



Fire design of concrete structures – structural behaviour and assessment

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State-of-art report prepared by
Task Group 4.3

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Cover image: The Windsor building in Madrid, during the fire in February 2005 (source Calavera et al., 2005; see chapter 7)

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Preface

Concrete is well known to behave efficiently in fire conditions, since it is incombustible, does not emit smoke, and provides good thermal insulation. Furthermore, in reinforced concrete structures, the concrete cover gives a natural protection to the reinforcement, and the size of the sections often delays the heating of the core, thus favouring the fire resistance of the structural members. In addition, concrete structures are often robust and thereby able to accommodate local damage without major consequences to the overall structural integrity.

However, past experience with real fires shows that a thorough understanding of concrete behaviour and structural mechanics is necessary to improve the design of R/C structures with respect to fire.

Improving the understanding of concrete and concrete structures under fire is the objective of this state-of-the-art report, that is the outcome of the works of Task Group 4.3 "Fire Design of Concrete Structures", serving under *fib* Commission 4 "Design of Concrete Structures". The results of the most recent research activities on the **structural performance** of concrete subjected to fire are reported. Special attention is paid to the indirect actions caused by the restrained thermal deformations and several basic examples show the influence that a local fire has on the global structural behaviour.

Not only the design of new structures, but also the analysis and repair of existing fire-damaged structures are addressed in this bulletin, that is the second one issued by Task Group 4.3, since a companion bulletin (*fib* Bulletin 38, "Fire design of concrete structures – materials, structures and modelling") has been recently published, mostly on materials behaviour.

Working Party 4.3-2, headed by Prof. Luc Taerwe, performed both the writing and the editing. The members of WP 4.3-2 are well-known researchers and experts in the domains of (a) materials and structural behaviour at high temperature, and (b) fire design of concrete structures. All members (see previous page) contributed actively to the outline and contents of the various chapters, however each chapter was finalized by single members or small groups, as indicated at the beginning of the chapter.

Prior to publication, the draft contributions were presented and discussed in several meetings and in three workshops organized by the Task Group in Malta, 2001; Milan, Italy, 2004 and Coimbra, Portugal, 2007. The workshops favoured interaction with international experts outside the Task Group, to the benefit of the final version of this bulletin.

It is our hope that this state-of-the-art report on concrete structures in fire will improve the understanding of their behaviour in such extreme conditions, to the advantage of practicing engineers looking for better and safer design standards.

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Convener of Working Party 4.3.2

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Convener of Task Group 4.3

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1 Introduction

In the world of construction, fire is definitely a danger that has to be prevented and fought by all possible means. Although the probability is low, fire may occur anywhere, in any season, and in any phase in the lifetime of a building – construction, service or refurbishment – like in the Windsor Tower in Madrid (fire on February 2005). Fires may be triggered in different ways, including terrorism and war, as demonstrated by the fires in the Twin Towers and in the Pentagon, after the infamous attacks of September 11. However, trivial breakdowns like an electrical short circuit in a coffee machine may also be critical, as demonstrated by the fire in the Delft Architectural Engineering School building in May 2008.

Even when limiting our attention to the effects that fires have on structures, numerous topics are still open to investigation:

- Materials: thermal properties and thermal diffusivity as a function of the temperature (first heating, cooling and reheating); aggregate and cement types; influence of fiber addition (metallic, inorganic and/or polymeric fibers); toughness and fracture parameters at high temperature; modeling of mass transport of water and water vapor.
- Sectional analysis: M-N envelopes and failure modes of reinforced concrete sections made of different cementitious composites (NSC, LWC, HPC, HPLWC, FRC, HPFRC, SCC, HPSCC); validity of the reduced-section approach – based on a reference temperature – under an eccentric axial force, at high temperature and after cooling.
- Structural analysis: transient creep and its role in structural behavior; failure modes during and after a fire; effects of the restrained thermal expansion; cover spalling (local, extended, in high-performance concrete with/without silica fume); column stability at high temperature.
- Assessment after fire: non-destructive methods based on the residual concrete color and on the resistance to drilling, in order to evaluate the maximum temperature locally reached by the concrete; shear sensitivity and bond sensitivity in damaged R/C and P/C structures; residual strength of ordinary reinforcement (outwardly-tempered, stainless-steel, low-/high-carbon rebars) and prestressing tendons (high-strength wires and strands).
- Real fires, large-scale tests and model validation: failure modes ensuing from materials decay, restrained thermal expansion, thermal expansion of nearby members and loss of bond in P/C members; thermal field in hollow-core slabs; actual temperature of the reinforcement.
- Connections: ultimate capacity of the different types of fasteners at high temperature and after cooling (failure modes under axial and shear forces; design models).
- Codes: should they be more detailed, more general, more materials-oriented, more member-oriented, more structure-oriented?

With reference to cementitious materials, their behavior in direct tension at high temperature is still a challenge, and the test results available in the literature are scanty indeed. Further results are badly needed, since they are instrumental in evaluating such fracture parameters as materials specific fracture energy, toughness and characteristic length, not to speak of the whole stress-crack opening curve. These parameters have been extensively investigated after cooling, with reference to the maximum temperature reached in the

material, but there is mixed evidence in terms of loss of toughness, increased ultimate strains and greater damage diffusion. For sure, the material becomes more strain-tolerant at high temperature and after cooling.

With reference to sectional analysis, the reduced-section (or effective-section) approach is known to work well in pure bending, but its validity in the case of combined bending and axial loading is not completely proved, even if some results show that this approach is conservative. However, we know that being too conservative is not the key to sound design.

With reference to the structural behavior, the redundancy in continuous beams and frames has still some aspects open to investigation because of the thermally induced axial forces in the beams, causing shear forces in frame columns. Also the failure mode of the various members is generally affected by high temperature, during and/or after a fire, often with less bending sensitivity and more shear sensitivity, as may occur in point-supported continuous slabs, with light punching reinforcement.

With reference to repair and assessment, there are several approaches aimed to assess the properties of the damaged concrete and to evaluate the maximum temperature reached locally, but user-friendly methods are still to be developed. However, remarkable headway has been recently made in such diversified fields as concrete drilling resistance and concrete colorimetry.

Another very specific subject that has captured the interest of an increasing number of researchers – both in the industry and in the academy – is the behavior of the fastening devices in extreme environmental conditions because of fire and/or corrosion. Some results at high temperature are available, but further studies are needed in order to formulate user-friendly design methods. As a matter of fact, there is a strong demand in this field, since heavy-duty fasteners are increasingly used in very severe conditions, such as those occurring in tunnels, where fasteners support various primary systems, that should work even in fire conditions (for ventilation, electric-power supply, fire extinguishment and traffic control).

A few words on concrete thermal behaviour: since sectional and member behaviour is strictly related to temperature evolution in space and time, the thermal properties of the materials should be introduced into the codes as exhaustively as possible. Just to quote an example, does SCC exhibit the same thermal diffusivity as vibrated concrete? The answer seems to be yes, but further studies are needed.

In this context, this bulletin tries to strike a balance between structural and materials issues. As a matter of fact, the many new and highly innovative cementitious composites now entering the scene put a lot of pressure on the research activities concerning the materials side, but we must remember that any newly developed material should be checked against its structural advantages!

