Engineering Failure Analysis 40 (2014) 114–140

Contents lists available at ScienceDirect

Engineering Failure Analysis

journal homepage: www.elsevier.com/locate/engfailanal

Cement and concrete as an engineering material: An historic appraisal and case study analysis

Colin R. Gagg *

The Open University, United Kingdom

A R T I C L E I N F O

Article history:
Received 24 July 2013
Received in revised form 5 February 2014
Accepted 5 February 2014
Available online 28 February 2014

Keywords:
Cement and concrete
Historic development
Mechanical response
Structural failures
Failure case studies

A B S T R A C T

Today, second only to water, concrete is the most consumed material, with three tonnes per year used for every person in the world. Twice as much concrete is used in construction as all other building materials combined. There is little doubt that concrete will remain in use as a construction material well into the future. However, with such extensive use of the material, discovery of any shortcoming or problem associated with concrete or reinforced concrete structures will become a matter of considerable public concern – both from a safety perspective and associated costs of rectification. Accordingly, this paper will initially review the historic development of cements and concrete and will then focus on the mechanical response of concrete and reinforced concrete to its working environment. At appropriate points within the narrative, case study input will be used to illustrate or highlight principal themes.

1. Introduction

There is a general lack of understanding of the difference between cement and concrete, with the two terms often used interchangeably. However, cement is actually an ingredient of concrete and can be considered the ‘glue’ that binds aggregates together to form concrete. Therefore, concrete is basically a mixture of aggregates and paste – the aggregates being sand and gravel or crushed stone; the paste being water and Portland cement. Portland cement is not a brand name it is the generic term for the type of cement used in virtually all concrete, just as stainless is a type of steel. Cement will constitute 10 to 15 percent of the concrete mix by volume and, through a process of hydration the cement and water harden and bind the aggregates into a rocklike mass. This hardening process will continue for years implying that concrete will get stronger as it gets older.

Varying the mix of cement, sand and aggregate used in a concrete blend enables its use in a range of applications. Construction of a typical family home will require 14 tonnes of cement, a kilometre of motorway will contain as much as 2,500 tonnes of cement, and a building can be made to last for 100 years. Products can be designed, coloured and shaped to accommodate a variety of environmental conditions, architectural requirements and to withstand a wide range of loads, stresses and impacts.

Today, second only to water, concrete is the most consumed material, with three tonnes per year used for every person on earth [1]. Twice as much concrete is used in construction as all other building materials combined. There is little doubt that concrete will remain in use as a construction material well into the future. However, with such extensive use of the material,

* Tel.: +44 01908 564891.
E-mail addresses: colingagg@yahoo.co.uk, c.r.gagg@open.ac.uk

http://dx.doi.org/10.1016/j.engfailanal.2014.02.004
1350-6307/

discovery of any shortcoming or problem associated with concrete or reinforced concrete structures will become a matter of considerable public concern – both from a safety perspective and associated costs of rectification. Accordingly, this paper will initially review the historical development of cements and concrete and will then focus on the mechanical response of concrete and reinforced concrete to its working environment. At appropriate points within the narrative, case study input will be used to illustrate or highlight salient themes.

2. Cement and concrete

'Cement' is a generic term that can be applied to all binder materials. From earliest times, builders have used binders in conjunction with rock and stone to form more stable structures. Simple mud was employed as a binder, and is still in use in parts of the world today. In the days of early civilisations of Egypt, Greece and Rome, a lime cement was made by a process of 'burning limestone' to give Quicklime [2,3]. When mixed with water, quicklime formed slaked lime (calcium hydroxide) and, when mixed with more water to form a paste (now a lime mortar), slaked lime slowly hardened by reacting with carbon dioxide in the air to form calcium carbonate – or chalk. This production process is one of the oldest in the chemical industry, the reactions of which are described as:

1. Calcining or burning of chalk/limestone to produce quicklime

   $$CaCO_3 + HEAT = CaO + CO_2$$

2. Hydration or slaking of quicklime to produce hydrated lime

   $$CaO + H_2O = Ca(OH)_2$$

3. Carbonation of hydrated lime to produce calcium carbonate.

   $$Ca(OH)_2 + CO_2 = CaCO_3$$

2.1. Hydraulic cement

In contrast to lime mortars that harden by the action of carbon dioxide, the Greeks and Romans also developed cements that harden by reacting chemically with water – hydraulic cements – that, once hardened, form a product unaffected by further water contact. In particular, these civilisations discovered that when mixing a volcanic material (Pozzolana) and burnt brick with quicklime, the resultant cement was of far superior quality than simple lime mortars [2]. Known as Roman cement, these mortars set slowly, attained a much higher ultimate strength when cured in water than air, and became extremely hard. The hydraulic setting reaction of Roman cement is a testament to its quality as demonstrated by the many examples of Roman brickwork still standing today (Fig. 1).

The knowledge of making hydraulic cements was lost with the fall of the Roman Empire. It was not until the mid-eighteenth century before hydraulic cement was rediscovered by John Smeaton (1724–1792), a civil engineer commissioned by the Royal Society to rebuild the Eddystone lighthouse over a four year period in 1755–59 (Fig. 2). Smeaton began experimenting with cementitious materials that would harden (and stay hard) in severe marine conditions, and that would set and develop some strength in the twelve hour period between successive high tides. He found that the best water-resistant hydraulic cements were obtained by burning limestone with considerable quantities of clay. However, the discovery of cement used today was still some years away.

Fig. 1. The Colosseum or Coliseum, origionally the Flavian Amphitheatre, in the centre of the city of Rome.

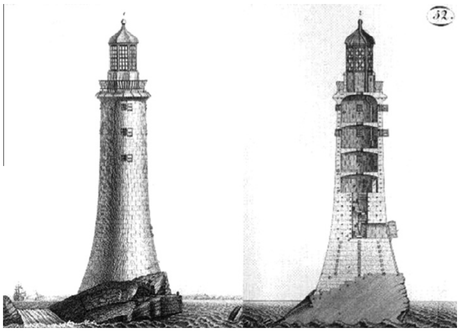


Fig. 2. Eddystone lighthouse built between 1755–59 using an early form of hydraulic cement developed by John Smeaton (1724–1792).

2.2. Portland cement

Joseph Aspdin, an English mason, is generally credited with the invention of Portland cement. In 1824 he obtained a patent (21 October 1824; No. 5022) for his product, which he named portland cement as it produced a product that resembled the colour of oolitic limestone quarried on the Isle of Portland, England. The specifications for the new cement were somewhat vague; a very pure limestone was to be burned to lime, the lime mixed with a definite quantity of clay, and the mixture pulverized wet. The wet mixture was to be dried and crushed and then calcined in a vertical kiln and finally the calcine was to be powdered. The patent does not state what proportions of lime and clay should be used, or at what temperature the mixture was to be burned. Today, the name 'Portland Cement' is used worldwide, with many manufacturers adding their own trade or brand names. Materials used in manufacturing Portland cement must contain appropriate proportions of lime, silica alumina and iron. During manufacture, frequent analyses are made to ensure a uniformly high-quality product. The raw materials are pulverised and mixed in the desired proportions. After blending, the prepared mix is fed into a rotary kiln, where it is sintered or fired at temperatures of 1400–1650 °C. The product is a lumpy mixture of stable compounds termed clinker. Clinker contains iron (Fe) and aluminium (Al) as well as silicon and calcium, in four main compounds:

- Alite (Ca_3SiO_5).
- Belite (Ca_2SiO_4), known to geologists as larnite.
- Aluminate ($Ca_3Al_2O_6$).
- Ferrite (Ca_2AlFeO_5).

The clinker is then cooled and pulverized. During this operation, a small amount of gypsum is added to regulate the initial chemical reaction of the cement. This pulverised product is finished Portland cement, ready for use in making concrete. Normal Portland cement is the type most commonly manufactured, with high early strength and sulphate-resisting types also available. Other special cements – such as masonry, oil well, expansive, regulated set – may not always be readily available from manufacturers due to low demand. However, on its own, pure cement is inadequate as it shrinks and cracks and, as a product, it is also much more expensive – in both energy and monetary terms – than sand and gravel.

2.3. Concrete

Concrete is a construction material that consists of cement (normally Portland cement), sand, crushed stone or gravel (aggregate), chemical admixtures and water. Typical concrete mixes are proportioned by absolute weight, with some typical mixes shown in Table 1. After mixing (and placement), water reacts with the Portland cement through a process of hydration. As the mixture cures, four main substances are produced:

- CSH (calcium silicate hydrates).
- Portlandite ($Ca(OH)_2$).

Table 1
Some typical concrete mixes proportioned by absolute weight.

	Cement (%)	Water (%)	Air (%)	Fine agg (%)	Coarse agg (%)
Mix I	15	18	8	28	31
Mix II	7	14	4	24	51
Mix III	15	21	3	30	31
Mix IV	7	16	1/2	25–1/2	51

- Ettringite ($Ca_6Al_2(SO_4)_3(OH)_{12}·26H_2O$; includes some Fe).
- Monosulfate ($[Ca_2(Al,Fe)(OH)_6]·(SO_4$, OH, etc.)$·xH_2O$).

The concrete mix then solidifies and hardens with the Portland cement acting as a matrix, binding the sand and aggregate to form a composite material. Therefore, the final product is a composite material that can be considered a type of artificial stone, and one advantage that concrete has over natural stone is that it can be moulded into complex shapes.

One of the more recent advances in concrete technology was the development of air-entrained concrete in the late 1930s [4,5]. Air-entrained concrete is produced through the use of air-entraining Portland cement, or by introducing air-entraining admixtures as the concrete is mixed [6]. The amount of entrained air is usually between 5% and 8% of the volume of the concrete, but may be varied as required by special conditions. Air-entrained concrete contains billions of microscopic air cells (Fig. 3). These relieve internal pressure on the concrete by providing tiny chambers for the expansion of water when it freezes. Thus air-entrained concrete is highly resistant to severe frost action and cycles of wetting and drying or freezing and thawing and has a high degree of workability and durability.

In terms of tonnage, concrete (particularly reinforced concrete) is now used more than any other engineering materials including steel and timber. The pre-mixed concrete industry consumes the greatest volumes of cement, using it in applications that include:

- Concrete slabs and foundations for buildings, roads and bridges.
- Precast panels, blocks, and roofing tiles.
- Fence posts, reservoirs and railway sleepers.

Cement is also used in bulk quantities in other diverse applications including:

- Stabilisation of roads and rocky surfaces.
- Backfill mining operations and casings in oil and gas wells.
- Renders, mortars and fibre board.

Once the useful life of a concrete structure has passed, the concrete can be recovered during demolition and recycled for use as construction road base and aggregate.

3. The age-strength relationship of concrete

Strength of concrete is generally given as a compressive value, expressed in mega pascals (MPa) at an age of 28 days. However, other test ages can also be quoted, therefore it is important to recognise the relationship between the 28-day strength and other test ages. Seven-day strengths are often estimated to be about 75% of the 28-day strength and 56-day and 90-day strengths are about 10–15% greater than 28-day strengths as shown in Fig. 4.

The compressive strength that a concrete can achieve will be a result of the water–cement ratio, the extent to which hydration has progressed, the curing and environmental conditions and the age of the concrete. The relationship between strength and water–cement ratio is shown in Fig. 5, which presents 28-day compressive strengths for a range of concrete mixtures and water–cement ratios. It can be seen that strength increases as the water–cement ratio decreases. This factor will also affect the flexural and tensile strengths and bond of concrete to steel.

The direct tensile strength of concrete is about 8–12% of the compressive strength (Fig. 6) and is often estimated as 0.4–0.7 times the square root of the compressive strength (in MPa). Splitting tensile strength is 8–14% of the compressive strength [7].

