4.0–5.0 m, and the width of a reinforcement mesh again twice that. The cast-in plates were positioned in the middle of each formwork element and between each pair of adjoining formwork elements. In recent years, there has been a changeover from continuous cast-in parts to individual plates positioned at regular intervals.

That has had little effect on the dimensions of the wall formwork. Practical construction aspects such as ease of handling, limiting the weight with respect to crane capacities and the use of existing parts remain unaffected (Figs. 8.11–8.13). The height of one complete wall ring lies between 3.75 and 4.30 m.

Fig. 8.11 Climbing formwork being repositioned for next lift.

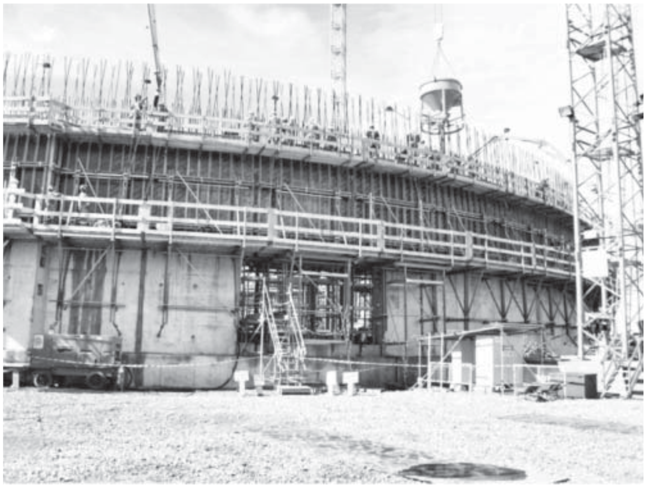

Fig. 8.12 Climbing formwork showing working platforms.

Fig. 8.13 Tank wall formwork.

The height of a wall ring depends on the pressure of the fresh concrete, the length of the vertical reinforcement and the amount of concrete that can be placed in one day. Three tie levels are required for such a wall ring, the topmost of which is above the concrete. To improve water-tightness, every tie must be provided with a waterstop.

For a tank diameter of 80–85 m a total number of 56 to 64 formwork elements are required to keep them manageable. Additional formwork elements and special parts are required on the outside due to the irregularities caused by the buttresses. It generally takes several days to dismantle and reposition a complete set of formwork.

Below the platform for fixing the steel reinforcement, there is normally another platform on the formwork for carrying out any finishing works on the concrete surface and removing and closing-off the climbing cones. The topmost (smaller) working platform is required for the concreting operations.

8.3 Reinforcement

Some 17 000 to 19 000 m^3 of concrete and about 4000 t of reinforcement, about a quarter of which is cryogenic reinforcement, are required for the LNG storage tanks frequently built these days with capacities of 180 000–200 000 m^3. Apart from the base slab, the orientation of the reinforcement in all components is defined by geometry and method of construction. On a pile foundation, the layout of the reinforcement must match the pile grid. Piles are usually positioned on an orthogonal grid in the middle of the tank and in two or three rings around the perimeter. Where the depth of the base slab differs for the middle and perimeter areas, the result is a staggered arrangement of the bar laps on the underside of the slab. If the orthogonal and radial reinforcing bars cross in one plane, a sufficient bending radius must be ensured for the bent splice bars; a bending radius of 15 or 20 d_s is appropriate (Fig. 8.14). Where a raft foundation is being used, the transition from radial to orthogonal reinforcement can be positioned as required, is not determined by geometry or construction method (Fig. 8.15). Neither approach has any clear economic advantages. The wall starter bars in the base slab are made from normal steel; cryogenic reinforcement is first required in the wall itself.

Various options are available for installing the wall reinforcement. It is normal to use individual bars, pre-bent reinforcement meshes (Fig. 8.16) and pre-assembled reinforcement cages (Fig. 8.17), with normal reinforcement on the outside of the wall, cryogenic reinforcement on the inside, spacers, ducts for

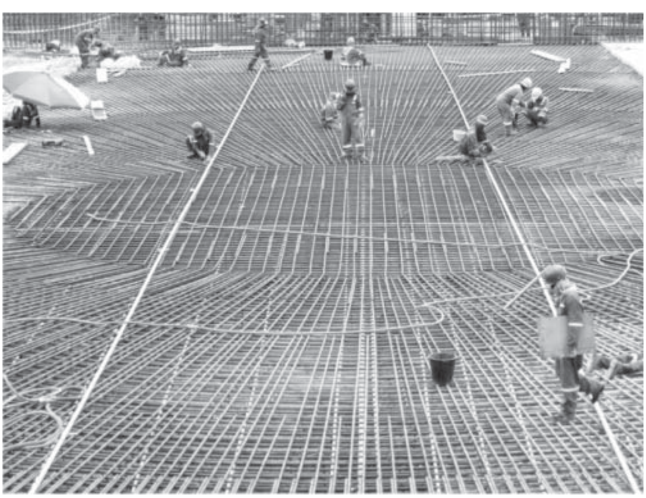

Fig. 8.14 Orthogonal reinforcement layout in middle of base slab.