2 Fire action and design approach*

2.1 Fire action

Several nominal fire curves are proposed in the codes to be used in the design process for representing the action of the fire. The most often used are the ISO 834 fire curve, the ASTM E119 curve, the hydrocarbon fire and the external fire curve. All have similar characteristics:

- They are formed of a simple relationship giving one temperature, supposed to be the temperature of the gases in the compartment, as a function of time. They are thus representing a fully developed fire. For a large compartment, such a situation is not encountered before a significant amount of time has elapsed since the very beginning of the fire. This initial period of time is thus not taken into account in the calculated fire resistance whereas, as far as safety of people is concerned, this is the most important period, in fact the only one during which evacuation from the fire compartment is possible.
- All these relationships are monotonously increasing functions of time. The cooling down phase of the fire is not taken into account. In fact, when a certain fire resistance time is required, it is sufficient to check the load bearing capacity for this duration of fire. No consideration is given to the period beyond this duration.

These relationships hardly depend on the particular characteristics of the situation for which the design is performed. The quantity of fuel, the dimensions of the compartment, the conditions of ventilation, for example, are not taken into account. In fact, a limited choice exists in order to take the situation into account: the hydrocarbon curve, for example, is chosen when the characteristics of the fuel are supposed to be pertinent to this name. In usual building constructions, the standard fire is nearly systematically chosen, be it either the ISO or the ASTM curve, and the characteristics of the situation are all lumped in the required fire resistance such as, for example, the consequences of the fire (underground car parks), the amount of floors (low, medium or high rise buildings), the size of the compartment (smaller or bigger than a threshold area), or the occupancy (hospital, school, theatre, office building, dwelling).

2.2 Consequences of a fire on a concrete structure

2.2.1 Heating of the structure

The most direct effect of a fire on a structure is that the temperature in the structure will increase, in a first phase, then decrease progressively as the fire decreases until extinction. During the heating phase, heat is introduced in the structure by a combination of:

- convection from the surrounding gas,
- radiation
 - from the surrounding gas if it is opaque,
 - from the fire source,
 - from the compartment walls and other heated objects.

* by Jean-Marc Franssen, Jean-Claude Dotreppe, Kees Both and Joris Fellingner

During the cooling phase, heat is evacuated from the structure by a combination of:

- convection to the surrounding gas,
- radiation to the compartment walls and other heated objects, including the ashes that remain from the combustible material.

For separating structural members such as walls and slabs, heat is also lost from the unexposed side of the member to surrounding that is at ambient temperature. This happens mainly by convection because radiation is lower at the lower temperatures that are normally observed on the unexposed sides.

Temperature in a concrete structure heated by a fire is by far not uniform. At any moment in the fire, there exist significant differences of temperatures between different locations. For linear elements such as beams and columns, the gradients along the axis of the elements are usually limited and the most significant gradients are observed in the cross section. For flat elements such as walls and slabs, the gradient is most important across the thickness of the elements. Gradients along the linear elements or in the plane of flat elements can nevertheless be observed in the case of highly localised fires such as, for example, one car burning in a car park.

It has very often been pretended that temperature is not uniform in concrete elements whereas it is more or less uniform in steel elements because of the significant difference in thermal conductivity: approximately 45 W/mK for steel and 2 W/mK for concrete. In fact, this is not entirely correct. The shape of the sections and, in fact, the thermal massivity is the main reason. Indeed, numerical simulations show that if a section that has the same dimensions as a hot rolled steel section could be built in concrete, it would have a uniform temperature distribution. The temperature at any time would in fact depend on the thickness of the plates forming the profile. On the other hand, a block of steel with the same dimensions as those usually encountered in concrete sections would have a fairly non uniform temperature distribution.

The level of temperature differences observed in a section depends on several factors, the most important ones being:

- The increase rate of the fire. The faster the elevation of temperature, the higher the temperature differences. The hydrocarbon fire will, for example, generate higher temperature differences than the ISO curve, this one creating higher differences than some slower parametric fires.
- The severity of the fire, in terms of duration and maximum temperatures. A short fire will obviously not allow sufficient energy to be introduced in the section for high temperature differences to develop. The same holds for a fire where the developed temperatures remain limited.
- The shape of the section. In fact, the thermal massivity considered, as is commonly done for steel sections, as the ratio between the exposed surface and the volume to be heated, is a good indication of the level of possible temperature differences. A thin column will have lower temperature differences than a massive one. Members with more complex shapes such as, for example, a TT section, will have a more complex distribution of the temperatures. In this case, the local massivity of the web and of the slab determines the local differences.
- The moment in the fire. With fires starting quite rapidly, the gradients are particularly severe during the first moments of the fire. As the fire continues and the gas temperature tends to level off to a constant level, the temperature differences in the structure tend to decrease. When the fire enters in the cooling phase, the temperature gradients in the section change in direction, first in the zones near the surface, then later also in the centre.

- The thermal properties of the concrete. Calcareous concrete have lower conductivity than siliceous concrete and therefore generate higher temperature differences. Lightweight aggregates have an even lower conductivity.

Figure 2-1, for example, shows the temperature distribution across the thickness of a 15 cm thick wall at various times in the fire. The right hand side of the wall is exposed to air at ambient temperature while the left hand side is exposed to a fire with a heating and a cooling phase: the gas temperature follows the ISO curve during 120 minutes, then decreases linearly to 20°C from 120 to 180 minutes and keeps the value of 20°C thereafter. It is usually accepted that the influence of reinforcing bars on the temperature distribution in concrete elements is rather negligible and the bars have not been taken into account in this analysis. The thermal properties of concrete are those of siliceous concrete as defined in Eurocode 2 (EN1992-1-2, 2004).