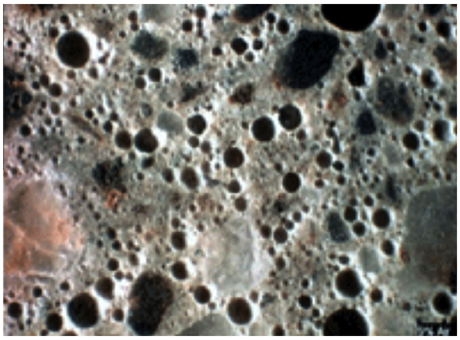


Fig. 3. Optical micrograph of air entrained concrete.

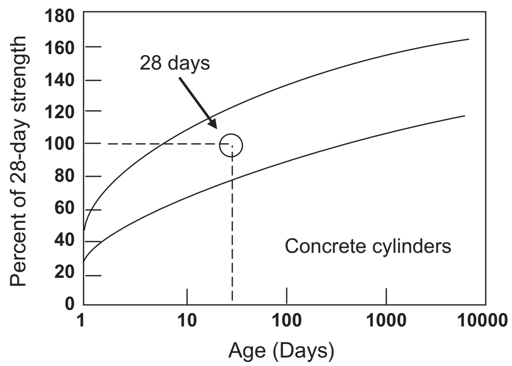


Fig. 4. Compressive strength development of various concretes illustrated as a percentage of the 28-day strength.

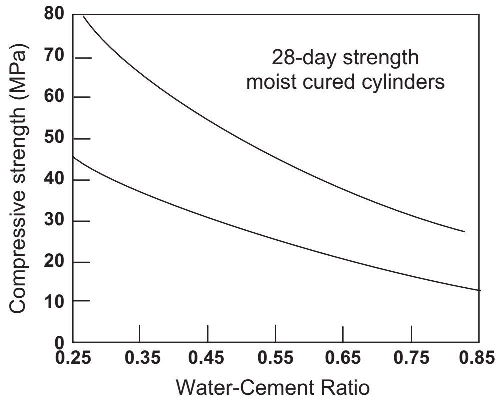


Fig. 5. Range of typical strength to water–cement ratio relationships of Portland cement concrete.

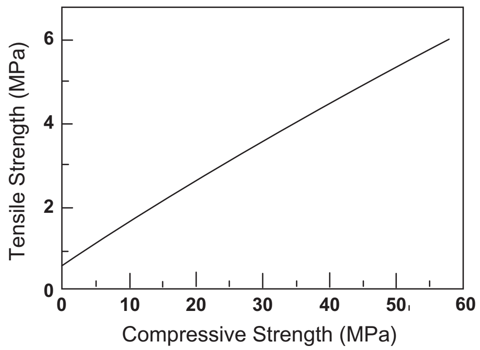


Fig. 6. Relationship between compressive and tensile strength of Portland cement concrete.

3.1. Case study: Willow Island cooling tower

Twenty-eight day cylinder strength is an indicator of strength of a concrete cylinder cured in a laboratory for 28 days. It does not necessarily reflect concrete strength in a structure. The strength gain of concrete is highly dependent on ambient temperature. In cold weather, concrete in a structure will be far weaker than laboratory cylinder strength would predict.

The Willow Island, West Virginia cooling tower collapsed (Fig. 7) while under construction on April 27, 1978, killed 51 workers in the worst construction disaster in U.S. history [8–10]. A jump form system was being used, with the formers secured by bolts in one-day and three-day-old concrete. The formers were designed to be progressively raised up the tower, as it was built. However, the temperature had dropped to one or two degrees Centigrade during the night. The National Bureau of Standards found that the concrete had not attained enough strength to support the formers. The report concluded that

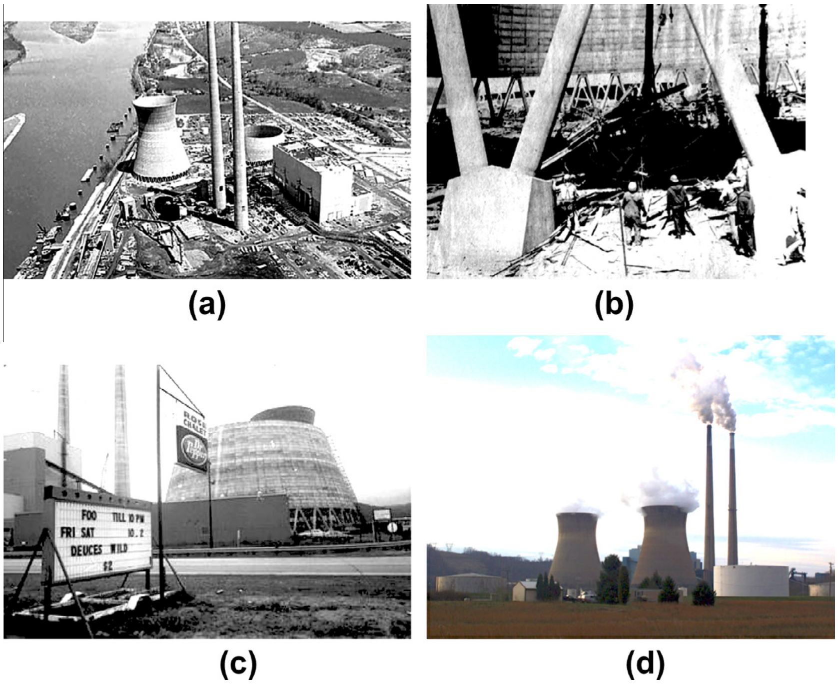


Fig. 7. The collapse of a cooling tower at Monongahela Power's Willow Island Power whilst under construction (a–c); the site as it is today (d).

"the most probable cause of the collapse was the imposition of construction loads on the shell before the concrete of lift 28 had gained adequate strength to support these loads [11].

3.2. Concrete alkali-aggregate reaction

Most concrete structures are designed to provide for normal and expected volume changes without excessive cracking. However, occasionally concrete structures can exhibit a long-term continuous increase in volume i.e. they will undergo concrete growth. Concrete growth can result from a variety of reactions, such as the hydration of unstable oxides included in the concrete mix, or the oxidation of minerals or from an outside attack of sulphates. The principal reaction creating concrete growth is that between minor alkali hydroxides from cement and the concrete aggregates. In most concrete, aggregates are more or less chemically inert. However, some aggregates react with the alkali hydroxides in concrete, causing expansion and cracking over a period of many years. This alkali-aggregate reaction has two forms—alkali–silica reaction (ASR) and alkali–carbonate reaction (ACR). Alkali–silica reaction is of more concern as aggregates containing reactive silica materials are more common. In ASR, aggregates containing certain forms of silica will react with alkali hydroxide in concrete to form a gel that swells as it adsorbs water from the surrounding cement paste or the environment. These gels can swell and induce enough expansive pressure to damage concrete. Concrete deteriorating from an alkali-aggregate reaction will develop typical indicators that include an obvious network of pattern or map cracking and, in advanced cases, closed joints and attendant spalled concrete. These alkali-aggregate reactions and their accompanying concrete growth have presented numerous problems across a range of large concrete structures such as the Fontana concrete gravity dam.

3.3. Case study: Fontana concrete gravity dam, Tennessee River NC, USA

Dam failures and incidents involve unintended releases or surges of impounded water. They can destroy property and cause injury and death downstream. While they may involve total collapse of a dam, that is not always the case. At the time of planning and construction, Fontana dam (Fig. 8) was one of nine dams to be built to manage the Little Tennessee River. Standing at a height of 147 m (480 ft), with a width of 721 m (2365 ft), it was constructed between 1942 and 1945, by the Tennessee Valley Authority (TVA), in an attempt to provide hydroelectric power for the war effort. As it turned out, its generators began to produce electricity only a few months before World War II ended.

During early design stages, the TVA considered an embankment, arch and a buttress dam as potential solutions for the project. However, a gravity dam design was considered to be the most efficient and economical. Furthermore, many gravity dams had been built previously, giving added confidence to the project.

One potential drawback was that the downstream face of the dam faced south and would therefore absorb a large amount of heat from the sun. To allow for expansion and contraction, the design called for vertical contraction joints connecting large sections of the dam.

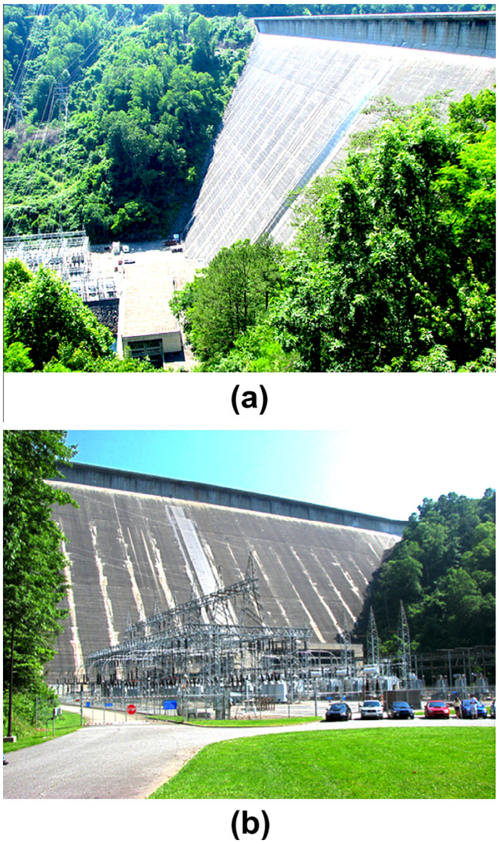


Fig. 8. The Fontana Dam and incline leading down to the powerhouse (a); electrical substation viewed from downstream side of dam (b).

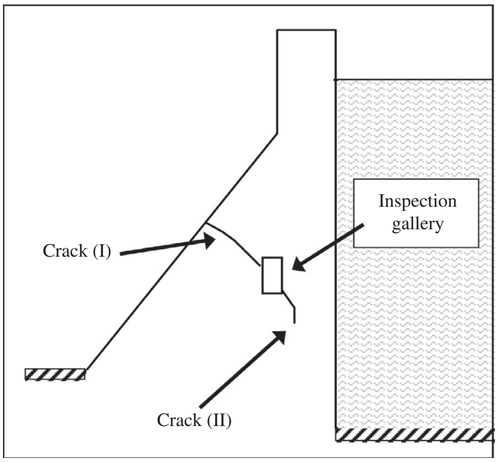


Fig. 9. Cracking in the Fontana concrete gravity dam as determined by the Tennessee Valley Authority (TVA) 1976.

During 1949 patterned cracking was first observed, along with what appeared to be the beginnings of an upstream movement of the dam structure. In 1972, deep cracking was observed in the walls of the foundation at the drainage gallery located in the curved portion of the dam (Fig. 9). Two hypotheses were postulated, either:

(a) The foundation was deficient under the curved portion of the dam.
(b) The main portion of the dam was expanding due to heat and/or other causes, thus straining the curved portion so that it began to bend.

The Tennessee Valley Authority (TVA) monitored crack growth during 1973 and found that the crack was opening. Solar heat gain was thought to be the culprit. Two-dimensional analyses were performed in an attempt to determine the actual causes of cracking. This analysis determined that the foundation was not the problem; the rest of the dam was expanding and causing cracking in the curved portion.

Petrographic examination revealed an alkali-aggregate reaction i.e. a chemical reaction was occurring between the cement and the aggregate components of the concrete. This chemical reaction combined with solar heat had caused the cracking near the curved portion of the dam, with cracking of the main body of the dam being resisted by the foundation. Temporary solutions included spraying cold water on the downstream face to lower the temperature, with crack opening diminishing as a result. In addition, the area around the crack was post-tensioned with steel wire, pulling the crack closed.

