Materials and structures

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Abstract

The collapse of the World-Trade Center towers, on September 11, 2001, has raised questions about the design principles in high-rise buildings. In this article, we first consider the likely failure mechanisms that may have ultimately led to the collapse of the Twin towers. This analysis is based on a materials-to-structures approach, in which we look both at the characteristic behavior of the construction materials and the design details of the buildings. The very fact that the buildings survived the crash of the planes into the buildings suggests that a time dependent behavior at the material level affected the structural stability of the structure to the point of failure. On the other hand, the failure per se reveals the existence of a weakest link in the structural system, which ultimately failed because of a lack of redundancy. We then turn to the question whether —from an engineering point of view— skyscrapers will continue to have a future in the 21st century despite the increased vulnerability of our mega-cities. New materials-to-structures engineering solutions are also discussed, which in time could provide a new technology of redundancy to ameliorate the vulnerability of critical engineering structures.

Introduction

The terrorist attack of September 11, 2001 at New York's World Trade Center towers (WTC) (Figure 1) was the first attack on a mega-city in the 21st century. The collapse of the towers revealed the vulnerability of a mega-city to terrorist attacks at multiple scales, from the level of structural components to the collapse of the towers, from the scale of individual heroic rescue operations to the scale of mass evacuation and emergency operations, from the interruption of local transportation systems to the freeze of air traffic nation wide. Everyone who lived through the day at Ground Zero can continue the list: This was not a day for business as usual!

Figure 1: World Trade Center Towers (Photo from AP)

As the WTC towers sunk to Ground Zero and below, the logic of a world collapsed: a building designed to rocket into the sky, imploded into the ground. Ever since that day, structural engineers all over the world seek for explanations as to how and why the towers collapsed, and how to prevent such failures in the future. Of course, in theory, it is possible to engineer a structure to withstand a devastating attack whether accidental or intentional. For instance, eight years before, on February 26, 1993, a bomb detonating in the parking area of the WTC did not challenge the stability of the structure, unlike the event of September 11. Roughly two hours after the impact of two planes into the towers, the icons of strength and prosperity of New York that had been standing there for almost three decades, disappeared almost instantly from the Manhattan skyline, transforming the 110-story towers into a big pile of debris a few stories high. Ever since, the question is raised whether our skyscrapers are safe considering the events which proved the limits of predictability, anticipation and prevention. To answer this question, from a structural engineering point of view, we first need to reconstruct, as much as possible, the sequence of events that led to the collapse of the towers.

How did the towers collapse?

Initial assessment of the collapse

On September 11, the first Boeing 767-200 aircraft hit the North Tower at 8:46am, near the center of the North face at about the 96th floor. The South Tower was hit at 9:03am by another Boeing 767-200 aircraft near the southeast corner of the building at about the 80th floor (Figure 2). In both cases, the planes appeared to have sliced into the buildings and exploded immediately after penetration. Smoke clouds discharged heavily from the impact face as well as the side faces of the buildings. In both cases, destruction looked local, and appeared at first not to have challenged the structural stability. People tried to escape from the impact area, while some were unfortunately trapped in the floors above the impact zones due to damaged egress routes and/or raging fuel fire.

The South Tower collapsed suddenly at 9:59am, 56 minutes after the impact. Tilting occurred in the upper portion (Figure 3), which was immediately followed by a total collapse top down in about 10-12 seconds. The North Tower collapsed at 10:28am in a very similar fashion, 102 minutes after the impact. Figure 4 shows the collapsed building with the perimeter

Figure 2: Boeing 767 aircraft approaching the South Tower (www)

Figure 3: Progressive collapse of the South Tower (Photos from AP)

steel columns several stories high still linked together at the lower levels. In the collapse of the two WTC towers, a three-step failure mechanism may have been involved at different scales:

Step 1 – Impact of the airplane:
The buildings had been designed for the horizontal impact of a large commercial aircraft. Indeed, the towers withstood the initial impact of the plane. This is understandable when one considers that the mass of the buildings was about 2500 times the mass of the aircraft, and that, as has been reported, the buildings were designed for a steady wind load of roughly 30 times the weight of the plane. The impact of the plane was instantaneously followed by the ignition of perhaps 40 m^3 of jet fuel. While a fully fueled Boeing 767-200 can carry up to 90 m^3 of fuel, the flights initiated from Boston may have carried perhaps half of this amount, comprising about one-third of the airplane's weight. The impact and the ensuing fireball definitely caused

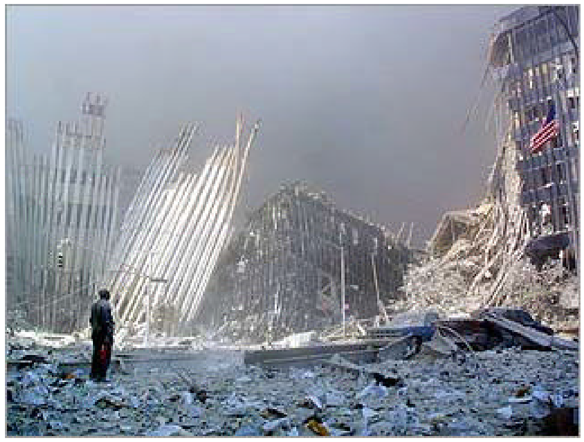


Figure 4: Collapsed tower with perimeter columns still linked at the bottom floors (www)

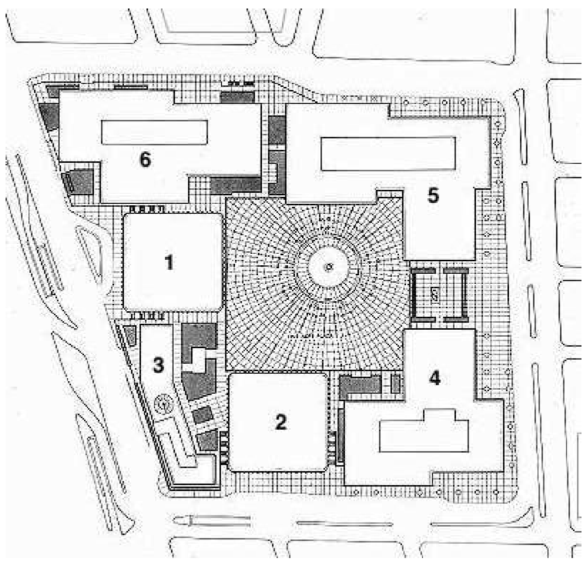
severe local damage to the building and, in fact, destroyed some perimeter and core columns across multiple floors. It has been argued that the damage to several floors should have overloaded the remaining intact columns in the damaged floors affecting their resistance to buckling. Yet, their resistance was sufficient to carry the loads of the upper floors almost one hour in the South Tower and almost double that much in the North Tower.