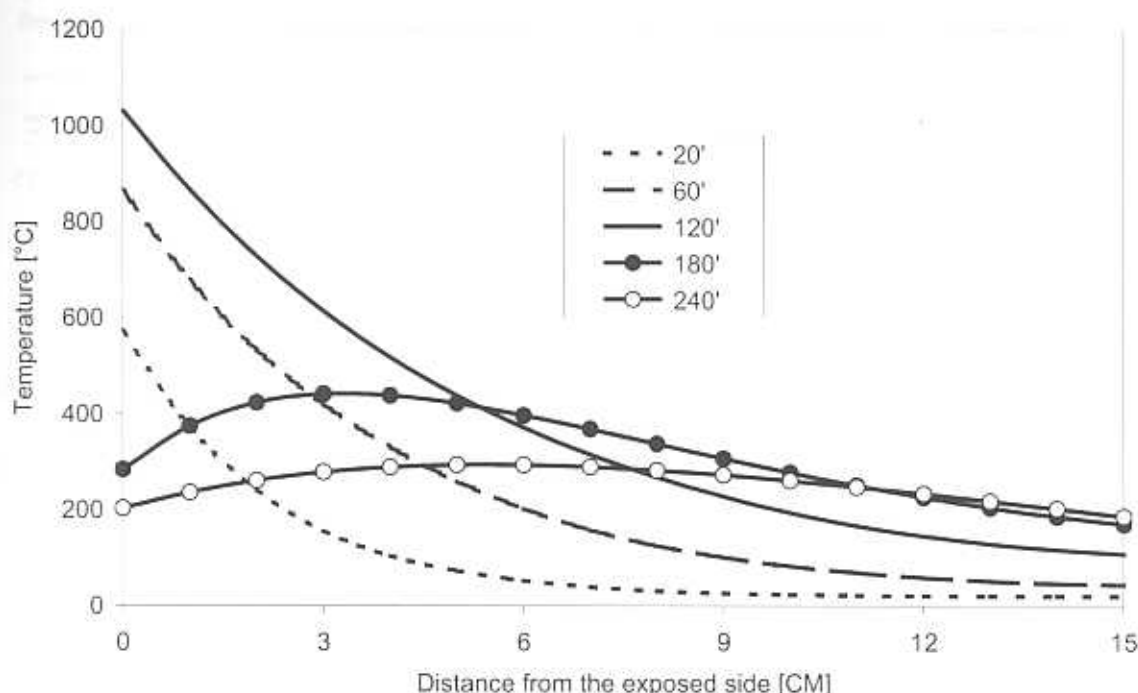


Fig. 2-1: Temperature distribution in a 15 cm wall

Figure 2-2 shows the temperature distribution across the web of a 30 cm wide beam. The beam is supposed to be deep enough for the heat flux to be one-dimensional at a sufficient distance from the lower side of the section. This could be, for example, on the horizontal line joining nodes 24 and 255 on Figure 2-2. Only half of the section is presented here owing to symmetry reasons.

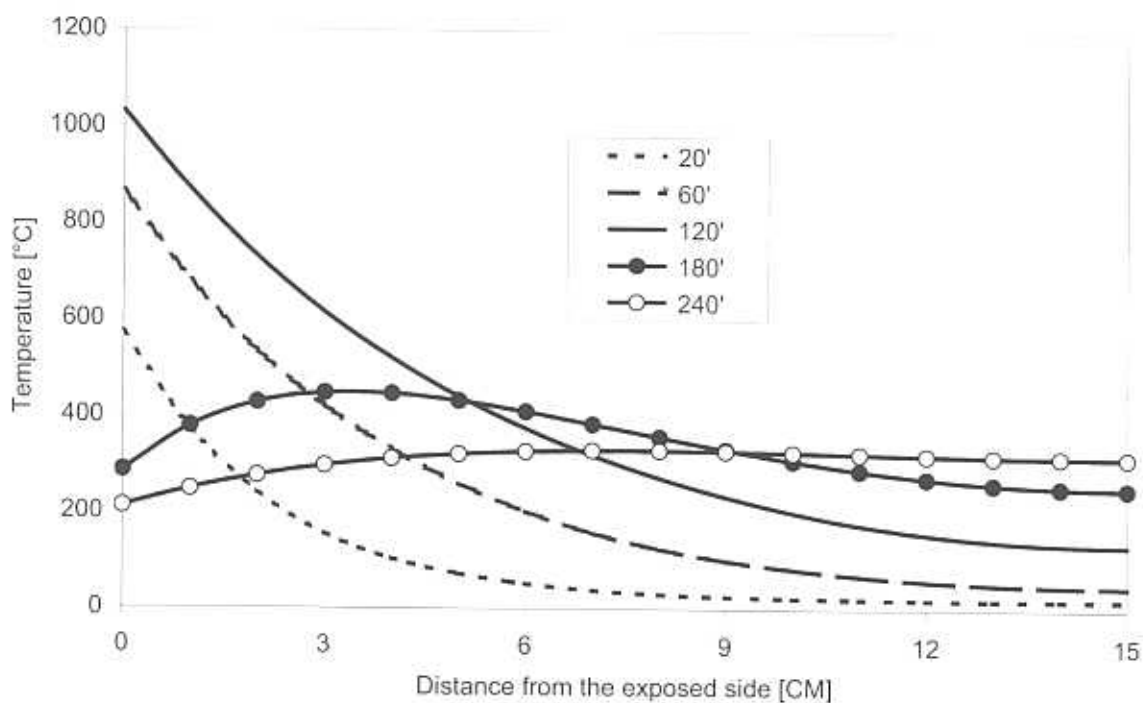


Fig. 2-2: Temperature distribution in a 30 cm wide beam

Figure 2-3 shows the isotherms in a 30 x 30 cm² section heated on four sides by the ISO fire. Only ¼ of the section is shown owing to symmetry reasons.

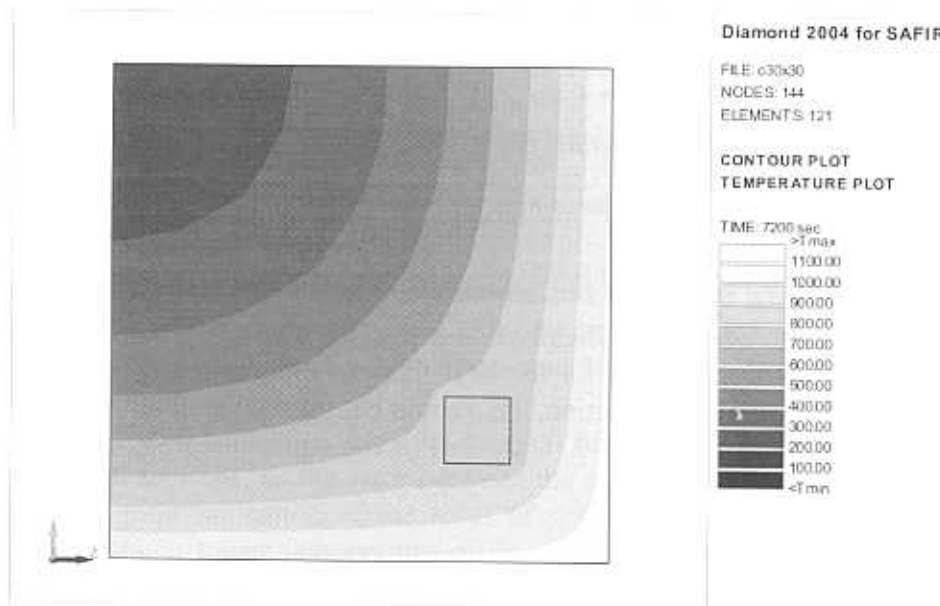


Fig. 2-3: Isotherms in a 30 x 30 cm² column subjected to the ISO fire

Figure 2-4 shows the isotherms in a 24 x 48 cm² beam section after 2 hours of ASTM E119 fire acting on 3 sides of the section (the upper side has been considered as adiabatic).