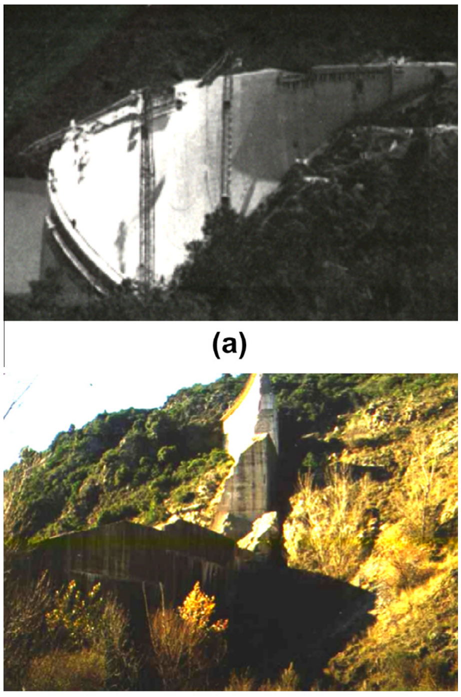
A third potential cause of cracking in the dam has since been uncovered. It transpires that during construction, cooling pipes were run through the dam to lower the temperature during chemical formation of the concrete. These pipes cooled the dam too much and, as a result, it had to warm back up on the inside, in order to reach equilibrium with the outside.

To date, the TVA continues to monitor the dam, taking weekly readings. Their observations confirm that the dam is still "growing", with the top centre section of the dam having moved 114.3 mm (4.5 in.) upstream since 1976.

3.4. Case study: Malpasset Dam, Frejus, French Riviera

There was no loss of life associated with the Fontana dam shortcomings. Indeed, the area is now a major tourist attraction. The majority of dam structures have storage volumes small enough that failures have few or no repercussions. However, failure of dams storing large amounts of water can be catastrophic, characterised by the sudden, rapid, and uncontrolled energy release of impounded water. Such dam failures are generally catastrophic if the structure is breached or significantly damaged, as was the case of the Malpasset dam failure in the south of France (Fig. 10).

The dam took 30 months to build, with construction having been halted several times due to lack of funding and labour disputes. These stoppages resulted in long periods of time for the cement to harden completely before work resumed and new cement was added. This may well have resulted in a non-homogeneous structure. The dam was finally completed in December 1954 and was gradually filled up over the following five years. Cracking was noticed near the base on the downstream side of the dam, but it was not investigated. It is undetermined when these cracks appeared. With the autumn rains of November 1959, the water was still 7 m below the top when small leaks were discovered along the right bank of the dam. The leaks grew rapidly, indicating a serious danger of collapse, but unfortunately the local population was not notified. At 21.13 h on December 2, 1959 the dam burst and a 40 m high wall of water thundered down the valley at speeds in excess of 70 km an hour.



(a)

(b)

Fig. 10. Malpasset concrete arch dam under construction (a); right-hand buttress remnants after failure (b).

Immediately below the dam were a couple of isolated houses, the hamlet of Malpasset, the Bozon mining hamlet and an autoroute construction site. When the dam burst, the mass of released water tore through, along with much of the dam itself, carrying 600-ton blocks of concrete as far as 1.7 km downstream – destroying and killing all in its path. Most of the residents died, including the Malpasset hamlet and the construction workers. The cost of lost lives was enormous, with the death toll being as high as 421 people – it is thought that there may have been as many as 100 additional unregistered workers who perished, undiscovered or unidentified that were not included in the final count – taking the final toll beyond 500 souls.

Little remained of the dam; failure had been sudden and catastrophic (Fig. 10b). The dam swung open and released the reservoir water. The dam itself broke away from the abutments and travelled downstream. A number of factors that probably contributed to the disaster seem obvious today. The geological condition of the rock was either not understood or not appreciated. Anthracite (hard coal) was then being mined at Boson just 4 km below the dam site. The thin coal deposits were mixed with grey schists and shale, and with large areas of soft white flourite. The different types of rock had incompatible qualities, exacerbated by a region with such weather extremes. In the summer, a severe lack of rain (dryness), and constant sun, baked the ground – with the differing rock types reacting in different ways. In the winter, torrential rains pounded the area, with some of the rock types absorbing the water. Furthermore, the situation would have been compounded by the probability of a non-homogeneous concrete structure, being a direct result of delays during construction.

Although this tragic failure will not be forgotten, valuable lesson have been learned:

– Three-dimensional computer analyses were developed to study the cause of failure; these types of analyses are now used to design new arches under new standards.
– The need to test the foundation for different traits was realized.
– The study of rock mechanics developed.
– The need for safety monitoring of arch dams was recognised.

4. Reinforcing concrete

Without the benefit of additional reinforcing, concrete will act as an inherently brittle material, exhibiting identical structural limitations as that of quarried stone. Both materials are hard and brittle, with the compressive strength of concrete being 10 times higher than that of its tensile strength. So, concrete is a strong material when compressed, but shearing forces or moderate tensile force will cause it to crack or buckle. The answer to these material shortcomings is to give concrete a supplementary internal support in the form of a metal 'skeleton' or 'cage' (Fig. 11a). This 'reinforcing cage' is fabricated from lengths of reinforcing bar (rebar for short) and links, forming complex networks designed to accommodate complicated loading patterns associated with complex structures (Fig. 11b). The rebar cages are then placed in position, wooden formers built to desired shape and finally the concrete is poured (or cast) into the cavity. Therefore the idea of reinforcing concrete is to offset low tensile strength of concrete by combining it with the tensile strength of steel to give a durable, inexpensive structural material capable of withstanding tensile, bending and shear loading. Civil engineers can choose from several different kinds of rebar when designing a concrete structure. An appropriate material choice is dependent on the service conditions that the (now) reinforced concrete will be exposed too.

4.1. Rebar form and material

The range of material types used for rebar include [12]:

Black steel rebar: cheapest type of rebar; most common rebar in buildings today; excellent where there is no moisture; susceptible to corrosion.
Epoxy coated rebar: more expensive than black steel rebar; there is difficulty in bonding to concrete; far less susceptible to corrosion; easily damaged, which could isolate and magnify any corrosion; best used in marine environments i.e. dock pilings.
Stainless steel rebar; good corrosion resistance; capable of withstanding shipping, handling, bending; available in magnetic or non-magnetic alloy; expensive.
Fibre-reinforced polymer (FRP): FRP is used when added strength and durability is required; resists changes in temperature; can be used in chemically aggressive environments; dependant on material choice, FRP is transparent to both radio frequency and magnetic fields.

Black rebar is the most commonly used material, being used in almost all standard buildings other than buildings in marine environments. However, some material types available for rebar are too expensive to use in most applications. Austenitic stainless steel rebar is non-magnetic and therefore an ideal candidate for construction of magnetic resonance imaging (MRI) rooms. However, its price is much higher than even epoxy-coated rebar forms. Fibre-reinforced polymer (FRP) is also lightweight, typically one-quarter to one-sixth the weight of standard steel rebar, but again the cost is significantly higher than traditional black rebar.

Fig. 11. Rebar cage acting as a structural skeleton (a); more complex structures require complex rebar networks (b and c).

There are three physical characteristics that combine to give reinforced concrete a unique set of properties [12]:

(1) The coefficient of thermal expansion of concrete is similar to that of steel, eliminating internal stresses due to differences in thermal expansion or contraction.
(2) When the cement paste within the concrete hardens this conforms to the surface details of the steel, permitting any stress to be transmitted efficiently between the different materials. Rebars are roughened or corrugated to further improve the bond or cohesion between the concrete and steel.

(3) The alkaline chemical environment provided by calcium carbonate (lime) causes a passivating film to form on the surface of the steel, making it much more resistant to corrosion than it would be in neutral or acidic conditions (see concrete corrosion).

4.2. Case study: Highway overpass collapses near Montreal, Canada

On September 30, 2006, at 12:30 PM EDT, an overpass on Boulevard de la Concorde (Exit 5) collapsed onto the busy Autoroute 19 in Laval Canada (Fig. 12). Two vehicles were crushed underneath, while three others and a motorcycle fell from the top. Five people were killed and six others were injured, including three critically. There have been numerous reports from witnesses who saw the two vehicles being crushed underneath the structure. The viaduct, built in 1970, had been rated for 35 years more service and had a maintenance check one year earlier, in 2005. The police called Transports Quebec to report fallen chunks of concrete one hour before the collapse, and a Transports Quebec team had visually inspected the span less than thirty minutes prior to the collapse. The section between Autoroute 440 and Boulevard Levesque was reopened four weeks later. An estimated 60,000 motorists use the highway and connected bridge to the Island of Montreal daily. A public investigation [13] into the collapse was instigated, and headed by former premier Pierre-Marc Johnson.

The commission found three major causes contributed to the overpass failure:

- Improper rebar support for the design, which caused a "plane of weakness" where cracks eventually occurred.
- Improper rebar installation at the time of the overpass's construction in 1970.
- Use of low-quality concrete to build the overpass.

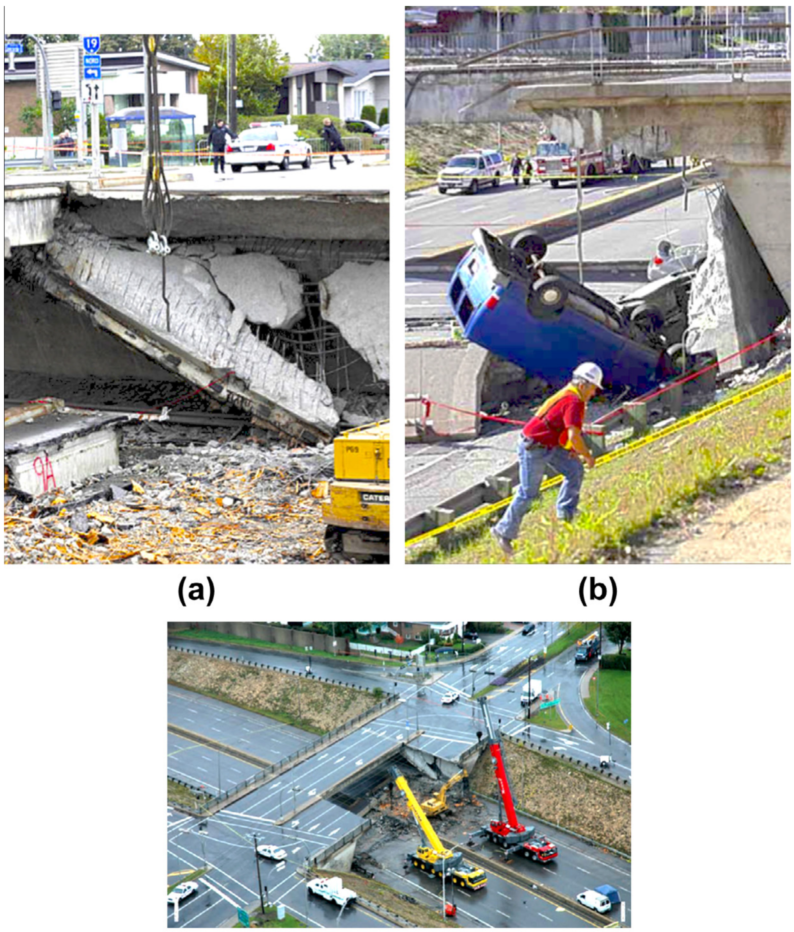


Fig. 12. Boulevard de la Concorde exit 5 overpass collapse in September 2006 (a–c).

The commission's report also outlined several other contributing causes – which experts testifying at the enquiry did not all agree on – for the overpass collapse:

- Shear vulnerability: The thick concrete overpass was vulnerable to shear failure because it was not reinforced to withstand cracking and deterioration.
- Lack of adequate waterproofing: A major repair job in 1992 called for a waterproof membrane. The engineer leading the project decided the concrete had deteriorated too much and did not install the protective membrane.
- Weakening of the structure in 1992: The routine replacement of expansion joints was a bigger repair job than expected, and engineers had to remove more concrete than originally anticipated to make the fix, weakening the structure.