Step 2 – The failure of an elevated floor system:
The fireball following the impact may have destroyed some of the thermal insulation of the structural steel members. The burning of the jet fuel may have easily caused temperatures in the range of 600°C-800°C in the steel. Under these conditions of prolonged heating, structural steel looses rigidity and strength. This may have caused further progressive local element failures, in addition to those failed from the initial impact, leading to a greater reduction of resistance of the connected two to three floor structural system. The load to which the column bracing system was subjected to was the weight transferred from the upper floors. At a certain stage, after some 50 minutes in the South Tower and some 100 minutes in the North Tower, the buckling resistance of the columns was reached and collapse of the columns became inevitable. Preceding this progressive failure within the damaged column-bracing system, the floor decking system may have failed first in a brittle way, releasing explosively the energy stored in the system. It has also been argued that the failure may have initiated by shearing of a critical floor from the floor-external/internal column connections. In reality, combinations of floor failure with that of column buckling may have occurred simultaneously. In fact, failure of a floor system would result in an instant loss of lateral column bracing, leading in turn to loss of column stability. The tower with the higher load on top (the South Tower) collapsed first; but both towers exhibited nearly identical failure mechanism.

Step 3 – Dynamic crash of the structure:
The failure of the floor system led to a free fall of a mass of approximately 30 stories and 14 stories onto the 80 and 96, respectively, floor structure below. The enormous kinetic energy released by this 2-3-floor downfall was too large to be absorbed by the structure underneath. The impact effect generated from this upper part onto the lower part was surely much higher than the buckling resistance of the columns below, which to this point may have been

essentially undamaged and were not affected by fire. The impact caused explosive buckling, floor after floor, of the WTC towers with the debris of the upper floors wedging with the lower part of the structures. As the floors failed, the collapse of the building accelerated downwards with the accumulation of the falling mass and the dynamic amplification of its impact on to the lower structure. Similar to a car crash in a wall, the towers crashed into the ground with a velocity close to that of a free fall.

While the first and the third step to failure are focus of two other contributions in this book, the initiation of the collapse of the WTC is still not clear. More precisely, the two key observations that deserve more attention are (1) the time elapsed between airplane impact and collapse, and (2) the abrupt failure of the structure with little warning. The first suggests that there was a time dependent mechanism involved, at the material and/or structural level. The second indicates a structural stability problem, which is always associated with an abrupt



1 WTC – North Tower
2 WTC – South Tower
3 WTC – Hotel
4 WTC – South Plaza Building
5 WTC – North Plaza Building
6 WTC – US Customs House

Figure 5: Plan of the World Trade Center complex (www)

failure, in contrast to a ductile failure. Understanding the combination of these two phenomena appears to be the key to explaining the collapse of the towers. This requires, first, a look into the structural system and construction materials employed in the structure.

Overview of the WTC

The world trade center was developed and constructed by the Port Authority of New York and New Jersey to serve as the headquarters for international trade. The center was located on Church St. in Manhattan of New York City. The complex consisted of two 110-story office towers (WTC-1 and WTC-2), a 22-story luxury hotel (WTC-3), two 9 story buildings (WTC-4 and WTC-5), an eight story US Customs house (WTC-6) and 47 story office building (WTC-7). The complex was bound by West Street to the west, Vesey and Barkley streets to the north, Church street to the east and Liberty street to the south. (Figure 5). Having a rentable space of more than 12 million square feet, the complex was housing more than 450 firms and organizations and more than 60,000 people working in these firms. About another 90,000 people were visiting the complex each day, with the shopping mall located below the plaza being the main interior pedestrian circulation level of the complex.

The complex was designed by Minoru Yamasaki and Associates of Troy, Michigan, and Emerith Roth and Sons of New York. The structural engineers were John Skilling and Leslie Robertson of Wortington, Skilling, Helle, and Jackson. The site excavation had begun in 1966 and construction of the towers started two years later. The first tower (WTC-1) was completed in 1970 and the second tower (WTC-2) was completed in 1972. Figure 6 shows one of the towers under construction.

Figure 6: Towers under construction (www)

Figure 7: View of the bathtub (www)

The WTC buildings were supported by gigantic foundations. They rested on bedrock 21m (70ft) below ground. In the area that contained the twin towers, more than a million cubic yards of earth and rock were removed to place a basement that was 299m × 155m × 21m (980ft × 510ft × 70ft). The basement housed a commuter rail station, a 2000 car parking area, mechanical equipment rooms, and storage. Prior to excavation, underground walls were built all the way down and into the bedrock to withstand the external water and earth pressure, and

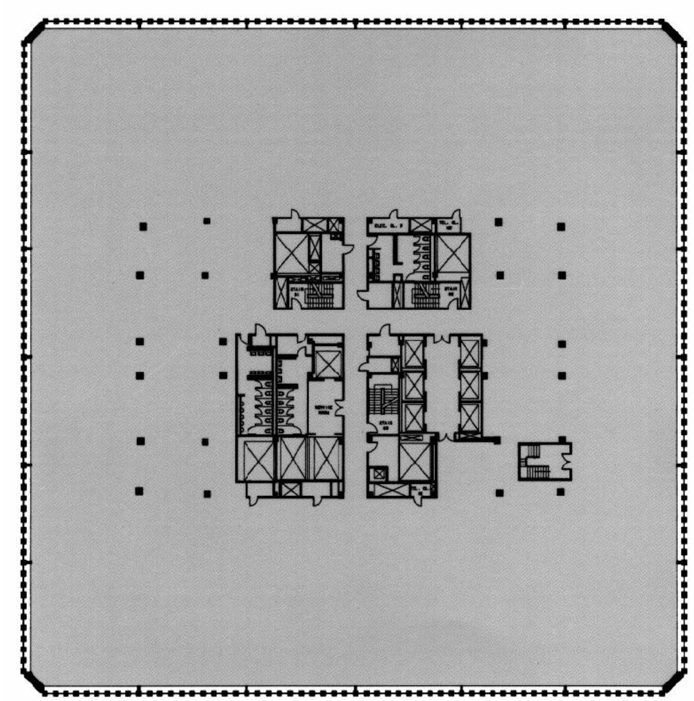


Figure 8: Typical floor plan (Hart et al., 1985)

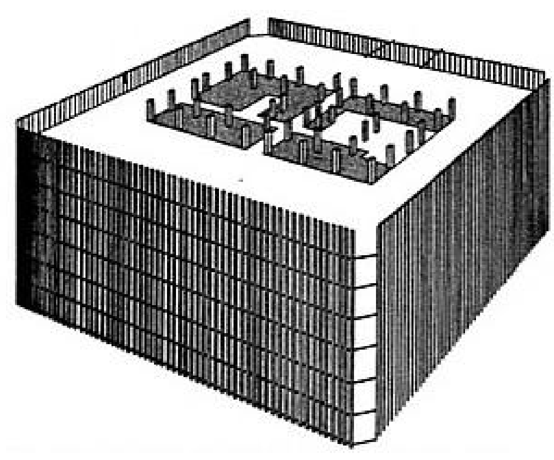


Figure 9: A conceptual view of the structural system (Hart et al., 1985)

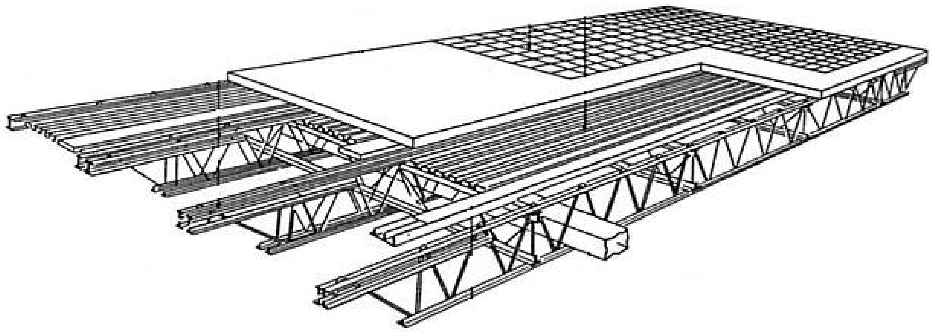


Figure 10: Conceptual view of floor system (Hart et al., 1985)

to prevent the undermining of adjacent buildings and streets. These walls were 7 story high, heavily reinforced concrete walls. The completion of the walls around the entire eight-block area resulted in a cutoff boundary around the site to be excavated. The excavated area, which is generally referred to as the "bathtub", is shown in Figure 7.