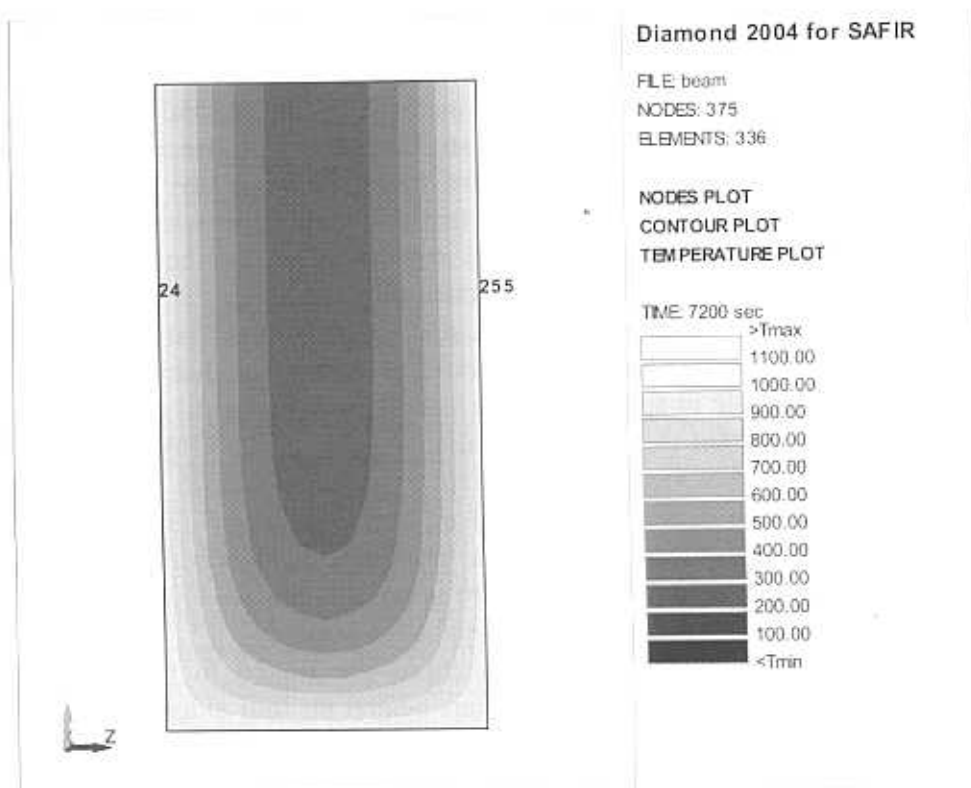


Fig. 2-4: Isotherms in a beam subjected to the ASTM E119 fire

Figure 2-4 shows the isotherms in a T section subjected to the hydrocarbon fire during 2 hours. The upper side of the slab is exposed to air at the ambient temperature.

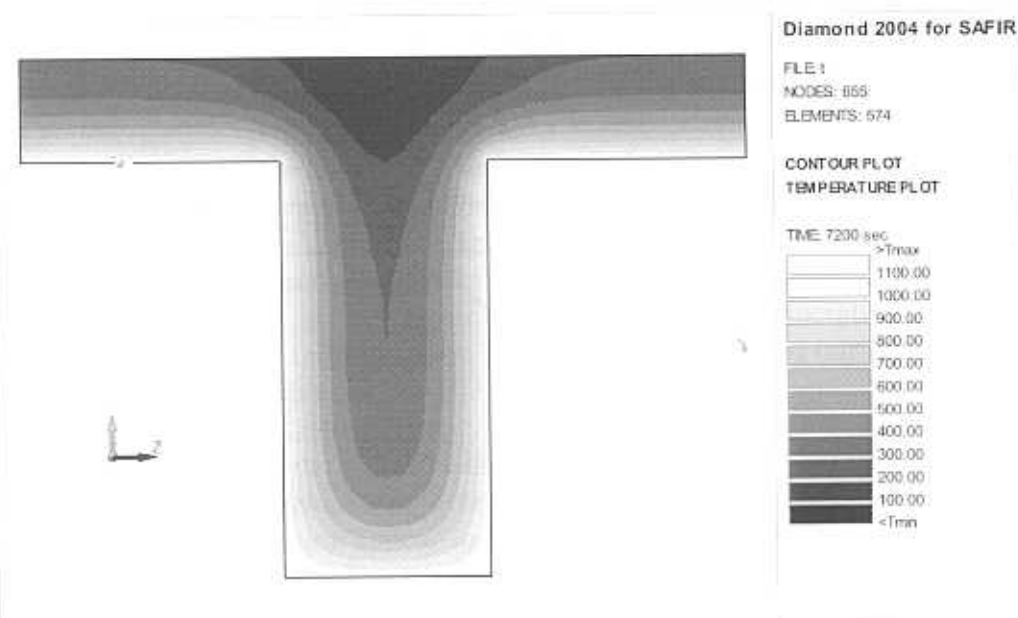


Fig. 2-5: Isotherms in a T section subjected to the hydrocarbon fire

The direct consequence of the heating of the concrete is a modification of the material characteristics like a decrease of the strength and of the stiffness of the material, see section 2.2.2, as well as the generation of additional deformations linked to the stress level during

first heating that are usually called Load Induced Thermal Strain (LITS) or transient creep in the literature, more details can be found in *fib Bulletin 38* (2007).

Another phenomenon linked to temperature increase is the thermal elongation, i.e. the elongation of the material that occurs even if the stress level is zero. Because the temperature distribution in the section is not uniform, these thermal strains are also not uniformly distributed on the section. These strains and the fact that they are non uniform have several effects on the behaviour of the concrete elements and the effect may be different depending on the section type. The most important effects have to do with:

- **Spalling.** Severe thermal gradients near the surface during the first moments of the fire generate important compression stresses parallel to the surface that are highly suspected for playing a crucial role in spalling. These stresses are present in all section types and are all the more important that thermal gradients are important (hydrocarbon fires, for example). See also a discussion of the phenomena of spalling in *fib Bulletin 38* (2007).
- **Elongation.** Elongation of a member is linked to the "average" thermal elongation on the section. Those section that are exposed on many of their surfaces, like columns, beams or T sections, will thus exhibit a more important thermal elongation than section exposed only on one side such as walls and slabs for example.
- **Lateral deflections.** Thermal lateral deflections are linked to the average thermal gradient in one direction or another. Columns heated on four sides, for example, have no thermal bowing. Beams heated on three sides show a downward thermal bowing that is all the more reduced that the beam is deep. This is because, in a very deep beam, most of the section (in fact, all except the lower side) has a uniaxial temperature field that generates no average gradient. Flat slabs and walls heated on one side are the elements that exhibit the highest level of thermal deflections. A T section is an heterogeneous section because part of it, the slab, would have important thermal curvature if it were alone whereas the web has no significant thermal bowing because it is more like a beam exposed on three sides. For example, in an experimental test performed on a section similar to the one shown on Figure 2-6, severe horizontal cracks were observed near the support at the junction between the slab and the web, see Figure 2-7, with the slab being separated from the web and being curved upward.