The remainder of the structure was demolished on October 21 2006, after further inspection of the remains. A nearby overpass was also ordered to be demolished due to structural concerns.

4.3. Case study: Pittsburgh Midfield terminal pre-cast beam collapse

Unless a sufficient development length of steel is embedded in concrete, the bar will pull out before it yields. In 1990 a portion of the Pittsburgh Midfield terminal failed during construction. In a pre-cast concrete beam, the bottom reinforcing bar was embedded only 185 mm, which is much shorter than necessary [14].

4.4. Reinforcing in action

Bending stresses induced in a beam cross section are primarily tensile in the bottom section reaching a maximum along the bottom face and compressive in the top section reaching a maximum at the top face. Bending stresses are zero at the neutral axis (Figs. 13 and 14). Induced bending stresses can be calculated by the elastic bending relationship.

Where y is the distance of the stress element from the neutral axis of the cross section; M is the bending moment and I is the second moment of area.

This relationship would suggest that consideration of longitudinal compressive and tensile stresses were all that was required for beam design. However, as the depth of a beam in relation to its span increases, this assumption becomes invalid. Shear forces become significant when the ratio of span to depth of beam is less than 10. Shear stresses are also significant around the supports of longer beams. The interaction between bending and shear stresses will produce resultant tensile forces that act diagonally – Fig. 14(a). These tensile forces can lead to early cracking as shown in Fig. 14(b). To resist this tensile stress, rebars need to be placed at right angles to the anticipated stress as shown in Fig. 14(c). A more common arrangement, using vertical stirrups, is shown in Fig. 14(d).

Fig. 15 shows how a reinforced concrete beam would behave under bending load. Clearly, to benefit from the tensile strength of rebar, reinforcing should be incorporated on the tensile side of a beam, as shown in the figure. As loading on the beam is increased from zero to failure, four different regions of behaviour can be identified. These are labelled I–IV on the load–deflection curve in Fig. 15(a). In region I, both the steel and concrete behave elastically, the concrete in compression above the neutral axis and the steel and concrete in tension below it. This is reflected in the linear stress distribution through the thickness of the beam Fig. 15(b). Region I finishes when concrete on the tensile face starts to fail at its fracture strain ($\sim$0.02%). In region II, beam deformation remains elastic, but with the cracked region increasingly growing towards the neutral axis, the principle tensile resistance is provided by the rebar – with the bending stiffness of the beam being lower as a result. Onset of yielding in the rebar signals the start of region III, with the slope of the load–deflection curve decreasing further and becoming non-linear. This trend is accentuated by the increasingly non-linear compressive deformation of the concrete. Collapse of the beam in region IV is precipitated by failure of the concrete in compression at the upper surface of the beam.

Accordingly, to make the most efficient use of the two materials, a beam should be designed so that the rebar starts to yield before the concrete fails in compression. This is also a safety measure, as tensile yielding of rebar is progressive and the beam will remain load bearing, whereas crushing of concrete is explosive and therefore catastrophic.

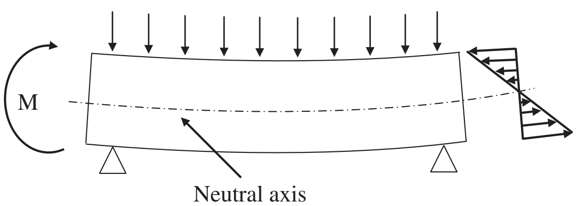


Fig. 13. Stress distribution in a simple rectangular beam in bending.

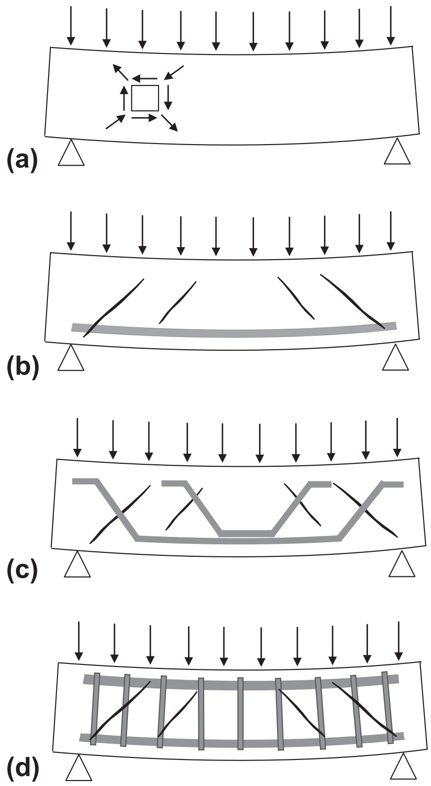


Fig. 14. Combined shear and bending stresses in beams (a); resulting cracks (b); the placement of rebars to counteract this cracking (c); an alternative rebar arrangement with vertical stirrups (d).

Consequently, the concept behind concrete reinforcement is to offset the low tensile strength of concrete by combining the superior tensile properties of the chosen rebar with the compressive strength of concrete. This combination of properties will give rise to a durable and inexpensive structural material capable of withstanding bending and shear loads that will be generated in a loaded structure.

4.5. Case study: Collapse of terminal 2E Charles de Gaulle International Airport

On May 23, 2004, a portion of roof at the Charles de Gaulle International Airport in Paris collapsed, killing four travellers and injuring three more (Fig. 16). The airport's terminal 2E consisted of a main passenger building, the fated concourse parallel to it, and an "isthmus" connecting the two buildings. A concrete and glass form rose from the second floor of the concourse and bulged out from its base, forming a flat arch (Fig. 16a). With a design capacity for 25,000 people, the terminal concourse was conceived as one long concrete tube having no internal supporting pillars. However, the roof collapse came less than a year after opening. An investigative commission under the direction of Jean Berthier, engineering Professor at France's Ecole Nationale des Ponts et Chausées, concluded that the building's structure had been fragile from the outset. It then progressively degraded under use – principally from the side walkways – to the point where the structure gave way.

Berthier's report pointed to four connected causes [15]:

(1) Insufficient or badly positioned structural steel.
(2) Lack of mechanical "redundancy," in that the stresses were concentrated and could not be shifted to other structural components.
(3) Concrete beams that offered too little resistance to stress and use.
(4) The positioning of metal supports within the structural concrete.

Several solutions for subsequent repair had been considered, but none offered a guarantee for the security of personnel and passengers. Therefore, the entire vault of terminal 2E was pulled down and rebuilt.

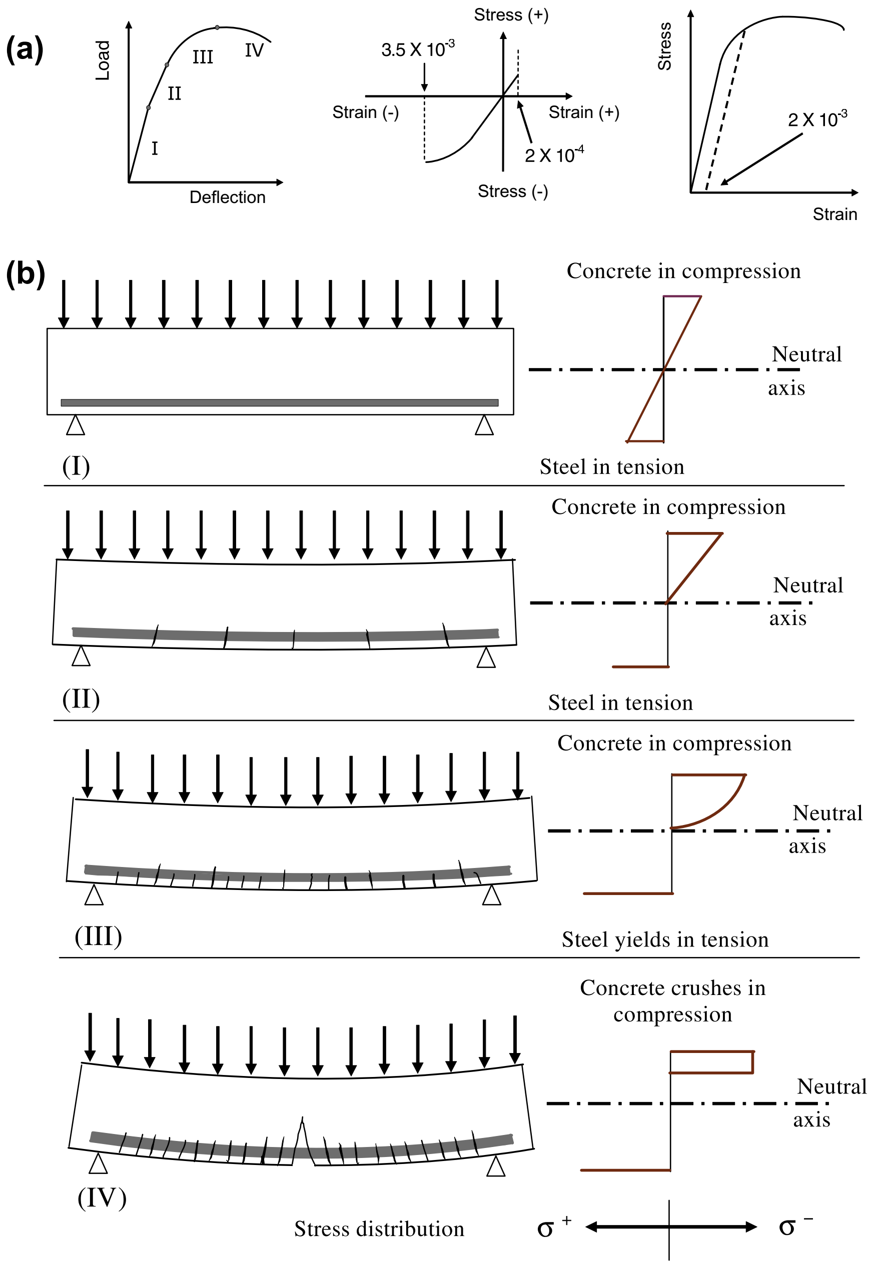


Fig. 15. Load–deflection and stress–strain characteristics of reinforced concrete, concrete and steel (a); loading to failure and stress distributions for a beam (b).

5. Pre and post-tensioning

In conventional reinforced concrete, the high tensile strength of a chosen rebar is combined with the exceptional compressive strength of concrete to form a structural material that is strong in both compression and tension. Designers use a system of pre-stressing as a way to reinforce concrete by introducing compressive stresses in an element or member prior to it entering service. The principle behind pre-stressing concrete is that compressive stresses can be induced by high-strength steel tendons in a concrete member, before service loads are applied. The level of compressive stress applied is designed to balance tensile stresses imposed in the member during service. This principle is analogous to a row of books being moved from place to place. Instead of stacking the books vertically and carrying them, the books may be moved in a horizontal position by applying pressure to the books at the end of the row. When sufficient pressure is applied, compressive stresses are induced throughout the entire row, and the whole row can be lifted and carried horizontally at once. In a