Structural system

The twin towers were built as a steel tubular structural system that differed radically from other structures of that time. The exterior walls were built of closely spaced steel columns to perform as load bearing walls and the interior columns were located only in the core area containing the elevators. The outer walls carried the vertical loads and also provided resistance to lateral effects such as wind, earthquake, and impact. Figure 8 shows a view of the exterior wall.

The towers were square in plan with sides of 63.7m (209ft). The structural height of each tower was 415m (1362ft). The height to the top floor was 411m (1348ft). The towers were built as framed tube cantilever structures with 0.45m wide built-up box columns (Figure 9) tied with 1.3m deep spandrel beams in the perimeter. The beams and columns were pre-fabricated into panels and assembled on site in a staggered fashion by bolting and welding. The perimeter member assembly made of 59 columns over the 63.7m-wide façade ensured the load bearing capacity of the outer skin for gravity load, lateral load, and torsional effects. The columns were spaced 1m apart and spandrels 3.6m apart. The 24m $\times$ 42m core was composed of 44 box columns. The core comprises steel beams and columns with reinforced concrete infill panels designed to share part of the gravity loads. The core was designed to resist vertical loads and was not assumed to transfer any lateral loads. The perimeter columns were tied to the core only by the truss-slab system and the horizontal forces were assumed to be resisted by the perimeter columns and their connecting spandrel beams. A typical floor plan is shown in Figure 10. The isometric view shown in Figure 11 helps conceptualizing the structural system.

The slab system consisted of primary vertical bar trusses spaced 2m apart spanning 20m from the core to the perimeter (connected to every other column). These primary trusses were braced by orthogonal secondary trusses. Figure 12 shows the original drawing of the floor system details. A conceptual view of the floor system is shown in Figure 13. All trusses were built up by four angle sections to form a top cord, two to form a bottom cord, and bent round bars to form the diagonals of a classic warren truss. The bars were sandwiched between and welded to the angles. The bent bars protruded above the upper angle sections and into the 10 cm thick concrete floor to act as a shear key. Trusses were connected at their ends by bolts.

The connection of each truss to the external columns was made by means of a truss seat (Figure 14), which was connected to the box columns. The truss seat was a built up section onto which the two angles of the top chord were bolted with two bolts. Connection of the truss to the core was made by bolting the bottom chord angle to a channel section, which was connected to the interior columns (Figure 15). The bolted connections were of friction (or slip-critical) type, 16mm – 19mm (indicating diameter of the bolt) A325 bolts possessing a tensile strength of about 110ksi were used. Corrugated steel decks were then secured on the orthogonal trusses, and 10cm lightweight concrete topped the decks to complete the slab. The corrugated steel decking acted as permanent formwork and as a composite with the concrete to support the floor loads. It is noted that at a later stage, viscoelastic dampers were attached to the ends of each floor truss connecting the lower truss chords to the perimeter box columns in order to reduce wind induced vibrations.

Structural and fireproofing materials

The major structural material employed in the towers was A36 structural steel, although higher strength steel was used in the lower elevations of the structure. Except for some selected floors, for which normal strength concrete was employed, the composite slabs were made of a 21MPa (3ksi) lightweight concrete.

Fire resistance of the perimeter columns was provided by a layer of sprayed concrete around the three sides of each column. The concrete layer had a thickness of about 5cm and included ceramic fibers in the mix. The interior face of each column was fire protected with approximately 5cm thick layer of vermiculate plaster (Figure 16). The exterior sides of each perimeter column were covered by aluminum to which the window frames were fixed. It has been reported that passive fire protection was provided to the underside of the floor systems by a fire rated suspended ceiling. Specifics of fireproofing implemented on these buildings including which structural members were treated and to what level of fire resistance are still being investigated.

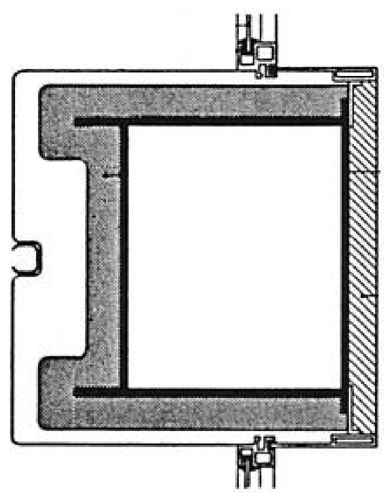


Figure 11: Fire proofing of external columns (Hart et al., 1985)

Could the impact have been the primary source for the collapse?

The penetration of the two aircraft into the towers seems to suggest that the primary source of the collapse of the building was the impact of the airplanes. There are several indications that support this view.

The first point relates to the load for which the structure was designed. According to Leslie E. Robertson Associates, the structural engineering consultant who engineered the buildings, both towers were designed to resist the impact of a Boeing 707. Such design was deemed necessary for the skyscrapers due to the possibility of having an aircraft crashing into them under inclement weather conditions. This was not without precedence; a B-25 bomber crashed into the Empire State Building in 1945 on a foggy morning. It has been argued that the damage inflicted by the Boeing 767 was far more substantial than the one of a Boeing 707, for which the building was designed. Indeed, while both planes have a similar take-off weight, the design scenario of a lost airplane is quite different from that of a suicide plane intentionally hitting a building. The speed of the planes and severity of the impact, the level of penetration, the excess weight of the aircraft on the slabs after penetration, the fireball following the collision, and the weight of debris accumulating on lower levels are among the factors not considered in the design of the towers for aircraft impact.

A second argument that might be given is a structural one, relating to the specific framed tube cantilever structures of the towers. Indeed, such a structural system is based on the premise that the perimeter columns and spandrel members resist gravity and lateral loads. These loads are transformed into axial, bending, shear, and torsion stresses and deformations. The function of the core is only to share part of the gravity loads carried by and transferred from the slab system. In order to have all the members function properly as designed, continuity has to be maintained at all times so that loads can be transferred from one member to another and eventually carried down to the foundation. The impact and penetration of the airplanes disrupted the continuity of the force flow in the outer skin; and floor trusses, slabs, and core columns in the vicinity of the impact were substantially deformed and destroyed. This disruption of continuity was confirmed by people who successfully escaped and who reported having seen widening of cracks in the stairwells during evacuation. From a structural mechanics point of view, these observations indicate that a significant stress distribution took place from damaged members to undamaged parts, establishing a new force balance. As the absence of equilibrium is associated with failure, this overall force balance was maintained during the time to failure, that is the 56 minutes and 102 minutes the towers still stood after the impact of the plane. It should also be noted that the buildings, which were approximately 95% air, could not tip over as a result of the initial impact, and they essentially imploded onto themselves at a later stage. In conclusion, we can state that, although the initial impact must have caused significant local damage to several floors by the impacting aircraft slicing through the perimeter frames, the impact alone falls short as a sole explanation of the towers' collapse.