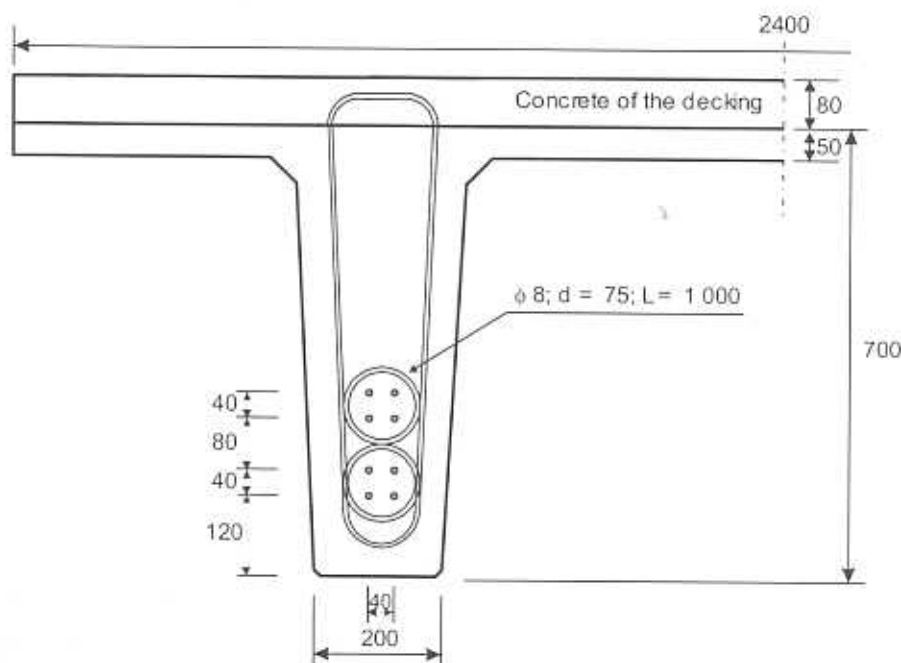


Fig. 2-6: The half of a typical TT section

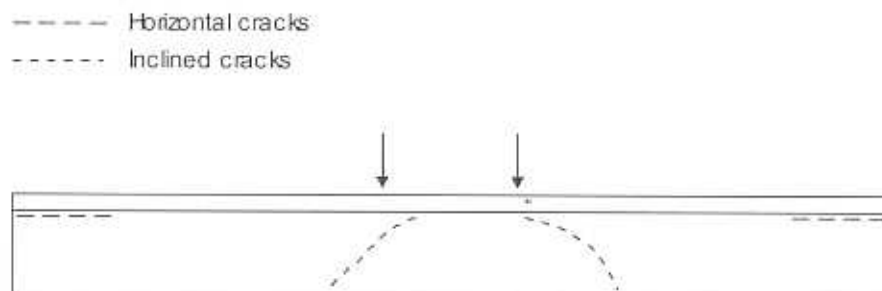


Fig. 2-7: Crack pattern observed during the test

- Stresses (this effect has already been mentioned as having an influence on spalling). If the thermal elongation or the thermal bowing of an element is restrained by the surrounding structure, this will generate indirect effects of actions, see sections 2.2.3 and 2.2.4, either compression axial force or variation in the bending moment diagram, and these will modify the stress level in the section. Even if the element is externally totally free to expand and deflect, the fact that the whole "surface" of thermal strains in the section cannot be represented by only the average elongation and the average gradient leads to internal thermal stresses. Tension stresses can develop in the centre of the section during the first moments of the fire, even if this section belongs to a column loaded by a compression force. Similar effects can occur in the central part of a slab. These thermal stresses will be treated more in detail in section 2.2.4.
- Second order effects. Thermal deflections, if taking place in elements that are subjected to a compression force, create important second order effects that, in certain cases, can be sufficient to lead to premature failure. This can be the case, for example, in concrete walls exposed on one side, especially for cantilever walls.

2.2.2 Modification of the material characteristics

The strength as well as the stiffness of steel and concrete are reduced by a temperature increase. The evolution of the strength and stiffness characteristics with temperature is yet not sufficient to describe the modification of the material characteristics because in fact the whole stress-strain relationships are modified. New characteristics may thus appear at elevated temperatures that are necessary to characterise the behaviour.

The modifications of the material characteristics have been studied by various authors. A lot of information can be found in a comprehensive reports compiled in the 80's on behalf of RILEM (Schneider, 1985). Although only normal strength concrete was covered in (Schneider, 1985) and many more research works have been carried out since, these reports still contain valuable information.

Compressive strength is the most extensively analysed property of concrete. The strength at room temperature, the water/cement ratio, the type of cement, the maximum size of aggregate and the rate of heating appear to have little influence on the relative reduction, in percent of the original strength. The type of aggregate has an influence, the decrease being less important with calcareous or lightweight aggregates compared to siliceous aggregates. The aggregate/cement ratio has also an effect, with the reduction being proportionally smaller for lean mixes than for rich mixes. Finally the reduction highly depends on the testing procedure, with more favourable results obtained when a certain stress level is maintained during heating.

The **modulus of elasticity** is influenced in the same way by the factors mentioned previously for the compressive strength. The reduction as a function of the temperature is bigger than for the compressive strength because the peak stress strain increases with the temperature.

Steady state creep is of importance essentially for service conditions, i.e. temperatures below 150°C applied for very long durations, in concrete reactor vessels for example. In a fire situation, the creep rates observed under steady state conditions are very much less important than the creep values observed under transient temperature conditions.

Load Induced Thermal Strain is the particular deformation that occurs in concrete during first heating under load. It is also influenced mainly by the aggregate type, by the aggregate/cement ratio and by the curing conditions; air cured and oven dried specimen exhibit a significantly lower transient creep than water cured specimens. see *fib* Bulletin 38 (2007).

The **tensile strength** of concrete has a tendency to decrease faster with the temperature than the compressive strength.

Although the experimental evidence is more scarce, some reports suggest that the **fracture energy** of concrete is not reduced at elevated temperature. It can even be slightly increased.

The **stress-strain diagram** of concrete reflects the modifications of the compressive strength, of the modulus of elasticity and of the peak stress strain. Figure 2.8 for example shows the stress-strain diagram at 5 different temperatures as proposed in Eurocode 2 for concrete in compression. According to the Eurocode, these relationships have been artificially softened in order to incorporate implicitly the effect of transient creep. The descending branch of the curve has to be introduced for numerical reasons in non linear modelling but too much credit must not be given to the real precision of the relationship in this part of the diagram.