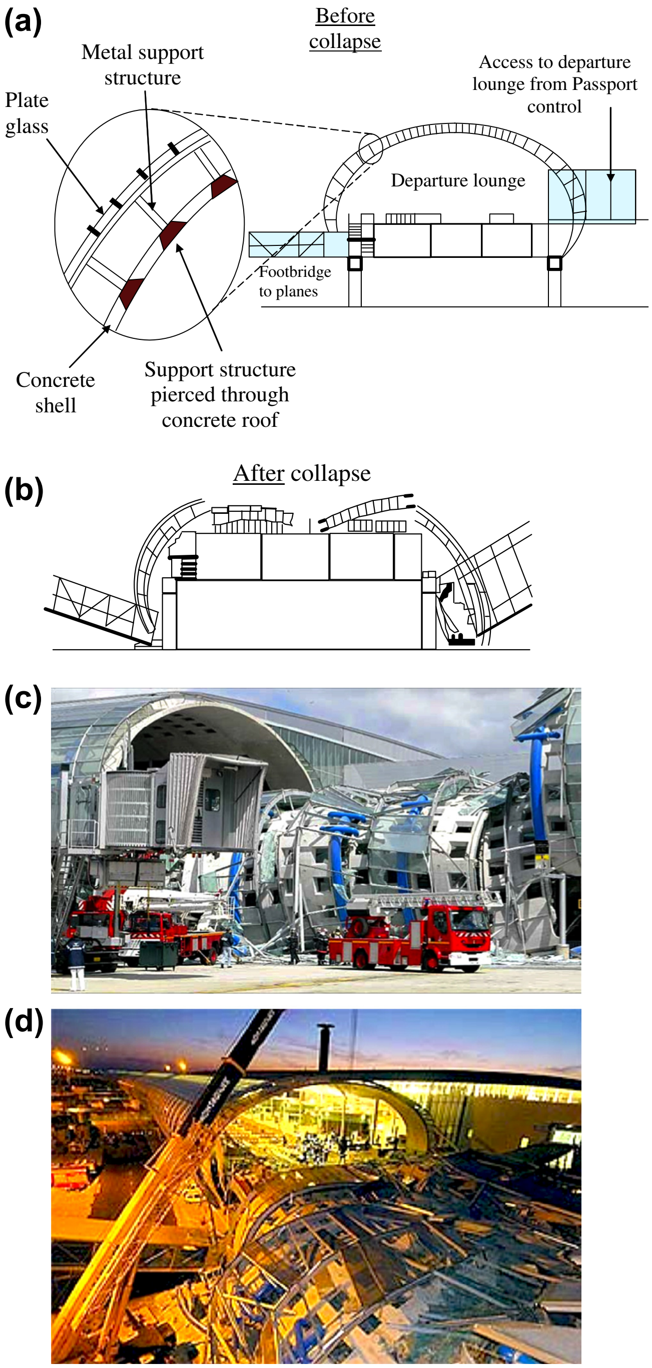


Fig. 16. Collapse of the newly constructed terminal 2E at Paris' Charles de Gaulle airport.

concrete structural element, such compressive stresses are induced in concrete either by pre-tensioning or post-tensioning steel reinforcement to obtain the desired level of pre-stress.

Pre-tensioning: In pre-tensioning, the steel is stretched before the concrete is placed. High-strength steel tendons are placed between two abutments and stretched to 70–80% of their ultimate strength. Concrete is poured into moulds around the tendons and allowed to cure. Once the concrete reaches the required strength, the stretching forces are released. As the steel reacts to regain its original length, the pre-stressing force is primarily transferred to the concrete through concrete/steel bond and, therefore, tensile stresses are translated into a compressive stress within the concrete.

Post-tensioning (PT): Post-tensioning is a method of pre-stressing in which the tendons are tensioned after the concrete has hardened and the pre-stressing force is primarily transferred to the concrete through the end anchorages. The use of post-tensioning offers several benefits, not least of which is the fact that the PT floor slabs are generally thinner than an ordinary reinforced concrete slab. They can also be up to 300 mm thinner than a floor in a steel frame. This minimises the building's height to the extent that this could mean an extra storey on a ten storey building. The amount of pre-stress can be adjusted to control deflection, thus enabling the minimum depth of slab to be used. PT slabs can economically span further than a reinforced concrete slab. This in turn reduces the required number of columns and foundations and increases flexibility for space planning. Flexibility is further enhanced by a post tensioned (PT) slab being able to accommodate irregular grids.

There are two methods of post-tensioniong: un-bonded and bonded.

Bonded; with bonded systems, the pre-stressing tendons run through small continuous flat ducts that are grouted up after the tendons are stressed. The bonded systems generally develop high ultimate strengths. However, the bonded ducts are larger than for un-bonded. This reduces the effective section depth for design purposes but there is less reliance on the anchorages after grouting.

Un-bonded; with un-bonded systems, the tendons run through a small protective sheath that allows the tendons to move independently of the concrete. They can be manufactured off-site thereby reducing the on-site programme. The tendons are more flexible and can be deflected in plan to be placed easily around holes. There is also no need for another trade to carry out the grouting.

5.1. Case study: Post tensioned bridge failures

The integrity of the tendons is key for the safety of segmental structures, as they are the reinforcing elements that hold any superstructure together. Therefore, corrosion of the steel strands must be prevented. The HDPE duct and cement grout act as double protection layers for the encased steel strands. Two corrosion-induced failures occurred in England: The sudden collapse of the Bickton Meadows footbridge in 1967 and of the Ynys-y-Gwas Bridge in 1985. Both failures were due to corrosion of tendons and, in addition to other documented problems with post-tensioned bridges, led to a ban in 1992 on the construction of new bonded post-tensioned bridges by the U.K. Ministry of Transportation [16,17]. In 1996, the moratorium on grouted post-tensioned cast-in-place construction was lifted but remains in force for segmental construction apparently because of concerns about the corrosion protection of tendons as they pass through the bridge segment joints.

In the USA, during the spring of 1999, a corrosion-related failure of an external tendon was found in the Niles Channel Bridge, Florida, after the bridge had seen 16 years of service. Niles Channel Bridge is one of a series of low-level segmental bridges stretching over seawater in the Keys area. Further inspection revealed that two steel strands were corroded in the tendon anchorage. In 2000, due to corrosion problems, eleven tendons out of a total of 846 were replaced in the Mid Bay Bridge after seven years of service. Also, in the same year, numerous corroded steel strands were discovered in segmental piers of the Sunshine Skyway Bridge, built in 1986. The corrosion was a result of seawater entering the ducts through a split. Fig. 17 shows the corroded steel strands in segmental bridge tendons on the Mid Bay Bridge.

5.2. Concrete corrosion

As discussed earlier, concrete is a complex composite material that displays low strength when loaded in tension. Improved tensile mechanical properties are obtained by reinforcing concrete with steel, with structures as diverse as bridges, buildings, elevated highways, tunnels, parking garages, offshore oil platforms, piers and dam walls all containing reinforcing steel (rebar). The principal cause of degradation of steel reinforced structures is corrosion damage to the rebar embedded within the concrete (Fig. 17). Therefore, concrete corrosion is actually the corrosion of embedded steel reinforcement within the concrete, and not corrosion of the concrete itself. Once initiated and allowed to progress, the mechanism of corrosion will generate ever increasing internal pressure on the concrete structure. Once the expanding steel has generated an internal pressure greater than the tensile strength of the concrete, cracking and rupture will follow. The presence of concrete

Fig. 17. Corroded tendons found on the Mid Bay Bridge.

corrosion will become apparent as a result of surface spalling, delamination and staining (Fig. 18). When concrete corrosion has been identified, instigation of remedial work will be required to ensure continued integrity of the structure.

5.3. Case study: Spalling and corrosion of a dock wharf

An in-depth inspection of four wharves was undertaken at a shipyard located in the estuary of the River Sado, Portugal [18]. Inspection revealed severe deterioration of different elements of the wharf structure. Built during the period 1973–1975, the length of the wharves varied between 120 m and 200 m with a width of 20 m. It's decking consisted of six pre-cast pre-stressed concrete beams.

Parallel to the length and a 30 cm thick cast-in-place concrete slab, pre-stressed in the transverse direction. Decking beams were then supported by hollow piles founded on the seabed. Being a marine environment, the deck slab and the upper part of the beams were exposed to the splash and spray zone, whilst the lower part of the beams and the upper part of the piles were exposed to the tidal zone and the lower part of the piles were submerged.

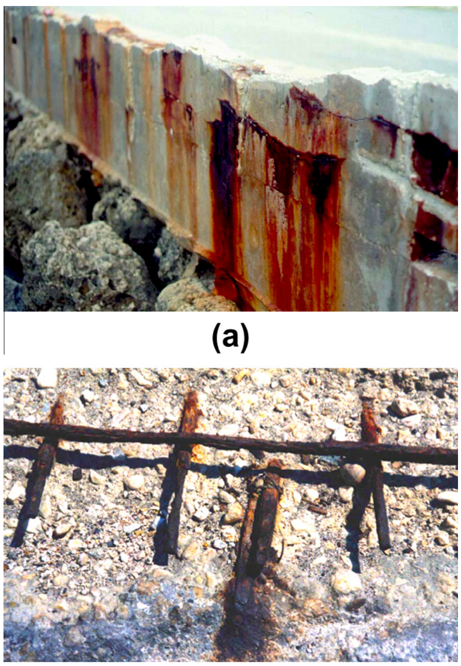
Pre-stressed tendons of the deck slab located below the crane rail gutters were in a badly corroded condition. The steel ducts were almost all destroyed by corrosion and in many cases stress corrosion cracking of the wires had occurred, as shown in Fig. 19(a). This failure mode will progress with little or no loss of cross-section. Sudden failure of the structural elements may occur with no visible warning of tendons deterioration.

In some instances, ducts were found to be totally un-grouted as a result of poor workmanship. In these situations the pre-stressed wires were not protected by the alkaline environment of the grout and a drastic degradation was found, as shown in Fig. 19(b).

In summary, extreme deterioration of the pre-stressing tendons was caused by an aggressive micro-environment, present in the rail gutters, that was linked to both poor materials and workmanship. Lack of drainage of the gutters had led to the accumulation of salt water on the concrete surface. Water evaporation deposited salts that lead to localised chloride concentrations. These conditions had lead to a high penetration rate of chlorides into concrete and therefore to the early development of the corrosion mechanism that overtook the tendons.

5.4. Non-steel reinforcement

Given that corrosion is the main cause of failure of reinforced concrete, it follows that a corrosion-proof reinforcement can extend a structure's life substantially. Fibre reinforced polymer (FRP) composite rebars have the potential to address this corrosion deficiency. FRP rebar can be used as non-prestressed reinforcement in concrete for members subjected to flexure, shear, and compression loadings. FRP composite rebars are resistant to chloride ion attack, offer a tensile strength of 1.5–2



(a)

(b)

Fig. 18. Corrision staining of concrete sea wall (a); spalling concrete as a direct result of corroding rebar on a bridge deck (b).

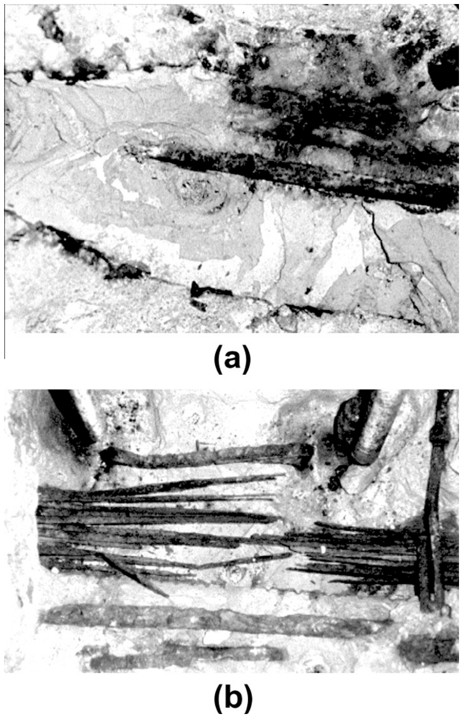


Fig. 19. Stress corrosion cracking found in pre-stressed wharf tendons (a); drastic degradation of wire tendons (b).

times that of steel, weigh only 25% of the weight of equivalent size steel rebar, and are a highly effective electromagnetic and thermal insulator. As it is resistant to corrosion, FRP rebar does not require a protective concrete cover of 30–50 mm or more – as is required by steel reinforcement. This means that FRP-reinforced structures can be lighter, have longer lifetime and for some applications be price-competitive to steel-reinforced concrete.