Why did the towers collapse?

Weakest link theory

A fundamental principle of engineering design theory is that a structure is only as stable as the weakest link in a chain of elements. This weakest link may exist at a material or structural level, and affects the entire structural system stability if no provisions for redundancies have been implemented in the system. In the collapse of the twin towers there is no doubt that there were many interacting factors involved that lead to the catastrophic failure. However, it can be argued that perhaps a weakest link may have played a critical role in the initiation of the failure process.

A key element in the failure process of the tower buildings was the time elapsed between the impact and the collapse, which indicates a detrimental role of a physical phenomenon that depends on time. The obvious one is related to the heat effects that started with the fireball and continued until failure. In the days following September 11, it was argued that the fire was the ultimate cause of the collapse of the towers, since it is known that steel looses strength and stiffness at high temperatures. But one can learn more by trying to reconstruct the different levels at which high temperature played an important role in the initiation of failure and the collapse mechanism.

Fire

There has been speculation with respect to the magnitude of temperature that may have resulted from burning of the jet fuel possibly leading to the melting of the steel in the WTC fire. It has been noted (Eagar and Musso, 2001) that although heat and temperature are related they should not be confused. Temperature is an intensive property, meaning that it does not vary with the quantity of the material, while the heat is an extensive property, which varies with the material volume. The two quantities are related through the heat capacity and the density. On the other hand, the dispersal of the jet fuel over several floors of the WTC did not necessarily imply an unusually hot fire. While burning hydrocarbons (jet fuel) using pure oxygen may reach approximately 3000°C, the same material burning in air produces about one third of that; that is, 1000°C. Thus the temperature experienced by the steel as a result of the fire may have been in the range of 750°C to 800°C, which is not sufficient to melt the steel. Typical value of steel melting temperature is in the range of 1400°C-1500°C. However, this level of temperature has significant effect on the structural behavior.

Behavior of steel under high temperature

Generally, unprotected steel in a high temperature environment does not perform well as a structural material due to the fact that steel has a high thermal conductivity and the members made of steel usually have thin cross sections. Typical fire proofing materials for steel structures are sprays (mineral fiber, vermiculite plaster), boards (fiber-silicate or fiber-calcium-silicate, gypsum plaster), and compressed fiber boards, (mineral wool, fiber-silicate). Typical thicknesses of insulating materials generally vary from 15mm to 50mm. Figure 17 shows two standard fire curves corresponding to combustible cellulosic material and a material of petrochemical origin. Steel temperatures for a structural beam for unprotected and protected steel together with a standard fire temperature is shown in Figure 18 (ISO, 1975; Buchanan, 2001).

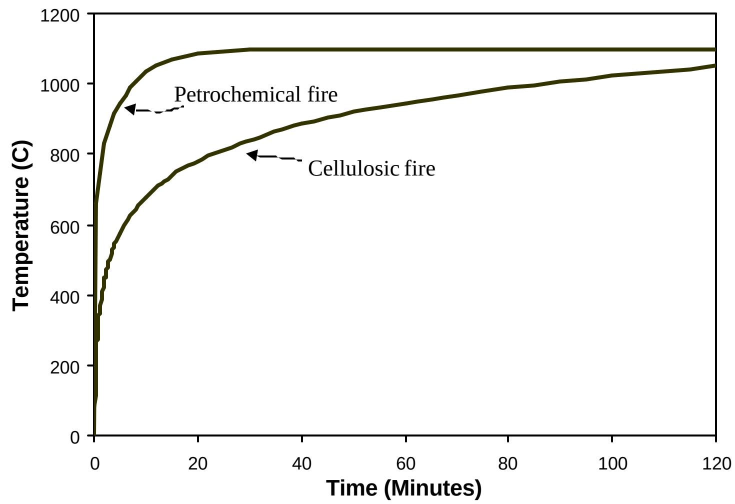


Figure 12: Fire curves (ISO, 1975)

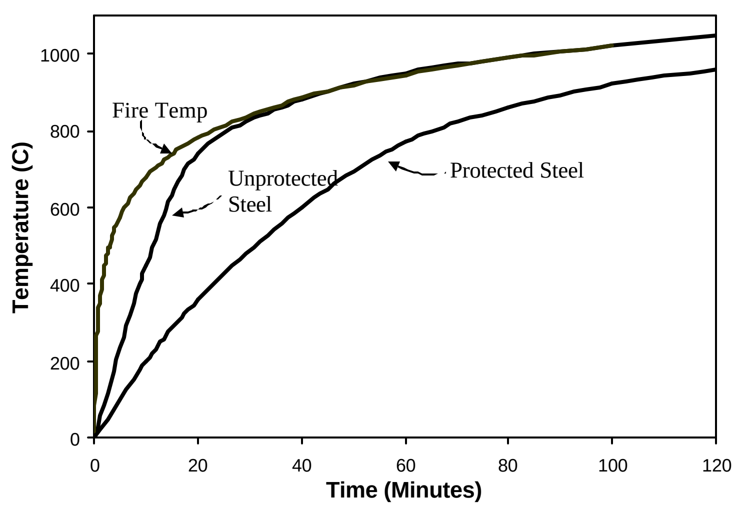


Figure 13: Temperatures for protected and unprotected beams exposed to fire (Buchanan, 2001)

We already mentioned the important role that the sustained temperature may have played during the time elapsed between the impact and the collapse in the failure process of the tower buildings. Considering the behavior of steel under high temperature one can now reconstruct the different levels at which the high temperature may have affected the building behavior.

Material level: thermal softening and thermal creep

Steel subjected to high temperature undergoes a substantial loss of strength and stiffness at a temperature level far below the melting temperature, which is referred to as thermal softening and thermal damage, respectively. By loss of stiffness (thermal damage), we mean an increase of the deformability of the material under load. A part of this increased deformability is known as thermal creep, and results from the higher agitation of the atoms of steel at high temperatures, which increases the susceptibility to and likelihood of deformation. Figure 19

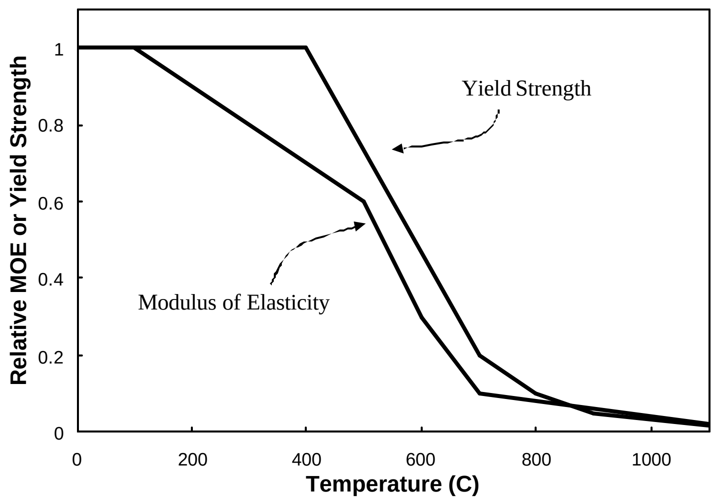


Figure 14: Reduction in yield strength and mo dulus of elasticity of steel as a function of temperature (EC3, 1995)

shows the relative strength and stiffness degradation upon increasing temperature. At a temperature level of about 600°C-700°C, which corresponds roughly to one half of the melting temperature of steel, strength and stiffness of steel are reduced to 50% and 30%, respectively, of the initial value. Still, we should note that the temperature dependence of strength and stiffness of steel is a material property, which only affects the structural response if the member is heated. This involves at least two further physical processes: heating rate and heat diffusion.