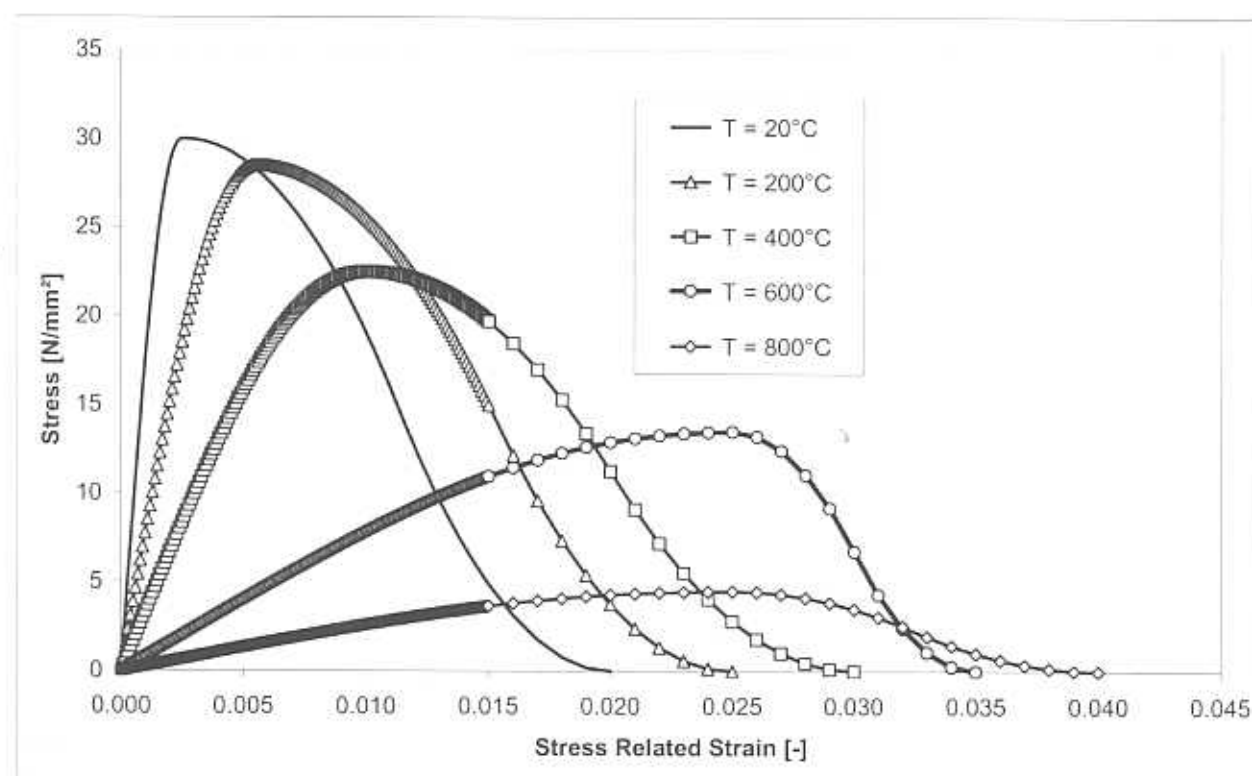


Fig. 2-8: Stress strain relationship in concrete

The thermal properties of concrete are also modified by a temperature increase. The thermal conductivity is normally reduced whereas the specific heat is increased by an elevation of the temperature. As a result, the thermal diffusivity decreases with increasing temperatures.

Yield strength of steel is reduced by an elevation of the temperature. The relative reduction does not depend on the value of the yield strength at room temperature but it varies with the type of steel, hot rolled or cold worked reinforcing steel, quenched and tempered or cold worked prestressing steel.

The **Young's modulus of steel** is also reduced at elevated temperatures, somewhat faster than the yield strength.

In fact, the whole **stress-strain diagram** is modified. Figure 2-9 shows the stress-strain diagram for reinforcing steel at 5 temperatures as proposed by Eurocode 2. A non linear behaviour appears clearly for low strains at elevated temperatures. It is usually modelled as an elliptic curve. This leads to the definition of the limit of proportionality, the stress beyond which the behaviour ceases to be linear. The linear descending branch proposed beyond a strain of 15% is also there for numerical reasons; with an infinitely long plateau, some numerical software are so robust that they could predict stability of structures in extremely distorted positions corresponding to unrealistically high strain level totally incompatible with the deflections that are acceptable. For example, the ductility in plastic hinge could be incredibly high if an infinite plateau is used. Of course, the precision of the curve in this descending branch is rather loose but this is not a main issue; what is important is that the software detect that the structure is very near to collapse as soon as some integration points enter in the domain of these high strains.

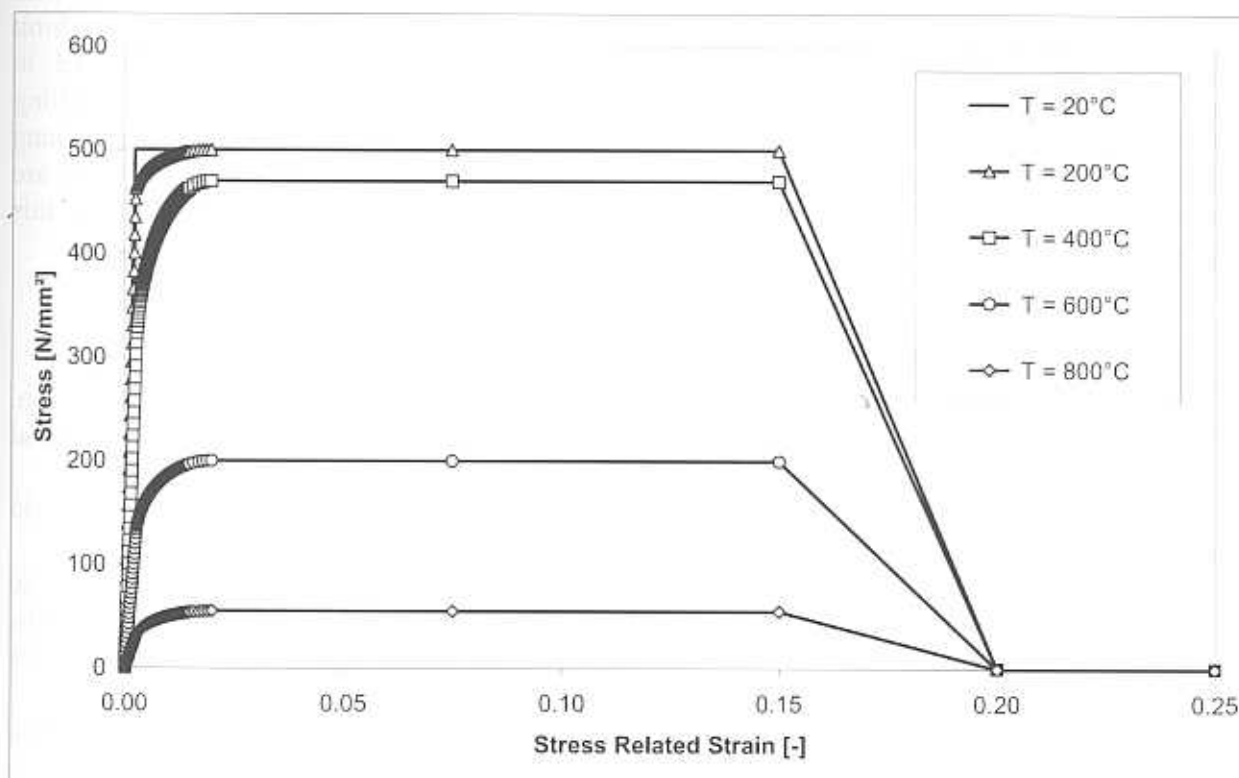


Fig. 2-9: Stress strain relationship in reinforcing steel

The thermal properties of steel are also modified by a temperature elevation, with a decrease of the thermal conductivity and a slight increase of the specific heat. This is not particularly relevant in concrete structures because the amount of steel is generally so low that it hardly influences the temperature distribution. One exception may be the peak in the specific heat curve that is observed around 735°C and that may delay slightly the temperature increase of steel bars at this temperature (provided that the stability is still ensured for so high temperatures).

The geometrical size of concrete and steel is modified by a temperature increase. This is the well known **thermal expansion**. The expansion is not linearly increasing as a function of the temperature. The order of magnitude of the thermal expansion can reach 1% at very high temperatures in the range of 800°C. This phenomenon plays an important role in the behaviour of structures because it induces either large displacements that may generate geometrical second order effects or indirect effects of actions if the expansion is restrained. Some effects of thermal expansion have already been mentioned in section 2.2.1. Indirect actions are treated in section 2.2.3 and, with more details, in section 2.2.4.

Bond strength between concrete and steel has been shown to reduce with temperature, at a rate more similar to the reduction of the tensile strength of concrete than that of the compressive strength. Experience has yet very rarely produced evidence of failures by debonding in reinforced structures. The problem is more critical for prestressed elements.