The choice of FRP rebar will offer the following features and benefits:

- Non-corrosive. FRP will not corrode, even when exposed to a wide variety of corrosive elements including chloride ions.
- High strength-to-weight ratio. It provides good reinforcement in weight sensitive applications.
- Non-conductive. Provides excellent electrical and thermal insulation.
- Excellent fatigue resistance. Performs well under cyclic loading situations.
- Good impact resistance. Will resists sudden and severe point loading.
- Magnetic transparency. FRP is not affected by electromagnetic fields; excellent for use in MRI and other types of electronic testing facilities.
- Lightweight. FRP is easily transported in the field without the requirement of heavy lifting equipment.

There are four general categories of applications where FRP rebars provide a suitable alternative to steel, epoxy-coated steel, and/or stainless steel bars:

- Exposure to de-icing salts: situations such as: parking structures; bridge decks; parapets; curbs; retaining walls and foundations; roads and slabs on grade.
- Marine environments: Possible applications include: quays; retaining wall; piers; jetties; caissons; decks; piles; bulkheads; floating structures; canals; roads and buildings; offshore platforms; swimming pools and aquariums, etc.
- Other corrosive environments: Typical applications include: wastewater treatment plants; petrochemical plants; pulp and paper mill and liquid gas plants; pipelines and tanks for fossil fuel; cooling towers; chimneys; mining operations of various types, nuclear power plants; and nuclear waste facilities.
- Low electric conductivity or electromagnetic neutrality requirements: may be utilised in toll-booths that read radio tags; electrical sub-stations; aluminium and copper smelting plants; structures supporting electronic equipment such as transmission towers for telecommunications; airport control towers; magnetic resonance imaging in hospitals; railroad crossing sites, and military structures with requirement for radar invisibility.

There are, however, limitations to the service use of FRP rebar – it is neither ductile nor fire resistant. Structures employing FRP rebar may therefore exhibit a reduced ductile structural response, along with decreased fire resistance. These limitations would have major ramifications in earthquake zones for example, and if employed in tunnel structures that could be subject to fire exposure. Issues regarding fire resistance are of major concern, and will be reviewed in more detail in the following section on fibre reinforcement of concrete.

5.5. Role of fibres in concrete

When service loads imposed on concrete approach that of failure, cracks will rapidly propagate. It has been discussed in prior sections how the introduction of rebar will arrest crack growth by acting as a long continuous filament. However, introduction of fibres, rather than rebar, into concrete will also provide a means of arresting crack growth. Fibre reinforcement will provide the same beneficial crack arresting effects as rebar. Furthermore, short discontinuous fibres have the added advantage of being uniformly mixed and dispersed throughout the concrete. Among the more common fibres used are steel, glass, asbestos and polypropylene (Table 2) [19], and are added to the concrete at the point of mixing – along with the cement, water and aggregate.

If the elastic modulus of the fibre is high with respect to that of the concrete or mortar binder, the fibres will carry part of the service load, thereby increasing the tensile strength of the concrete. Increases in the length to diameter ratio of the fibres will enhance the flexural strength and toughness of the concrete. However, the values of this ratio are usually restricted to between 100 and 200, as fibres which are too long tend to "ball" in the mix and create workability problems. As a rule, fibres are generally randomly distributed in the concrete. However, processing the concrete so that the fibres become aligned in the direction of applied stress will result in even greater tensile or flexural strengths. Rapid expansion of new developments and applications of fibre-reinforced concrete technology has greatly extended the range of service use (Table 3) [19].

The addition of polypropylene fibres to a concrete mix will significantly increase toughness but have little effect on tensile strength. On the other hand, mixtures of polypropylene and glass fibres will produce concrete with a high degree of both toughness and flexural strength (Tables 4 and 5) [19]. So it is clear that fibre reinforcing can improve both the toughness and flexural strength of concrete, and the choice of which fibre to use is based on availability, cost and fibre properties.

Fibres can also reduce creep strain of the concrete – creep strain being the time-dependent deformation under a constant stress [20,21]. Steel-fibre-reinforced concrete can have tensile creep values 50–60% of those for normal concrete. Compressive creep values, however, may be only 10–20% of those for normal concrete. Shrinkage of concrete is caused by the withdrawal of water from concrete during drying. With fibre reinforcement, shrinkage is lessened, with glass-fibre-reinforced concrete shrinkage decreased by up to 35% with the addition of 1.5% by volume of fibres [22,23].

Table 2
Physical and mechanical properties of selected fibres.

Fibre	Diameter (μm)	Specific gravity	Failure strain (%)	Modulus of elasticity (GPa)	Tensile strength (GPa)
Steel	5–500	7.8	3–4	200	1–3
Glass	9–15	2.6	2–3.5	80	2–3
Polypropylene	7.5	0.9	20.0	5	0.5
Mica flakes	0.01–200	2.9	n/a	170	0.25
Asbestos	0.02–20	2.5–3.4	2.3	200	3
Carbon	7.5	1.7–2.0	0.5–1.0	300–400	2–3

Table 3
Application of various fibres in cement products.

Fibre type	Application
Glass	Precast panels, curtain wall facings, sewer pipe, thin concrete shell roofs, wall plaster for concrete block
Steel	Cellular concrete roofing units, pavement overlays, bridge decks, refractories, concrete pipe, airport runways, pressure vessels, blast-resistant structures, tunnel linings, ship-hull construction
Popypropylene nylon	Foundation piles, prestressed piles, facing panels, flotation units for walkways and moorings in marinas, road-patching material, heavyweight coatings for underwater pipe
Asbestos	Sheet, pipe, boards, fireproofing and insulating materials, sewer pipes, corrugated and flat roofing sheets, wall lining
Carbon	Corrugated units for floor construction, single and double curvature membrane structures, boat hulls, scaffold boards
Mica flakes	Partially replace asbestos in cement boards, concrete pipe, repair materials

Combinations of more than one fibre type can be used for special purposes.

Table 4
Ratio of toughness values of some fibre-reinforced cementitious materials with respect to un-reinforced materials.

Composite	Volume percent (%) of fibre	Relative toughness*
Concrete		
Steel	0.5	2.5–4.0
Steel	1.0	4.0–5.5
Steel	1.5	10–25
Glass	1.0	1.7–2.0
Polypropylene	0.5	1.5–2.0
Polypropylene	1.0	2.0–3.5
Polypropylene	1.5	3.5–15.0
Mortar		
Steel	1.3	15.0
Asbestos	3–10	1.0–1.5
Cement paste		
Glass	4.5	2.0–3.0
Mica flakes	2.0–3.0	3.0–3.5

* These values are representative values only and may vary additionally dues to differences in test methods and specific process and mix variables.

Table 5
Ratio of flexural strength of some fibre-reinforced cementitious materials with respect to un-reinforced materials.

Composite	Volume of fibre (%)	Relative flexural strength*
Concrete		
Steel	1–2	2.0
Glass	1–2	2.5–3.5
Mortar		
Steel	1.3	1.5–1.7
Glass	2	1.4–2.3
Asbestos	3–10	2.0–4.0
Cement paste		
Glass	4.5	1.7–2.0
Mica flakes	2–4	2–2.5
Polypropylene	1–2	1.0

* These values are representative values only and may vary additionally due to differences in test methods and specific process and mix variables.

6. Fibre reinforcement and fires

Firestopping and fireproofing products can be ablative in nature. In this context, use of the term 'ablative' can signify endothermic materials, or merely materials that are sacrificial and become "spent" over time spent while exposed to fire. The latter version has also been used to describe silicone fire-stop products, which, by themselves, are sacrificial. In other words, given sufficient time under fire or heat conditions, these products actually char away, crumble and disappear. The idea is to put enough of this material in the way of the fire, so that a prescribed fire-resistance rating can be maintained, as proven in a fire test. Spalling of high strength concrete (HSC) during exposure to fires can be attributed to the build-up of pore pressure during rapid heating. HSC is believed to be more susceptible to this pore pressure build-up because of its low permeability – compared to that of normal strength concrete (NSC) [22,23]. The extremely high water vapour pressure, generated during exposure to fire, cannot escape due to the high density (and low permeability) of HSC. This pressure often reaches the saturation vapour pressure, which at 300 °C is about 8 MPa. Such internal pressures are often too high to be resisted by the HSC, which has a tensile strength of about 5 MPa. However, it has been found that the addition of polypropylene fibres will act as an ablative medium, and thus minimise spalling in HSC members under fire conditions [24].

An accepted theory [25] is that by melting at a relatively low temperature of 170 °C, the polypropylene fibres create "channels" for the steam pressure in concrete to escape, thus preventing the small "explosions" that cause spalling. The study showed that the amount of polypropylene fibres needed to minimize spalling is about 0.1–0.25% (by volume). Further research is being carried out to determine the optimum fibre content for different types of concrete. The effect of polypropylene fibres on spalling is illustrated in Fig. 20, which shows HSC concrete blocks after two hours of fire exposure [26].

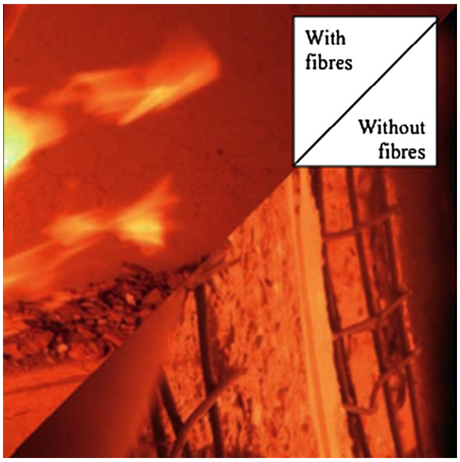


Fig. 20. The effect of polypropylene fibres on concrete spalling. The sample with fibre reinforcement stayed intact to a large extent; spalling of the sample without polymer fibre reinforcement exposes the steel rebar.

6.1. Case study: Channel tunnel fire

The channel tunnel is a rail tunnel beneath the English Channel connecting Coquelles, Pas de Calais region in France and Cheriton, Kent in England. The tunnel has a length of approximately 50 km, of which 37 km are under the English Channel. The majority of the tunnel is lined with pre-cast high strength reinforced concrete lining rings of 1.5 m wide, with a thickness

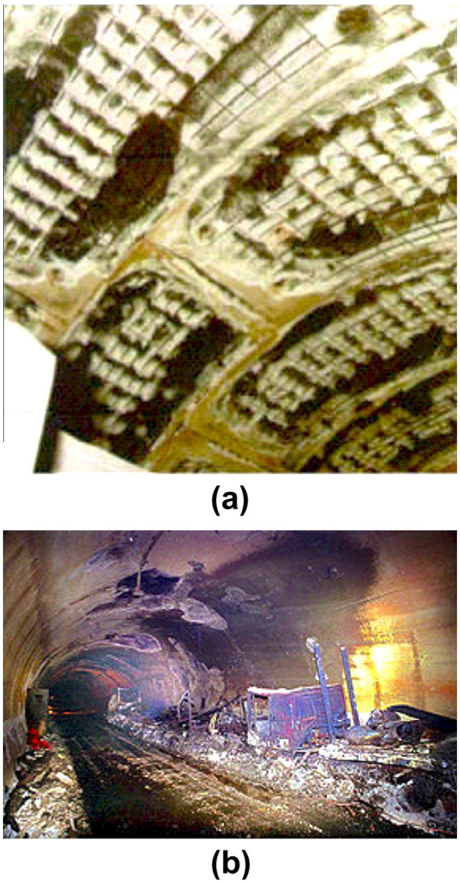


Fig. 21. Two examples of fire spalling of concrete: the channel tunnel (a); the Mont Blanc tunnel after the fire (b). In both cases, damage sustained by the concrete is clearly visible.

varying from 400 to 800 mm depending on the loading conditions. Where concrete lining was inappropriate, cast iron lining rings were used. On November 18, 1996, an England-bound train carrying expandable polystyrene caught fire 1 mile into the tunnel. The fire burned for 14 h, inflicting significant damage along a 480 m length of the tunnel lining structure. During the fire, large quantities of concrete had spalled off from the tunnel lining (Fig. 21a). This resulted in very fine concrete rubble collecting on the access walkway and the roof of the HGV wagons. The consistently falling off of the hot concrete debris endangered the life safety of the emergency personnel who were carrying out the rescue and fire fighting missions. Although there were no fatalities, 30 people were injured, and the England-bound rail tube was closed for 1 month for repairs. Subsequent forensic analysis, of the fire, highlighted the potential disaster of explosive spalling behaviour of high strength concrete (HSC) exposed to high temperatures [27,28]. Two possible ways of minimising the risk of extensive spalling of concrete lining for tunnels was highlighted:

(1) Providing fire-proof coatings to the exposed surface of concrete which is unable to resist fire.
(2) Adding an ablative medium, such as polypropylene fibres, into the concrete mix. The fibres will melt during a fire, thus creating paths in the matrix for water vapour to escape.