Fig. 8.15 Radial base slab reinforcement for a smaller tank.

Fig. 8.16 Large reinforcement mesh suspended from a crane spreader beam.

prestressing tendons and, in unfavourable conditions, shear reinforcement as well. Meshes with 100% laps can be installed with a crane spreader beam; meshes with shifted rebars and 50% laps are more awkward to handle. As meshes and cages become larger and heavier, so it becomes more difficult to install them and the inaccuracies increase. The time-savings hoped for are paid for in terms of quality and accuracy of construction.

The dimensions of the buttresses should be chosen such that the tendons can be arranged without a change in curvature or, if unavoidable, with only a very small reverse curvature. With a larger reverse curvature, stirrups (links) will be required in the buttresses already crowded with criss-crossing ducts (see Fig. 8.18).

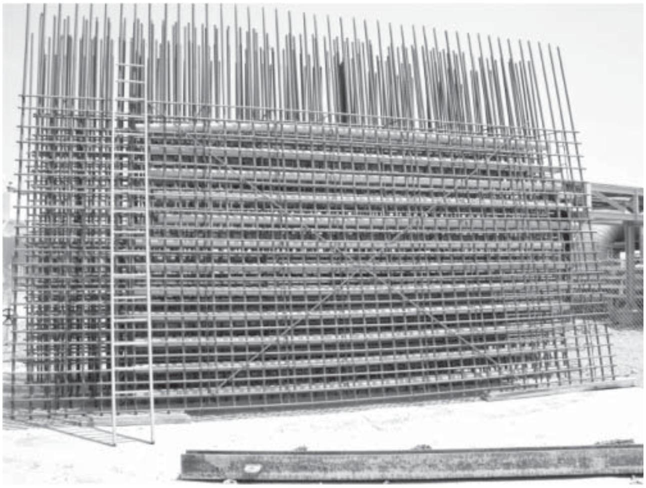

Fig. 8.17 Pre-assembled reinforcement cages.

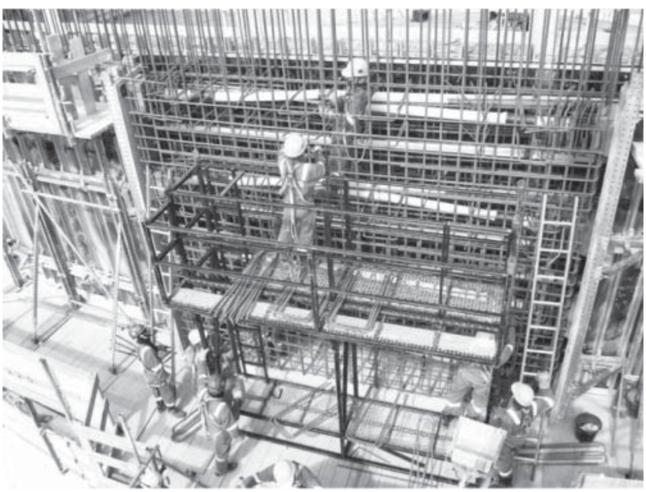

Fig. 8.18 Fixing buttress reinforcement.

The ring beam is the most heavily reinforced component of the tank. This is due to the discontinuity of the geometry and the large forces to be carried with the associated flow of forces. It is advisable to use threaded couplers to connect the reinforcement in the ring beam to the reinforcement in the roof. This ensures that the vertical construction joint is accessible so that it can be properly and relatively easily prepared for the next concrete pour. If the bars are lapped instead, access to the construction joint is restricted and preparing a good-quality joint becomes very time-consuming.

The interior of the tank is pressurised while the roof is being concreted and is therefore not accessible. Therefore, the concreting works should be completed as quickly as possible in order to avoid delaying the interior works any more than is necessary. To do this, the reinforcement for the entire roof is laid first and only afterwards is the concrete placed in rings or layers. Tank specifications or the supervising engineers often refer to BS 4466 (now withdrawn) for pre-bending of the reinforcement. This standard specifies for which radii it is necessary to pre-bend the reinforcing bars, depending on the bar diameter: 30.0 m for $d = 25$ mm and 43.0 m for $d = 32$ mm. Pre-bending is not necessary for the radial roof reinforcement, but is necessary for the hoop reinforcement for smaller radii.