Spalling in concrete structures is a very important characteristic linked to high temperatures. Different types of behaviour are usually named as spalling, from the very progressive sloughing off at the surface that progressively exposes the inner part of the section and the reinforcing bars to elevated temperatures, to the explosive spalling that suddenly destroys completely the material. Extensive research activity is still going on in order to understand and mitigate this phenomena, the problem becoming more crucial with the apparition of high strength concretes because these tend to be more prone to exhibit spalling than ordinary concrete. Further details of the phenomenon are discussed in *fib* Bulletin 38 (2007). The factors most often mentioned as playing a negative role in spalling are: fast temperature increase, high moisture content, high compression stress level, young age, low porosity, thin members, geometrical effects (corner spalling). Some of these are related to the material itself but it seems that structural effects also play a role in this phenomenon.

2.2.3 Main effects of indirect actions

Indirect actions are those effects of actions that arise from restrained thermal expansion. For example, a beam that cannot freely expand longitudinally will see an increase in the axial force especially during the first minutes of the fire.

In fact, even in a single member that is completely free to expand, indirect effects do appear at the local level. This is developed more in detail in 2.2.4.

If the member is not restrained externally, thermal expansion may anyway generate large displacements that, by geometrical second order effects, also modify the effects of actions. This effect is not usually classified as an indirect action, but it may play a crucial role, in free cantilevered walls exposed on one side for example.

Thermal gradients in concrete slabs generate the large deflections that are required for the membrane tension effect to develop. In this case, thermal expansion has a positive effect on the stability.

Restraint to axial expansion induces axial compression forces in beams which, depending on the position of the restraint, can have a positive effect (restraint at low level in the section) or a negative effect (restraint in the upper level in the section).

Restraint to thermal bowing dramatically modifies the bending moment diagram in continuous beams or slabs with a clear tendency to have more negative bending than under ambient conditions.

Thermal expansion has a great influence on the structural behaviour of both the fire exposed structure and the unexposed structure connected to it.

The calculation of the thermal stress distribution over the cross section is based on three principles i.e. kinematic requirements, constitutive laws and equilibrium, see Figure 2-10 (Fellinger, 1999).



For structural members with a high span to depth ratio, this means that Bernoulli's hypothesis must be fulfilled, implying that plane cross sections should remain plane. Thus, the distribution of the total strains over the cross section can be described by the curvature and the axial strain at one point in the cross section.

Secondly, with respect to the constitutive laws, it is assumed that the total strain at any position in a cross section can be split into thermal strains and mechanical strains.

The thermal strains solely depend on the temperature rise and the coefficient of thermal expansion, which is a material property. They do not depend on the stress level. Therefore, once the temperature profile over the cross section is known, the thermal strain profile can be calculated directly.

From these two requirements it follows that mechanical strains develop whenever the thermal strains do not comply with the kinematic requirements for the total strain field.

Mechanical strains cause stresses. In the calculation of these stresses, the reduction of strength and stiffness at elevated temperatures should be taken into account as well as increasing ductility. Calculation of thermal stresses in fire exposed structures without taking into account the non elastic strains leads to completely unrealistic results.

Eurocode 2 provides stress strain relationships for concrete at elevated temperatures that include increasing plasticity and reduced strength and stiffness. Also transient creep is implicitly taken into account. However, it is questionable whether the accuracy of the implicit formulation of transient creep is adequate to calculate the forces in restrained concrete structures.

Finally, the stresses must satisfy the equilibrium requirements. In a simply supported beam, the generalised forces in each cross section, i.e. the normal force N , the shear force V and the bending moment M are exclusively determined by equilibrium. So, in this case, these forces do not depend on any constitutive model nor on any kinematic relation. Thermal stresses and strains therefore do not affect the distribution of the generalised forces in simply supported structures.

In principle, the stresses should be calculated taking the imposed loading into account. So, the stress distribution over the cross section must balance the applied normal force N and bending moment M .

Thermal stresses can not simply be superimposed by stresses caused by imposed loading due to the highly non-linear constitutive behaviour of both concrete and reinforcement. Nevertheless, for sake of simplicity however, the thermal stresses are normally calculated ignoring the imposed loads. Vice versa, when evaluating the plastic bending capacity of a cross section, the thermal stresses are ignored since the critical cross section is generally cracked and the reinforcement yields.

As indicated in Figure 2-10, compressive thermal stresses develop at the bottom and the top of the cross section and tensile thermal stresses in the web. The tensile stresses in the web can easily lead to vertical cracks, especially when the web has a reduced thickness.

This phenomenon, illustrated in Figure 2-11, has been proved by Fellingner (1999). In this reference it is explained why vertical cracks develop at regular distances. It is also shown how thermal stresses build up over a certain development length at the end of a simply supported beam.

In hyperstatic structures the distribution of internal forces in each cross section will change under the effect of thermal stresses.

For a beam clamped at both ends, the boundary conditions of no rotations at the supports imply that the curvature in each cross section initially remains zero. To counteract the thermal strains, mechanical strains develop that cause an additional hogging moment in each cross section, i.e. the line of bending moment shifts upwards. In a continuously supported beam, the same type of phenomenon occurs (see Figure 2-12; Fellingner, 1999). If horizontal translation is allowed, no normal force can build up in the beam and the thermal stresses should satisfy equilibrium in the horizontal direction.



Fig. 2-11: Vertical cracks in simply supported fire test specimen sawn out of a hc slab

At the supports, the plastic hogging moment capacity will be reached and plastic hinges will develop, i.e. mechanical strains localise in a flexural crack that will propagate through the cross section starting from the top and yielding of the top reinforcement occurs. These plastic hinges at the supports form in an early stage after the begin of the fire. In a slab where the thermal gradient is very important, this can occur within 15-30 minutes. In a beam where the thermal gradient is less important, it will take more time.

After the plastic hinges are formed, the bending moment distribution is determined by the plastic hogging moment capacity at the supports. The thermal stress distribution on the cross section outside the zone of the plastic hinge will shift towards the one corresponding to the simply supported beam.