Structural level: heating rate and heat diffusion

Standard fire curves, shown in Figure 17, do not consider the initial explosion at impact. However, they can be considered as a first approximation to the rapid temperature rise to which the structural members in the towers could have been subjected after the impact. Given the rapid burning of the jet fuel, a temperature of 600°C-700°C (corresponding to about one half of the steel melting temperature) could have been reached essentially in a matter of minutes. Any structural steel member without or insufficient fireproofing (destroyed e.g. by the impact, and thus directly exposed to the high temperature) would have undergone substantial thermal damage and thermal softening. On the other hand, the fireproofing increases the thermal inertia of the member by delaying the heat diffusion into the material. It is likely that this heat diffusion, slowed down by fireproofing, was one of the rate determining mechanisms that delayed the collapse initiation in time. In fact, it can be shown, from dimensional analysis, that the critical time span t during which a steel member of thickness H with a fireproofing at its surface of thickness e, is protected by fireproofing (see Figure 20) is scaled by:

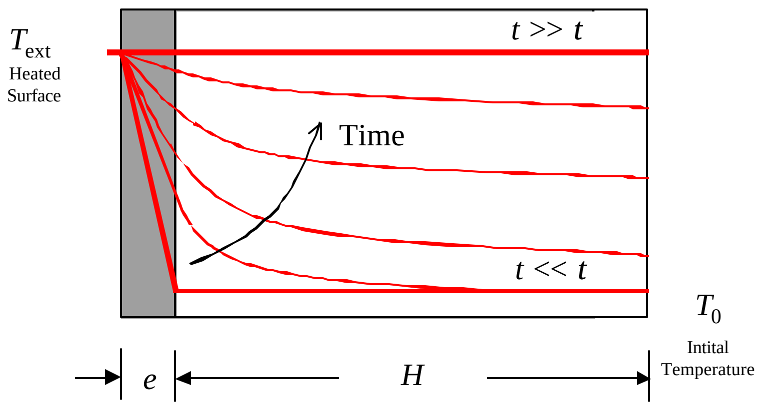


Figure 15: Temperature Profiles Through a Cross Section with a Fire Proofing Layer

$$t = \frac{H^2}{D} \times F\left(\frac{e}{H}, \frac{H}{k/l}\right) \qquad 1)$$

where F is a dimensionless function of the arguments e/H and Hl/k; $D = k/(rc)$ is the thermal diffusivity of steel, k the thermal conductivity, c the specific heat capacity, r the mass density; and l the thermal exchange coefficient of the fire proofing. The smaller l, the more efficient the fireproofing. For times $t << t$ the steel member, coated by a fire proofing layer, will not feel the external temperature, but for $t >> t$, the steel over its entire thickness will be at the external temperature. This time is inverse proportional to the heat diffusivity of steel. Note that k/l has the dimension of length, which needs to be compared to the structural dimension H. In fact, for a given steel member of size H and conductivity k, an efficient fireproofing must be such that $l << H/k$. Hence, the smaller H and the higher the conductivity, the smaller the required fireproofing heat exchange coefficient. The high heat diffusivity of steel, which is 5 times that of air, and the high conductivity of steel, which is some 20,000 times that of air (see Table 1), combined with the generally small characteristic dimensions of steel members, highlights the high vulnerability of steel members to high temperatures.

Structural performance of columns and slabs under high temperature

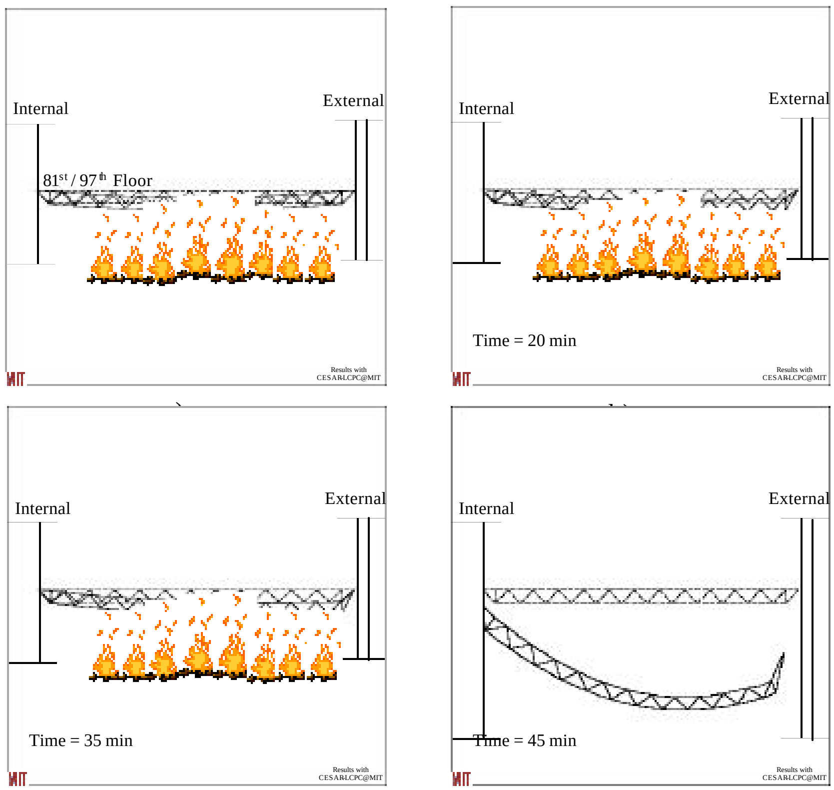
Thermal damage and thermal softening are material properties, and heat diffusion occurs at the sectional level of the steel member. The missing link in the initiation of the collapse is the

Table 1: Physical parameters of materials at 20°C

Material	r [kg m^{-3}]	k [W m^{-1}K^{-1}]	c [W s kg^{-1}K^{-1}]	D [m^{2}s^{-1} $\times$ 10^6]
Steel	7,800	45	420–510	11.3 – 13.7
Concrete	2,500	6–8	840–1,000	2.4 – 3.8
Air	1.2	0.0026	717–1,005	2.1 – 3.0

structural performance of the structural members subjected to heating. To this end, at the member level, we shall distinguish the slab system from the columns.