The starter bars projecting from the tank roof for the plinths of the roof platform should be planned with a generous length. The reinforcement should be adapted to the intended level once the concrete of the roof has set. The temporary openings in the tank wall can be closed off following the hydrostatic tests. All formwork inside the tank must be removed through the access openings in the roof. The reinforcement to the openings is connected with threaded couplers. Prior to concreting the openings, the ducts should be inspected for any potential obstructions.

8.4 Prestressing

Strands with a cross-section of 140 and 150 mm^2 weigh 1.10 and 1.20 kg/m respectively. They are supplied in coils, which should be of a manageable size. Therefore, the recommendation is to limit the weight of a coil to 2–3 t. A coil that weighs 3 t contains about 2500 m of prestressing strand, which – without allowing for any wastage – corresponds to roughly one tendon with 19 strands over half the circumference of the tank.

The horizontal prestressing of a tank with a capacity of 180 000 m^3 requires about 70–80 horizontal tendons, i.e. 140–160 coils for a tank with horizontal prestressing. Sufficient space must be provided on site for storing these coils under suitable climatic conditions (Fig. 8.19). In the past, tendons were also installed in the base slab in the circumferential direction in order to achieve a confining

Fig. 8.19 Storage of prestressing strands.

effect. Although this does reduce the amount of reinforcement needed, it does call for the stressing operations to begin at an early stage. These days, starting the stressing operations at a later stage is regarded as more economic, and the greater amount of reinforcement is accepted. Tanks that are to be prestressed in the vertical direction as well as the horizontal direction require anchorages in the base slab.

In exceptional cases the vertical prestressing is designed to be straight and an inaccessible fixed anchorage is provided in the base slab. An accessible fixed anchorage is possible in the case of elevated base slabs. The tensioning system known as the "loop tendon system" is common for tanks on a raft foundation. It consists of two vertical tendons connected at the bottom by a U-bend (see Fig. 8.20). In the case of a low tank design pressure and a low prestress, the U-bends are positioned adjacent to each other, whereas adjacent U-bends overlap when the prestress is greater.

At the ring beam, the tendons are stressed with a so-called multiplane anchorage. It is best if the stressing is carried out simultaneously from both ends. When threading the strands through the vertical ducts, they are often unwound directly from a coil (see Figs. 8.21 and 8.22).

The walls are provided with horizontal circumferential prestressing (Fig. 8.23). Each hoop tendon consists of two strands, each of which encompasses half the circumference. They are anchored in four buttresses at 90° to each other. Each neighbouring anchorage thus belongs to the next-but-one tendon. Six buttresses are required where tendons are even closer together, such as for the very large storage tanks that are built in Japan and Korea.

The stressing sequence can be chosen as required. Sometimes the tank owner will specify that vertical stressing must be carried out first. This is based on a corresponding requirement in BS 7777 (now withdrawn). The idea is to reduce

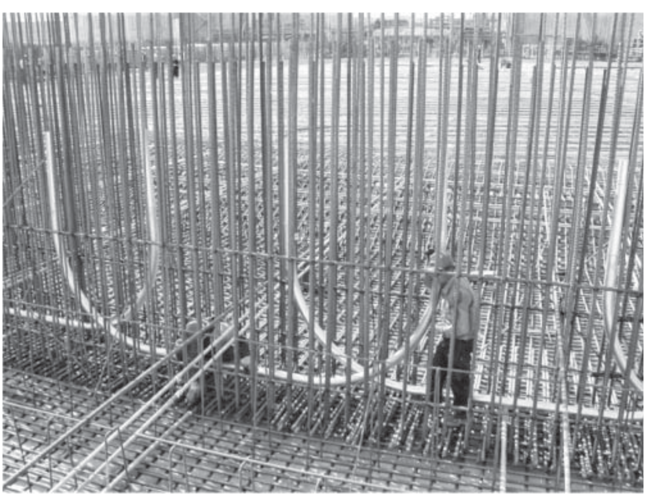

Fig. 8.20 Redirecting the vertical prestress in the base slab.