6.2. Case study: The Mont Blanc tunnel fire

The Mont Blanc tunnel is a 7.3 mile long single-bore, two-lane Alpine automotive tunnel connecting Chamonix, France and Courmayeur, Italy. The horseshoe shaped tunnel had a 7 m wide roadway with two 0.8 walkways on each side. At very 300 m, there were vehicle rest areas measuring 3.15 m wide by 30 m long – situated on alternating sides of the roadway and numbered from 1 to 36 in the France-Italy direction. Placed directly opposite to each rest area, there are designated U-turns for trucks. The roadway slab and ducts lying beneath were structures of reinforced concrete. However, the 0.5 m thick tunnel lining was pure concrete without the benefit of reinforcement. On March 24, 1999, a truck carrying margarine and flour caught fire whilst in the tunnel. The entire tunnel filled with combustion gases in less than 10 min. The ensuing fire continued to burn for 50 h, resulting with 39 fatalities – mainly the drivers trapped in the tunnel during the fire. Most of the drivers had stayed in or near their vehicles, but even those who tried to escape only managed to cover a distance of 100–500 m prior to being overcome by smoke inhalation. As a result of extensive structural damage (Fig. 21b), the tunnel was closed for repairs that lasted for 3 years.

7. Shotcrete

Shotcrete has become an important component of modern tunnelling technology, underground mining, slope and rock consolidation, repair of concrete structures and artificial rock structures (Fig. 22). There has been a rapid technological process development over the last 20 years, with wet and dry spraying methods vying for process dominance.

In a dry shotcrete process, all ingredients except water and sometimes liquid accelerators, are mixed in the dry state and the mix is conveyed by an air stream through hoses or pipes to a spray nozzle where water is added and the mix, including water, is thrown towards the structure to be covered. However, one of the main disadvantages of the shotcrete process lies with very high rebound loses. When using ordinary concrete mixtures, rebound losses for the dry shotcrete process may exceed 40% by weight of the total amount of concrete sprayed onto a surface. Furthermore, use of ordinary concrete mixtures will introduce a 5 cm layer thickness limit for a single pass.

Wet shotcrete involves batching and wet mixing of the cementitious mix according to controlled conditions in a mixer. This is then pumped along a hose and compressed air is added at the nozzle to facilitate the spraying of the mix onto the required surface. Until recently, dry shotcrete had been the dominating method, particularly in the area of rock support. However, the wet shotcreting method is rapidly gaining market share in relatively confined underground working areas.

7.1. Case study: Tunnel collapse at Heathrow airport

A serious tunnel collapse (Fig. 23) occurred at Heathrow airport on 20 October 1994 [29,30]. At the time, the Heathrow Express rail station tunnel was being constructed using a sprayed concrete (shotcrete) tunnel construction technique known as the New Austrian Tunnelling Method (NATM). Subsequently described in court as one of the UK's worst civil engineering disasters of the past 25 years, the collapse caused a huge crater to appear between the airport's two main runways – as well as inflicting damage to car parks and buildings. No one was injured but the clean-up operation lasted months and caused massive disruption. During a following trial, the Judge said in court that it was luck more than judgment that the collapse did not crush to death passengers using the nearby Piccadilly Line on the Tube. The contractor building the tunnel, admitted failing to ensure the safety of its employees and the public and was fined £1.2 m in February 1999. An Australian engineering firm, responsible for monitoring the project, was fined £500,000.

There were allegations that the NATM method was unsuitable for tunnelling in the London Clay; allegations that was deemed groundless by NATM proponents the success of the method in many other clay conditions all over the world. Two collapses in Germany (Fig. 24), the Munich Metro and Krieberg Tunnels received similar media attention, and in Turkey the Bolu tunnel experienced massive problems. As a result of the collapse at Heathrow, the Health and Safety Executive pub-

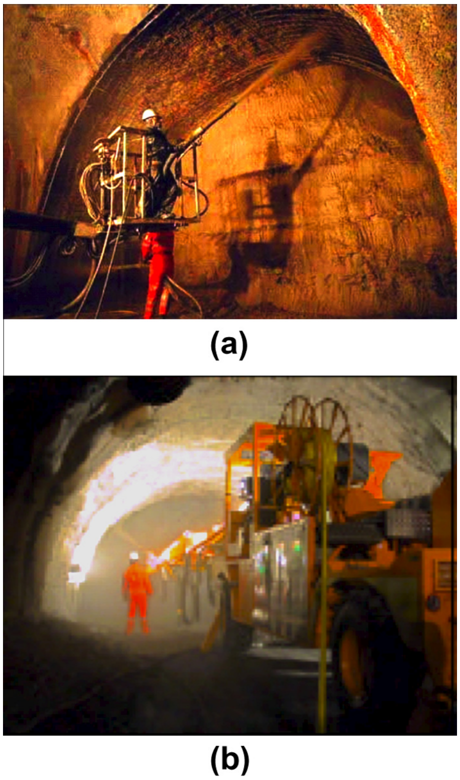


Fig. 22. Shotcrete layer applied by spraying onto reinforcement mesh.

Fig. 23. Express rail tunnel collapse at Heathrow airport 1994.

lished a report on the safety of NATM which revealed that over 100 incidents had occurred during tunnel construction in the 30 years that the technique has been used, with more collapses known to have occurred but not to have been reported. When analysing the background of these collapses, the most important fact to emerge is that the large majority of NATM tunnel collapses have occurred during construction. Moreover, the principle collapse in NATM is the face failure, i.e. collapses have occurred only at the face where the lining is still weak and cantilevered. Completed, correctly constructed NATM linings have almost never failed. However, the design of NATM tunnels relies heavily on the available data of the soil conditions [31,32]. It has been claimed, that designers of NATM tunnels often had insufficient data to work with, and in fact, most NATM collapses are connected with 'unexpected ground conditions'.

8. Concrete repair

Concrete structures are routinely exposed to freeze/thaw cycling, abrasion, chemical spillage, and thermal cycling. When subjected to such aggressive environments, concrete surfaces will degrade. This degradation may advance to a state where

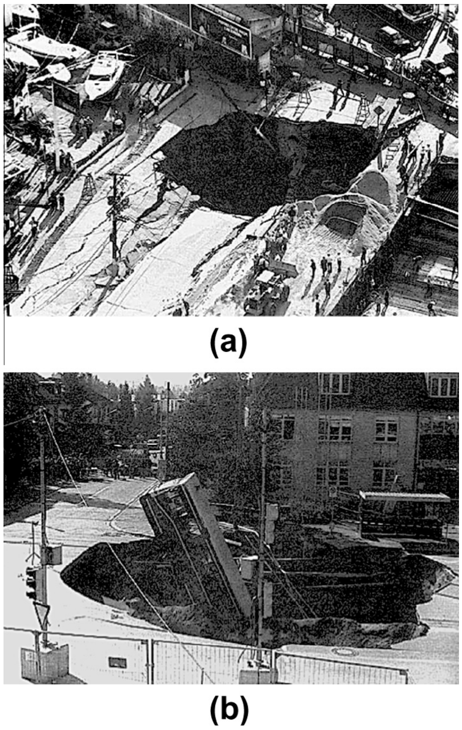


Fig. 24. Tunnel collapse in Sao Paulo Brazil 1993 (a) and Munich Germany 1994 (b).

the structure is rendered unserviceable. Nowadays, it is possible to undertake repairs that will restore the surface to a satisfactory operational standard. There are many causes of concrete failure and many methods available for the repair of failures. Effective repair requires a rational, analytical process that begins with diagnosing the reason for the failure, and using this information to select materials and methods that best meet the requirements for the repair. If the repaired area is not resistant to the original cause of failure, the repair will fail, or, the damage will be extended to adjoining parts of the structure in other words, a successful repair must be capable of resisting the stresses or agents that caused the original damage. When the principal cause of deterioration has been diagnosed, the removal of defective concrete, selection of appropriate repair materials and methods should be based on;

(a) Properties of the repair materials.
(b) Compatibility of such materials with the substrate concrete.
(c) Stability under service conditions.

In short, a repair method must be selected to be compatible with the physical and chemical properties of the concrete to be repaired. In addition, the constraints of service and application conditions must be considered.

There are two types of repair procedures that can be instigated; correction of existing deterioration of serviceability; and repair to damage of an aesthetic nature. Serviceability repairs restore surfaces to a satisfactory operational standard while cosmetic patching restores the concrete surface to a more pleasing appearance.

There are three main categories of surface repair material: polymer resinous mortar, polymer-modified cementitious mortar and plain cementitious mortar. The choice of the appropriate concrete repair material will depend on the service conditions (high traffic or impact loads, temperature, etc.) and the working conditions at the time of repair. Each category has specific physical properties (Table 6) [33]. It is important to understand these properties, as this will allow selection of a repair system that will match, as closely as possible, the properties of the concrete to be repaired. This can be achieved by examining the original mix design records (if possible) and/or taking core samples to determine compressive strength, porosity and chloride content to gauge electrochemical compatibility with prospective repair systems [31].

Effective concrete repair should return the structure to a good condition rating, with many more years of service without the need for major intervention. After any repair is completed however, it must be monitored on a regular basis to ensure that it is both durable, and that no damage to the adjacent concrete is occurring.

Table 6
Properties of typical concrete repair materials.

Repair system property	Polymer resin mortar	Polymer-modified cementitious mortar	Plain cementitious mortar
Compressive strength (MPa)	50–100	30–60	20–50
Tensile strength (MPa)	10–15	5–10	2–5
Modulus of elasticity (GPa)	10–20	15–25	20–30
Coefficient of thermal expansion (per °C)	$25–30 \times 10^{-6}$	$10–20 \times 10^{-6}$	10×10^{-6}
Maximum service temperature (°C)	40–80	100–300	>300

9. The versatility of concrete

An appreciation of the structure, mix variation and bonding will allow an understanding of service behaviour of concrete. Furthermore, awareness of the nature of concrete reinforcing, pre-stressing and post-tensioning will provide an insight to load bearing capacity and service performance. Structural integrity can be maintained by an appreciation of environmental (corrosion) issues that may compromise the natural protection of steel in concrete. However, to appreciate the versatility of concrete as a structural material, attention should turn to Universities in North America, where concrete canoes are built and raced – as described by the last case study in this chapter.

9.1. Case study: The concrete canoe

To date, engineers and students continue to advance concrete design as they experiment with admixtures such as latex, polymer-modified super-plasticisers, fly ash, high-tech aggregates and integrally coloured admixtures to develop extremely lightweight, super-strong and aesthetically-pleasing concrete mixes. One area that will stretch the imagination is in the design and build of a concrete canoe. The history of concrete boat building dates back to 1848, when Joseph Louis Lambot built thin-walled reinforced concrete boats for use at his estate in Miraval, France. Concrete was also used to build barges during World War II to replace scarce steel supplies.

Each year, final year Civil Engineering undergraduates undertake the design and construction of an optimal concrete canoe, working within constraints of a competition (Fig. 25). Teams across the continent participate, finally coming together to race their canoes against each other in a series of regional and international competitions. The final awards are judged on design, oral presentation, aesthetic qualities and race results.