The slab system carries the load primarily in bending. From past-fire experiences in factories and buildings, it is well known that bending members subjected to fire undergo large deformations. Such a ductile response is readily understood from the fact that in steel, the loss of strength occurs faster than the reduction in stiffness. In other words, with heating, the structural member reaches the yield limit of the steel faster, and as a result, undergoes a large plastic deformation. Such a ductile deformation mode, involving large plastic deformation, could hardly have occurred in the towers. In fact, such a ductile deformation mode would have further delayed the structural collapse, or would have even prevented it. By way of conclusion, it appears that the slab could not have undergone a uniform thermal softening and thermal damage, which would have essentially led to a ductile failure of the slab system.



c) d)

Figure 16: Finite element simulation of the floor collapse from end joints

Figure 21 (a), (b), (c), and (d) displays the results of finite element simulations of the slab truss system subjected to non-uniform heating. This non-uniform heating may have resulted from locally damaged or insufficient fireproofing and the higher thermal damage of the high-strength bolt connections with the outer façade. In the numerical study, the non-uniform heating effect is taken into account by a 100 times higher thermal exchange coefficient (see eq. (1)) of the end diagonal truss at the external façade, thus considerably reducing the fire protection time. As the results show, this weak point in the fireproofing system could actually be the weakest link of the entire system: while uniform heating would have led to a uniform bending and yielding (see Figure 21 (a)), an increased local thermal damage of one structural element leads to a rigid body motion of the statically determinate truss system (see Figure 21(c)), which ultimately causes failure of the truss system. This weakest link situated at the end supports, to which the steel truss system was mounted by bolts, may explain the failure of the truss system in a shorter time span than the nominal fireproofing time. This finding based on model-based simulation is consistent with forensic studies carried out on the WTC-site after Sept 11, which showed that several of the end supports were either strongly deformed (indicating exposure to very high temperatures), or perforated by the bolts.

With still limited knowledge, the most likely scenario that may have triggered the collapse of the WTC towers is the local failure of the support structures of the slab, initiated most probably through an insufficient fireproofing or a higher thermal damage and softening of the bolts. The local failure of one or several supports could have caused a zipper effect leading to the loss of the slabs load bearing capacity. Upon failing of a floor system, the lower floor had to carry the additional weight. While the trusses may well have been able to carry the load, the supports are typically designed for twice or three times the nominal weight. Hence, if we assume that one or two floor systems were already destroyed by the impact, it suffices one additional floor failure, and/or the dynamic amplification effect, to make the lower floor fail. However, this is not yet the complete scenario leading to total collapse.

The last missing link is the failure behavior of the supporting core column system. These core columns were designed primarily to carry the vertical load from the slabs to the ground. In the absence of horizontal forces, columns are designed in a way that the applied normal force N is always smaller than the maximum admissible (buckling) force F^b, at which the column looses (almost) instantaneously its capacity to carry load. From classical column stability problem, expressed in terms of a safety factor $\gamma = F^b/N$, the structure will keep its load bearing capacity provided that:

$$g = \frac{EI}{a^2 L^2 N} > 1 \tag{2}$$

where E is the elasticity modulus, I the moment of inertia of the column section, L is the length of the column between two horizontal supports provided by the intact bracing slab systems, and $\alpha \in [0.5,2]$ is a coefficient relating to the end bearing conditions of the slabs. The length magnitude (αL) is generally referred to as "effective length". At the level of impact, it is likely that γ was much larger than unity, typically 5–10. The three factors that may have affected the collapse are, with decreasing importance:

- The increased effective length of the columns: Once a slab system failed, the distance between the horizontal supports by the slabs doubles, thus decreasing γ by (at least) a factor of 4 for the first slab, 9 for the second slab, and so on. Failure of two or three floors would be sufficient to bring the column load to the critical buckling value, that is γ=1, leading to the collapse of the columns.
- The initial load in the columns: The axial force in the columns at the impact floor is roughly proportional to the load of the floors above the impact floor. Thus, the axial force at the 80th floor of the South Tower having 30 floors above was roughly twice as much as the one at the 96th floor in the North Tower (with 14 floors above). With regard to this initial load, the additional load due to failing slab system is quite small (roughly 1/14th of the initial force in the South Tower, 1/30th in the North Tower), and can be excluded as a major contributor to the buckling failure.

- The thermal damage of the columns: the reduction of elastic stiffness, as a function of temperature, T, i.e. $E=E(T)$ linearly decreases the safety factor γ.

In addition to these factors, the important effect of dynamic amplification of the impact loads should be considered. This aspect is covered elsewhere in this book.

Given the higher initial load of the South Tower columns, it is likely that the longer time to failure of the North Tower of 102 min (versus 56 min of the South Tower) may have well involved some substantial thermal damage of the columns prior to failure. But it is more likely that an additional failure of one slab system occurred. Indeed, for all structural and material parameters constant, buckling in the North Tower will occur, theoretically, for a buckling length of $L_{North}/L_{South} = (N_{South}/N_{North})^{1/2} \approx 1.4$, where L_{South} is the buckling length which made the South Tower fail first, and $N_{South}/N_{North} = 2$ is the axial load ratio between South and North Towers. Thus, the lower initial load in the North Tower, which translates into a higher buckling length, made the North Tower gain 46 minutes of time for evacuation. These 46 minutes compare well with the characteristic time scale of failure of one slab system due to heat effects at the end supports (see Figure 21(a), (b), (c), and (d)). It then appears, indeed, that the North Tower collapsed once an additional floor had failed, indicating some redundancy of the failure mechanism. This confirms that the key to understanding the failure is the time dependence of the failure mechanism of the weakest link in the system.

Could the collapse of the buildings have been prevented?

The world trade towers were ingeniously designed for the physical and social reality prior to September 11, 2001. In fact, it appears to us that the structure as a general system was built with high level of redundancy against failure. The towers did not significantly tilt throughout the failure which no doubt avoided an even greater catastrophe and destruction far beyond lower Manhattan. They withstood both the initial impact of the aircraft and the resulting fire balls. The preceding analysis indicated that the collapse mechanism of the towers involved failure of the floor system from heat affected joints with the ensuing domino effect of progressive collapse, one could cite the perceived weaknesses at several levels: a) the end joints of the floor systems, which involved rather simple bolt connections, b) the fire proofing of the joints of the floor trusses, c) the transfer of internal forces among the elements, (lateral and vertical members) within the external tube system.

The tower structures were built with a breakthrough innovation, given the physical and social realities at the time of their construction, in creating a highly redundant and efficient system for external effects. It is ironical that 30 years later the very same structures had to collapse by imploding onto themselves through primarily a local mechanism within the strong external envelope.

Could the use of concrete have prevented the collapse?

The answer to this question requires, first, an analysis of different levels at which collapse was initiated. Concrete is non-combustible, and has a low thermal conductivity compared to steel; but this alone does not explain the better fire performance of concrete compared to that of steel.