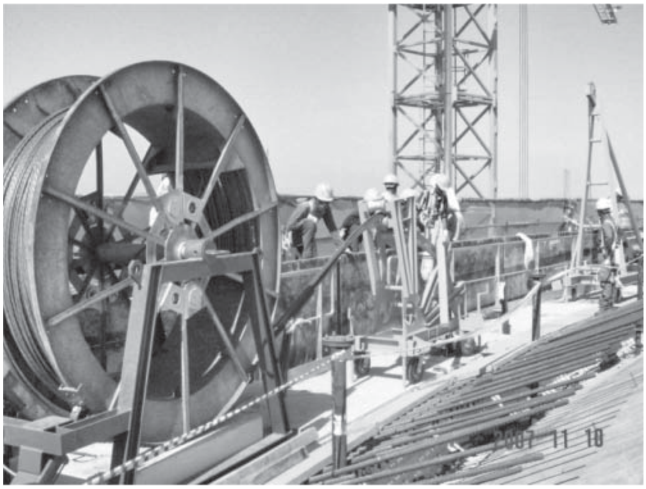

Fig. 8.21 Installing vertical tendons.

Fig. 8.22 Stressing the vertical strands.

the moment at the wall/base junction. If this method is used, stressing cannot begin until the concrete of the wall and the ring beam has fully hardened.

A different approach has been employed by DYWIDAG and others. The idea here was that after the concrete has reached the necessary compressive strength, several horizontal tendons are stressed in each wall ring, the intention being to reduce the cracking during the early phase. This method calls for the stressing crew to be on site throughout the entire period while constructing wall, ring beam and roof.

For a 180 000 m^3 LNG storage tank, the total weight of the prestressing steel amounts to about 600–700 t for the horizontal and 100–150 t for the vertical

Fig. 8.23 Working platform for horizontal stressing.

direction. This huge range is due to the specific requirements, especially for limiting crack widths and the specification for the liquid spill load case.

8.5 Tank Equipment (Inclinometers, Heating)

Elevated tanks do not require base slab heating because there is sufficient air circulation beneath the base slab. However, tanks on a raft foundation require heating in the base slab to prevent the ground freezing. The heating system in the base slab should be designed in such a way that the temperature does not drop below 0°C at any point. Automatic controls keep the temperature in a range between 5 and 10°C. The heating system must be designed with redundancy, i.e. consist of two independent cable and control installations.

What this means is that conduits must be installed for the heating cables at a spacing of $\ll$ 1 m (Fig. 8.24). In the middle of the tank a slab depth of at least 50 cm will be required in order to accommodate reinforcement, criss-crossing inclinometer casings and heating cables. The cable conduits are positioned about

Fig. 8.24 Cable conduits for heating cables in middle of base slab.

20–25 cm below the upperside of the slab (Fig. 8.25). Around the perimeter the slab is > 1 m deep, but the conduits remain at the same level.

Where differential settlement > 30 mm is expected, the settlement at the centre of the tank must be measured as well. To this end, two horizontal inclinometers are installed in the base at 90° to each other and crossing at the centre of the tank (Fig. 8.26). Each inclinometer casing should be installed with a gentle fall to the outside in order to ensure automatic drainage of any water that collects. The inclinometer casing is assembled from separate pieces about 3 m long which have two (sometimes four) grooves on the inside to guide the rollers of the measuring

Fig. 8.25 Laying cable conduits for heating cables around the perimeter.

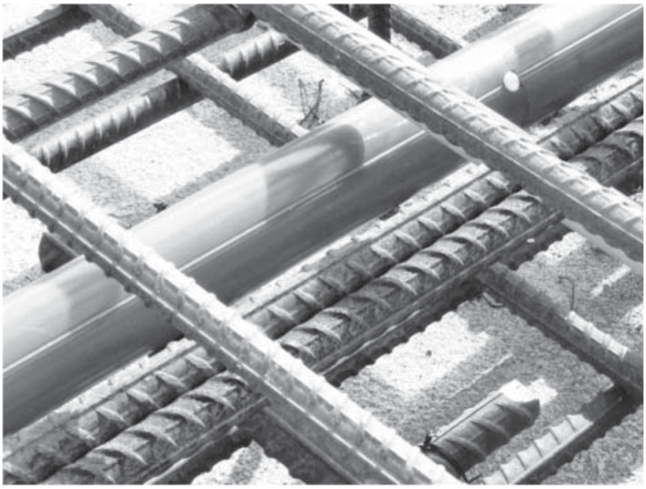

Fig. 8.26 Inclinometer casing in base slab.

probes. The ends of each section of casing are fitted with couplers so that the grooves continue at a regular spacing over the full length.