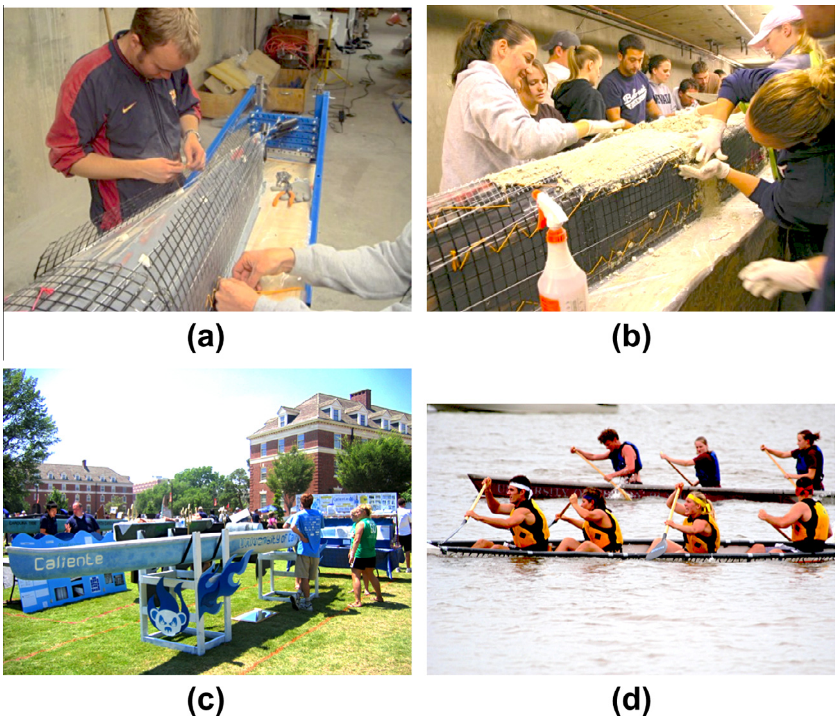


Fig. 25. Concrete canoe construction: reinforcing (a), 'pouring' or casting (b), technical judging (c), the race, and (d).

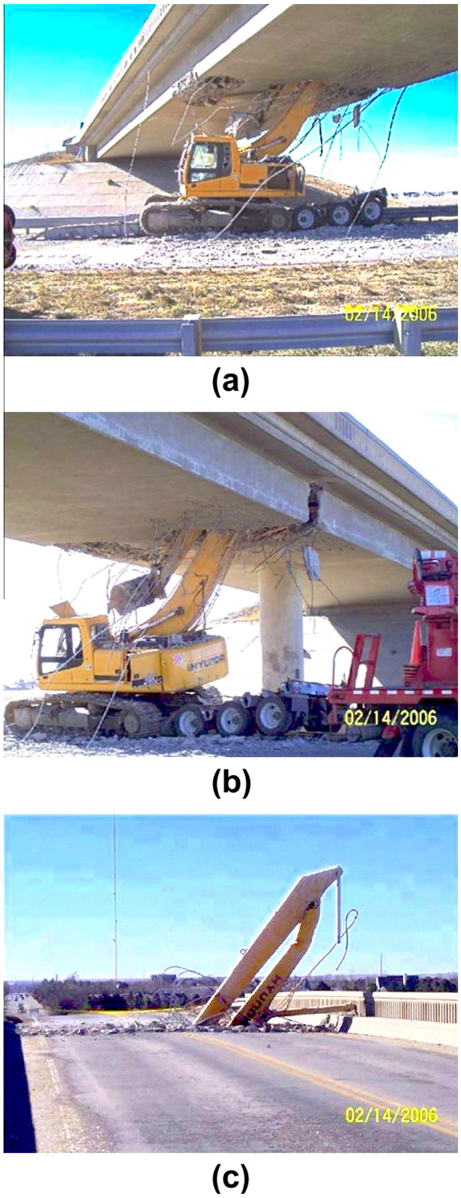


Fig. 26. It is impossible to design for every eventuality !!.

When considering the design element alone, there are three important steps within the process:

1. Concrete mix design and testing

A mix of concrete needs to be created that is lightweight while also having a high compressive strength. Therefore various types of materials are mixed and tested to find the strongest mix.

2. Reinforcement design and testing

While needing to be lightweight enough to float, the boat needs to be reinforced to maintain strength and structure within the hull. Various types of reinforcement such as carbon fiber, armor mesh and steel fiber mesh are researched and tested.

3. Hull design/mould construction

Hull design also plays an important part in the overall process. The hull must be made to go through the water quickly and resist the stress and strain from various loadings.

Generic computer programs are employed to analyse the efficiency and safety of the canoe. This will include structural analysis by finite elements (FEA) to determine stresses the canoe will experience when in the water and computational fluid dynamics (CFD) to calculate the drag the canoe will face during races. This information forms a feed-back loop to revise the original design and to improve performance within a reasonable factor of safety. Different shape characteristics are researched using 3D computer aided design (CAD) software, and it serves as a basis for the analyses of the canoe. Finally, the computer model of the canoe hull is translated into a physical mould on which the canoe will be cast. The prepared 3D model is instructs a milling machine, forming pieces of styrofoam that will make up the body of a mould.

10. Designing for every eventuality??

As a parting thought, it may be impossible to design for every eventuality. A backhoe weighing 8 tons was being transported on top of a flatbed trailer. The combination was heading east on Interstate 70 near Hays, Kansas, USA. When the shovel arm hit the overpass, it sliced almost halfway through the reinforced deck (Fig. 26). The overpass deck was constructed from commercial-grade concrete, reinforced with 37 mm diameter steel rebar spaced at 15 cm intervals in a criss-cross pattern layered in 30 cm vertical spacing. Strong enough to support the passage of a tank regiment, but not the cutting action of a backhoe.

References

[1] Hendrik van Oss G. Prepared for the US geological survey. Mineral commodity summaries; 2007.
[2] Professor Dr. Joseph Davidovits. The history of concrete and the Nabataeans. <http://nabataea.net/cement.html>.
[3] Lechtman Heather, Hobbs Linn. Roman concrete and the roman architectural revolution. In: Kingery WD, (editor) Ceramics and civilization volume 3: high technology ceramics: past, present, future. The American Ceramics Society; 1986.
[4] Kennedy HL. The function of entrained air in concrete. J Am Concr Inst 1943;14:529–42.
[5] Powers TC. The air requirement of frost-resistant concrete. Proc Highway Res Board 1949;29:184–202.
[6] Gokce A, Nagataki S, Saeki T, Hisada M. Freezing and thawing resistance of air-entrained concrete incorporating recycled coarse aggregate: the role of air content in demolished concrete. Cem Concr Res 2004;34(5):799–806. May.
[7] Hanson JA. Effects of curing and drying environments on splitting tensile strength of concrete. Development Department Bulletin DX141, Portland Cement Association; 1968. p. 11.
[8] Feld J, Carper K. Construction failure. 2nd ed. New York, NY: John Wiley & Sons; 1997.
[9] Kaminetzky D. Design and construction failures: lessons from forensic investigations. New York, NY: McGraw-Hill; 1991.
[10] Ross S. Construction disasters: design failures, causes, and prevention. New York, NY: McGraw-Hill; 1984.
[11] Lew H, Fattel S, Shaver J, Reinhold T, Hunt B. Investigation of construction failure of reinforced concrete cooling tower at Willow Island, W.V. Washington, DC: US Department of Labor, OSHA/National Bureau of Standards; 1979.
[12] ASTM international specifications for rebar: A615/A615M-05a: standard specification for deformed and plain billet-steel bars (grades 40 and 60/soft metric grades 420 and 520). A616: standard specification for rail-steel deformed bars (grades 50 and 60). A617: standard specification for axle-steel deformed bars (grades 40 and 60). A706/A706M-96b: standard specification for low-alloy-steel deformed and plain bars (grade 60 only). EN/ISO 17660 – welding of reinforcing steel. BS4449: 2005 steel for the reinforcement of concrete weldable reinforcing steel, bar, coil and decoiled product. BS4482: cold reduced wire for the reinforcement of concrete. BS4483: 2005 steel fabric for the reinforcement of concrete. BS8666: specification for scheduling, dimensioning, bending and cutting of steel reinforcement for concrete. BS7123: specification for metal arc welding of steel for concrete reinforcement.
[13] Commission of inquiry into the collapse of a portion of the de la Concorde Overpass. Library and Archives Canada, Gouvernement du Québec; 2007 [ISBN 978-2-550-50961-5].
[14] Thornton CH, DeScenza RP. Construction collapse of precast concrete framing at Pittsburgh's Midfield Terminal. In: Rens KL, (editor) Forensic engineering: proceedings of the first congress. ASCE, New York, NY; 1997. p. 85–93.
[15] Ho Christian. Paris air terminal collapse report. Architecture week; April 2005. p. N1.1
[16] Woodward RJ. Collapse of a segmental post-tensioned concrete bridge. Transportation research record, vol. 1211. Transportation research board. Washington, DC: National Research Council; 1989.
[17] Raiss ME. Recent developments in the United Kingdom in the design and specification of durable post-tensioned concrete bridges. In: Transportation research board 74th annual meeting. Washington, DC; January 22–28, 1995.
[18] Costa António, Appleton Júlio. Case studies of concrete deterioration in a marine environment in Portugal. Cem Concr Compos 2002;24(1):169–79.
[19] Beaudoin JJ. Fibre-reinforced concrete. Canadian building digests, CBD-223; April 1982.
[20] Majumdar AJ, Ryder JF. Glass fibre reinforcement of cement products. Glass Technol 1968;9(3):78–84.
[21] Majumdar AJ, Nurse RW. Glass fibre reinforced cement. Mater Sci Eng 1974;15:107–27.
[22] Cheetham CJ, Maguire P. Coating of glass fibres. US patent 4,173,486; 1979.
[23] Building research station. A study of the properties of Cem-Fil/OPC composites. Building research establishment current paper, CP38/76. Garston, England; 1976.
[24] Kodur VKR. Spalling in high strength concrete exposed to fire – concerns, causes, critical parameters and cures. In: Proceedings, ASCE structures congress. Philadelphia, PA; 2000.
[25] Ulm FJ, Acker P, Levy M. Chunnel fire. II: analysis of concrete damage. J Eng Mech 1999;125(3):283–9.
[26] Kodur VKR, Sultan MA. Structural behaviour of high strength concrete columns exposed to fire. In: International symposium on high performance and reactive powder concretes. Sherbrooke, QC; 1998. p. 217–32.
[27] Lie TT, Woollerton JL. Fire resistance of reinforced concrete columns: test results. Institute for research in construction, National research council of Canada. Internal report No. 569, Ottawa; 1988. p. 302.
[28] Bilodeau A, Malhotra VM, Hoff GC. Hydrocarbon fire resistance of high strength normal weight and light weight concrete incorporating polypropylene fibres. In: International symposium on high performance and reactive powder concretes. Sherbrooke, QC; 1998. p. 271–96.
[29] Tunnel lining removal prompts LA metro collapse. New Civil Engineer; 29 June 1995. p. 3–5.
[30] Health and safety executive. Safety of new Austrian tunnelling method (NATM) tunnels. A review of sprayed concrete lined tunnels with particular reference to London Clay. Sudbury, HSE Books; 1996.
[31] Health and safety executive. Post-construction audit of sprayed concrete tunnel linings. Sudbury, HSE Books; May 1996.
[32] Institution of civil engineers. Sprayed concrete linings (NATM) for tunnels in soft ground. London, ICE Design and Practice Guide; April 1996. p. 104.
[33] Mays G, Wilkinson W. Polymer repairs to concrete: their influence on structural performance. ACI-SP 100-22 Katherine and Bryant Mather international conference; 1987. p. 351–38.