In fact, on a purely material level, thermal damage and softening of normal concrete is quite similar for concrete and steel, although the involved chemo-physical mechanisms are quite different. In contrast to steel, the thermal damage of concrete is due to several sources: a differential thermal expansion behavior between the cement paste matrix and the aggregate inclusion, thermal instability of some mineral components of hydrated cement at some 400 °C, transformation of aggregates at some 800°C, and so on. The thermal softening of concrete results in addition from a dehydration of concrete, leading to a loss of strength of the material. Typical curves of thermal damage and thermal softening for steel are shown in Figure 19 and thermal damage and thermal softening of concrete are shown in Figure 22 and Figure 23,

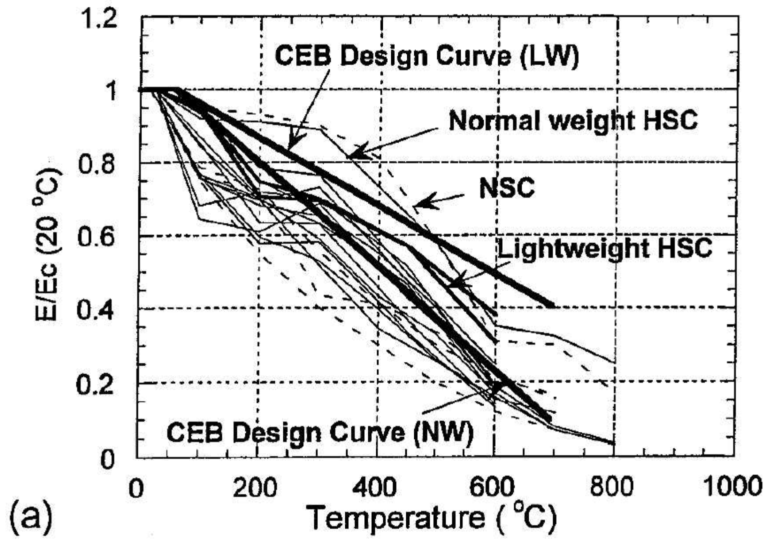


Figure 17: Thermal damage of concrete: loss of stiffness at high temperature
(from the compilation of data by Phan, 1997)

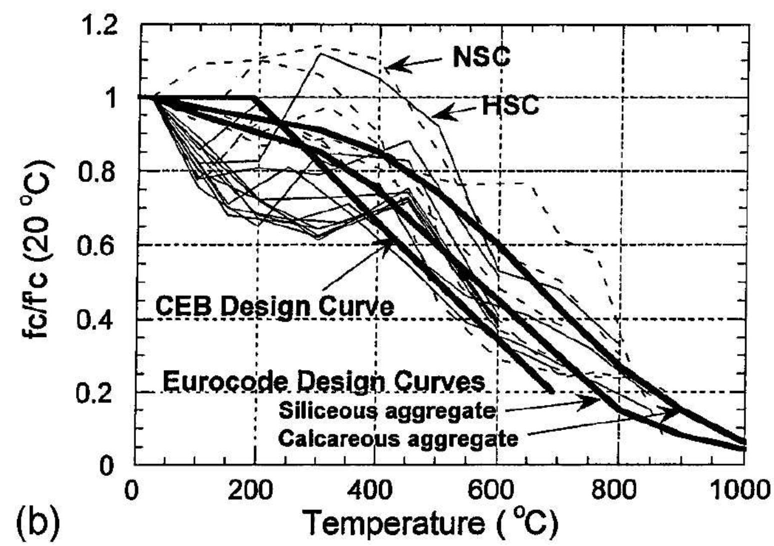


Figure 18: Thermal softening of concrete: loss of strength at high temperature
(from the compilation of data by Phan, 1997)

respectively. Figure 24 shows the combination of these two effects as design curves. A comparison of these four figures shows that there is indeed little difference between concrete and steel on a material level.

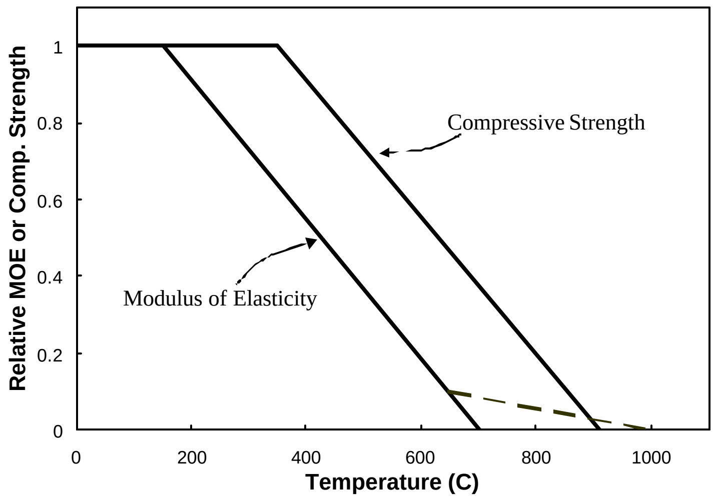
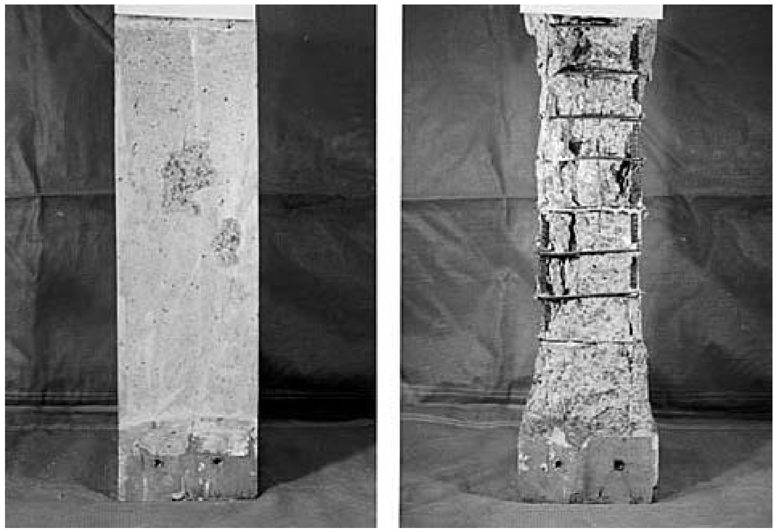


Figure 18: Idealized curves for thermal damage and thermal softening of concrete at high temperature (BSI, 1985)

However, several other mechanisms enter when one considers the behavior on a section member level. First, we should note that concrete material in the context of structures is generally used in conjunction with reinforcing bars. The concrete cover, that is the distance between the fire exposed surface and the steel reinforcement, needs to be designed so to protect the steel reinforcement over sufficient time. Furthermore, the mechanism of failure of concrete members under high temperature is different than that of steel, as it involves spalling of thin layers of concrete from the face of the concrete. The first aspect relates to the heat propagation properties of concrete, the second to stress and pressure build-up in structural members.

Table 1 compares the physical values of heat propagation of steel and concrete. Use of these values in eq. (1) shows that for a given fireproofing (same value of l) and same structural dimension H, the fire protection time t of concrete is at least 5 times the one of steel. Furthermore, the characteristic size H of concrete members is generally much larger than the one of steel. Therefore, a combination of these two effects explains why fireproofing is generally not required for concrete. Indeed, because of its low heat conductivity, concrete in different forms is commonly employed as fireproofing material. For instance, shotcrete (that is a sprayed concrete) has been employed in the WTC for fireproofing the façade columns.