The casings must be installed in such a way that the grooves are aligned vertically. This vertical alignment is extremely important, as the measurements are carried out based on a level defined by the grooves. Deviations lead to inaccuracies in settlement measurements.

The inclinometer measuring system consists of measuring probe, cable drum, measuring cable and the associated computer. The measuring probe runs on wheels and measures the inclination at defined intervals in the measuring direction defined by the grooves. To do this, the probe is pulled through the inclinometer conduit (Fig. 8.27). The change in level is calculated from the

Fig. 8.27 Taking inclinometer readings.

inclination in a section and the length of the section defined by the spacing of the running wheels. The measurements therefore determine a vertical deformation profile.

8.6 Construction Joints

Construction joints represent discontinuities in the concrete structure and also weak spots in the case of poor workmanship. Therefore, the number of joints should be minimised when planning the concreting works. Their positions should be carefully considered and defined prior to starting work. Wherever possible, they should be perpendicular to the general direction of the component and the reinforcement (Fig. 8.28).

After concreting a section and after the concrete has hardened to the extent that it can retain its shape, all cement laitance and loose material must be removed to an adequate depth to reveal the aggregate. All splashes and deposits of concrete on reinforcing bars and formwork surfaces must be cleaned off after completing a concrete pour and also after any unscheduled interruptions to the concreting work.

The surfaces of all construction joints should be roughened – but not damaged – with mechanical tools 24 h after placing the concrete. The roughened surface should be washed with clean freshwater. High-pressure water jets can also be used to prepare the surfaces of construction joints; mechanical roughening is then unnecessary.

The surfaces of construction joints must be covered with damp hessian sacking at least 12 h prior to concreting. Such sacking must be kept thoroughly damp the whole time. When concreting, the existing concrete should be kept saturated with water but the surface dry.

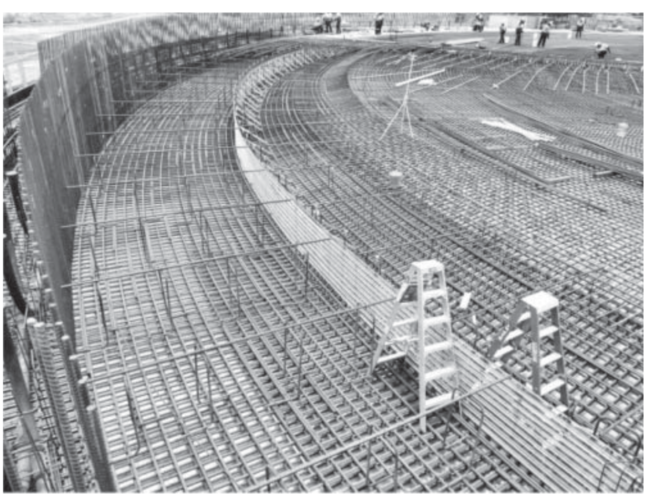

Fig. 8.28 Construction joint being prepared with expanded metal.

8.7 Curing of Concrete Surfaces

The concrete should be covered with damp hessian sacking and PE sheeting as soon as the formwork is removed. Boards or battens are used to hold the two layers in position on the base slab and walls. A perforated water hose along the top edge of the wall can be used to keep the sacking permanently and thoroughly saturated. Curing measures should be maintained for a period of seven days.

References

1 Deutscher Beton- und Bautechnikverein e.V.: Merkblatt Gleitbauverfahren. Berlin, 2008.

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9
Summary

This book is intended to provide an overview of the design and construction of storage tanks for liquefied natural gas (LNG). As there are many facets to this wide-ranging topic, it has only been possible to introduce the various aspects.

Seen globally, natural gas will remain an indispensable primary source of energy for many decades to come. This applies universally to industrialised, newly industrialised and developing countries. In the coming years, as the countries of East Africa begin to export natural gas and LNG, so this will strengthen their economic independence and prosperity. New applications will appear as ships and vehicles change to this new fuel.

The trend towards smaller liquefaction and regasification facilities as well as smaller storage tanks (small- and mid-scale terminals) has led to new players becoming involved in the market. Unfortunately, they frequently exhibit insufficient technical expertise and experience, something that has already led to considerable shortcomings, and problems and incidents during design and construction.

In order to curb this development, clients and investors especially must prioritise safety, quality and sustainability ahead of purely economic criteria. However, there is also a need to revise and adapt European standards. This need has been recognised and therefore commissions and standards committees at national and international level have begun the work of drawing up mandatory requirements and criteria for design and construction and revising the standards of the EN 14620 series.

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