On the other hand, concrete members subjected to high temperatures exhibit a very particular behavior, known as spalling, that is the successive disintegration of surface layers of a concrete member similar to the peeling of an onion. Figure 25 displays spalling of the concrete cover of a reinforced concrete column subjected to heating. Spalling is an interesting phenomenon that is known to be related to the types of constituent materials, thermal stress concentrations, and is dependent of the behavior of the cement paste. The two physical mechanisms that affect the thermal stability of concrete members with regard to spalling are:



a) b)

Figure 19: Effect of heat on concrete after 2 hours of exposure to 1000°C Fire
(a) Fiber reinforced concrete and (b) Ordinary reinforced concrete
(Takenaka Co, 2000)

The compressive stress build-up due to restrained thermal expansion, which is readily understood as a combination of the low heat diffusivity of concrete and its thermal expansion behavior. Like most materials, concrete subjected to heating undergoes a thermal expansion. Because of the low diffusivity, the temperature rise is not uniform over the structural member, but restricted to a surface layer that is scaled in time by $x \propto \sqrt{Dt}$, while the rest of the section remains close to the initial temperature. Since a structural member cannot expand in a non-uniform fashion without disintegrating, the expansion in the surface layer is restrained, which induces high compressive stresses in the surface layer, on the order of the compressive strength of concrete per 100 Kelvin of temperature rise [Ulm et al., 1999a]. The stresses in the surface layer, therefore, reach quickly the compressive strength of concrete, which in turn is subjected to thermal softening (see Figure 23). Concrete, under such compressive stresses, typically fails in planes parallel to the surface.

The vapor pressure build-up due to vaporization of free water or moisture in concrete at high temperature. Concrete is made of cement, water and aggregates, and the material hardens by chemical reactions between cement and water. After hardening, part of the initial water remains in the pores of the material and is subjected, under normal conditions, to a very slow drying process, roughly 300 years for 1m of concrete. Hence, there is always some water left in concrete. At 100°C the liquid water becomes vapor, expanding in the pore space previously occupied by water. While the water–vapor phase change is an endothermic reaction, reducing a small part of the heat during vaporization, the vapor cannot expand within concrete or to the outside. Therefore, the vapor pressure increases, exerting an increasing pressure on the solid part of the concrete. This pressure reduces the confinement of the solid generated by the thermal compressive stresses, and increases the susceptibility of concrete to spalling, particularly in concrete with high moisture content.

A combination of these two phenomena leads to the spalling of the concrete surface layers with a rate of roughly 3 mm/min: The compressive stresses in the surface layer generated by restrained thermal expansion are released by explosive spalling of the surface layer, which disintegrates from the remaining section triggered by the vapor pressure. However, in contrast to steel, concrete sections in general are large, and therefore deterioration in layers from the fire exposed faces of a cross-section does not lead to a rapid catastrophic failure of the entire section. The remaining section remains intact, providing a built-in redundancy for the structural load bearing capacity. This built-in redundancy ensured, for instance, the stability of tunnel liners in recent long-term tunnel fires in several transport tunnels in Europe, such as the 1996 fire in the Channel tunnel, the 35 km tunnel connecting England with France [Ulm et al., 1999b]. Clearly, as far as material and structure is concerned, concrete is less sensitive to fire than steel, and therefore performs well in fires.

But, perhaps what is more important is that concrete, in contrast to steel, comes today in an almost infinite variety of mixes, that can be fine-tuned to generate a new material with a high degree of built-in redundancy. For instance, addition of polypropylene fibers to concrete mix is known to improve material behavior under fire by reducing spalling. This is because the fibers melt under high temperatures, leading to the increased porosity through which water vapor can escape. Figure 25 shows the stunning effect of such fibers on the thermal stability of a reinforced concrete member.

Still, we should note that this built-in redundancy on a material level affecting the structural performance of a member, becomes only efficient as part of a global structural system with built-in redundancies at multiple scales. Indeed, the use of reinforced concrete for the column cores in the WTC would have surely improved the thermal stability of the columns. However, prevention of the failure of the slab system would still require implementation of redundant end joint connections with respect to structural and fire proofing and perhaps, also provision of a reinforced concrete core tube system well integrated with the lateral load transfer mechanism within the building structure. Thus, a materials-to-structural sequence of failure highlights the necessity of redundancy at different scales, from the material level to the structural level, and beyond.

New technology of redundancy

We believe that a built-in redundancy in design and operation of mega-cities and society at large could significantly reduce vulnerability. Redundancy of a system may be defined as a provision of multiple added failure mechanisms that prevent the total system from collapse upon failure of single or several of its components. Therefore, implementation of redundancy in a system will improve its reliability. Redundancy in a system can be defined as that of the active type or the standby type. In the active redundancy all components of the system are simultaneously contributing to the system stability at all times. On the other hand, in the standby redundancy, some of the elements of the redundancy may be generally inactive and become active when some of the active redundancy components fail. Generally, redundant structural systems are examples of the active redundancy type. In the structural context, redundancy may be provided at the material as well as at the system level.

Fiber-reinforced material systems: a multiscale redundant system

A material possesses redundancy if it responds to the same action using more than one mechanism. Below we will illustrate this concept via the fire resistant mechanism of fiber reinforced cementitious composites, a high performance material that has gained increasing popularity in tall building as well as other infrastructure construction.

Recent advances in concrete science and engineering provide the basis for a fine tuned material design of concrete materials, which overcome the traditionally weaknesses of concrete-type materials, that is brittleness and low compressive strength compared to steel (steel typically has a strength that is 10 times higher than that of standard concrete). This brought about a totally new generation of High Performance Cementitious Composites (HP2C), which are based upon the optimization of both the packing density of the cementitious matrix, and the length-diameter spectrum of the reinforcing fibers. In comparison with ordinary concrete, HP2C materials have enhanced microstructural material properties and an enhanced material ductility obtained by incorporating small-sized steel or organic fibers. A typical HP2C mix-composition gives a mean 28 days cylinder compressive strength of 190 MPa, and a ductile tensile strength of 10-15 MPa. The high compressive strength-to-low mass density of this material makes it an ideal material for skyscrapers, in which weight is always a limiting factor. In fact compared to steel, HP2C is 30 to 50% more efficient in terms of strength-to-weight. Furthermore, the ductile tensile strength of HP2C is sufficiently large that one can employ this material without steel reinforcement. As to the fire performance, this is a first advantage of this material in comparison with standard concrete materials. But the real built-in redundancy with regard to fire resistance is that the polypropylene fibers in the material, which contribute in service to the ductile tensile behavior, melt under high temperatures, offering to the vapor an additional connected expansion space to escape. This second function of the fibers, which is only activated in the extreme case of a fire, reduces the susceptibility to spalling of the structural member, thus providing a superior structural performance of the structure under high temperature.

Furthermore, this new generation of high performance materials may well serve, in the future, for the retrofitting of existing structures. The low heat diffusivity combined with the high strength and low weight (compared to steel) of this new class of materials make it an ideal material for structural fireproofing in skyscrapers, which can fulfill more than one function: (1) increase of thermal inertia (like standard fire proofing materials), (2) increased mechanical resistance to blast loading, (3) structural load bearing capacity when the steel member thermally softens. The multi-functionality of this new class of materials can provide, if employed properly for retrofitting of steel members at a material to structural level, a built-in redundancy similar to a second or third airbag built into a car, which would inflate if the first ever failed. This built-in redundancy is a general principle of a sound engineering design, and encompasses materials and structures.

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