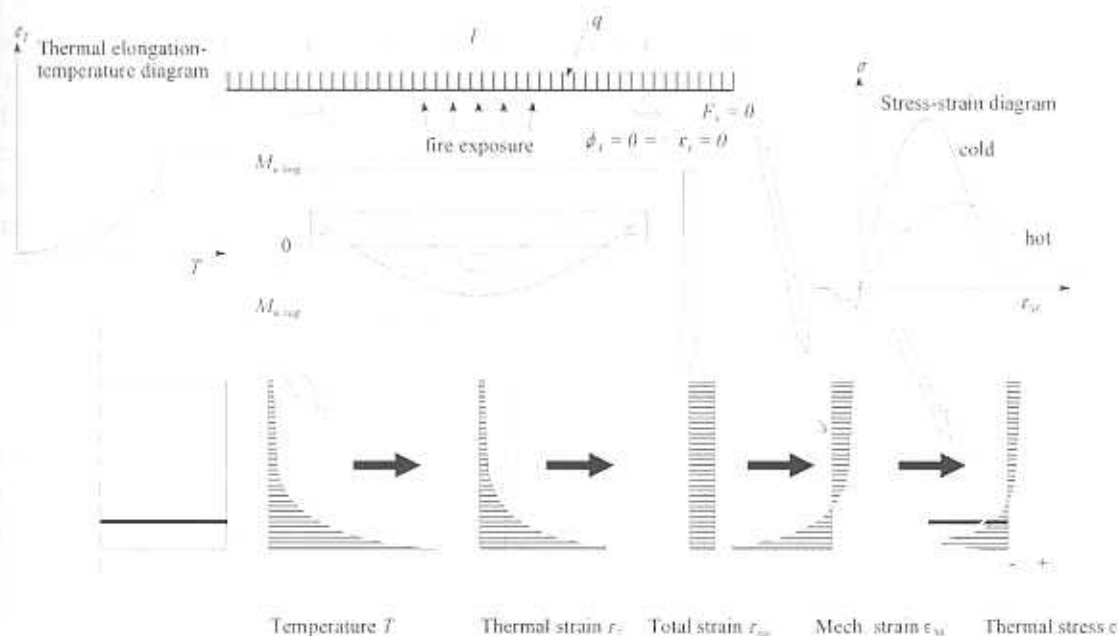


Fig. 2-12: Graphical representation of the calculation of thermal stresses in a continuously supported beam

The designer of continuously supported slabs and beams should take into account the shift in the bending moment distribution under fire exposure and the consequences on the length over which the top reinforcement should be extended from the support. Moreover, in order to be able to reach the full plastic moment capacity at mid span, special attention should be paid to the ductility of the top reinforcement at the supports.

In a beam with hinged supports that prohibit horizontal translation, a horizontal normal force will develop in the beam. The eccentricity of this force depends fully on the vertical position of the centre of the hinge at the supports. Obviously, no bending moment can develop in the hinged supports. However, due to the presence of the normal force, this is only true with respect to the axis of the horizontal force. As a result, a bending moment will develop, relative to the neutral axis of the beam. This bending moment is constant over the length of the beam.

If the centre of the hinge is positioned below the neutral axis of the beam, a hogging bending moment is introduced by the normal force, which counteracts the imposed loading. Vice versa, if the centre of the hinge is positioned above the neutral axis of the beam, an additional sagging bending moment is introduced on top of the imposed loads.

Therefore, the vertical position of the axial force has a great effect on the structural behaviour, as has been shown by tests performed in Germany (Haksever and Walger, 1980).

However, the bending moment resulting from the eccentricity of the restraining force is hard to calculate. First of all, the position of the neutral axis in the cross section varies with time. Furthermore, in some practical support details, the position of the centre of the hinge is undefined and can shift during the fire as well.

Therefore, when calculating the thermal stresses in a cross section of a restrained structure and the restraining forces, special consideration should be given to the modelling of the support.

In summary, thermal stresses are the result of mechanical strains that have to develop in order to counteract incompatible thermal strains. Thermal stresses occur in both restrained structures as well as unrestrained structures in which the temperature is non-linearly distributed. In concrete structures, the thermal stresses can only be calculated accurately by taking into account the non-linear constitutive behaviour including cracking, transient creep and plasticity. When evaluating the load bearing capacity of concrete structures, thermal stresses should be considered whenever the capacity can not be determined by a pure plastic analysis.

2.3 Design approach

2.3.1 Ultimate limit state

The design philosophy of the Eurocodes is based on the concept of limit states, i.e. states beyond which the structure no longer satisfies the design performance requirements. The fire situation is recognised by Eurocode 1 (EN 1991-1-2, 2004) to be an accidental situation that requires only verifications against the ultimate limit state (as opposed to the serviceability limit state). Ultimate states are these states associated with structural collapse or other similar forms of structural failure such as loss of equilibrium, failure by excessive deformation, formation of a mechanism, rupture or loss of stability.

In the semi probabilistic approach, the design against the ultimate limit state is based on the comparison between the resistance of the structure calculated with the design values of material properties, on one hand, and the effects of actions calculated with design value of actions, on the other hand, see Eq. 2.1:

$$R_{d,fi}(X_{d,fi}) > E_{d,fi}(F_{d,fi}) \quad (2.1)$$

where $R_{d,fi}$ design value of the resistance in case of fire

$X_{d,fi}$ design value of the material properties in case of fire,

$E_{d,fi}$ design value of effects of actions in case of fire,

$F_{d,fi}$ design value of the actions in case of fire.

The resistance and the effects of actions are both based on characteristic values of geometrical data, usually the dimensions specified in the design, for cross section sizes for example. Geometrical imperfections such as bar out of straightness or frame initial inclinations are represented by design values.

Eurocode 1 describes how the design values of actions, $F_{d,fi}$, are calculated.

The partial factor method considers that design values are derived from representative, or characteristic, values multiplied by scalar factors. The general equations are:

$$G_{d,fi} = \gamma_G G_k \quad \text{for the permanent actions} \quad (2.2)$$

$$Q_{d,fi} = \gamma_Q Q_k, \gamma_Q \psi_0 Q_k, \psi_1 Q_k \text{ or } \psi_2 Q_k, \quad \text{for the variable actions} \quad (2.3)$$

$$P_{d,fi} = \gamma_P P_k \quad \text{for the prestressing actions} \quad (2.4)$$

where G_k, Q_k, P_k characteristic values of the permanent, variable and prestressing action,

$G_{d,fi}, Q_{d,fi}, P_{d,fi}$ design values of these actions in case of fire,

$\gamma_G, \gamma_Q, \gamma_P$ partial factors for these actions

ψ_0 coefficient for combination value of a variable action, taking into account the reduced probability of simultaneous occurrences of the most unfavourable values of several independent actions,

ψ_1 coefficient for frequent value of a variable action, generally representing the value that is exceeded with a frequency of 0.05, or 300 times per year,

ψ_2 coefficient for quasi-permanent value of a variable action, generally representing the value that is exceeded with a frequency of 0.50, or the average value over a period of time.

Different actions generally occur simultaneously on the structure. In an accidental situation, they have to be combined as follows:

- Design values of permanent actions
- Design value of the accidental action
- Frequent or quasi-permanent value of the dominant variable action
- Quasi-permanent values of other variable actions.

When it is not obvious to determine which one amongst the variable actions is the dominant one, each variable action should be considered in turn as the dominant action, which leads to as many different combinations to be considered.

In case of fire, and if the variability of the permanent action is small, i.e. in most cases, the following symbolic equation holds as Eq. 2.5a or 2.5b:

$$E_{d,fi} = \gamma_{GA} G_k + \gamma_{PA} P_A + \psi_{1,1} Q_{k1} + \text{SUM}(i>1) \psi_{2,i} Q_{ki} \quad (2.5a)$$

$$E_{d,fi} = \gamma_{GA} G_k + \gamma_{PA} P_A + \psi_{2,1} Q_{k1} + \text{SUM}(i>1) \psi_{2,i} Q_{ki} \quad (2.5b)$$

where γ_{GA} partial factor for permanent action in accidental situation,

γ_{PA} partial factor for prestressing action in accidental situation.

The choice whether the frequent value (2.5a) or the quasi-permanent value (2.5b) has to be used for the dominant variable action is a nationally determined parameter.

The motivation to change from the frequent to the quasi-permanent value for the dominant variable action when the ENV were changed into prEN was that this is the solution used for earthquakes, which are also an accidental action, just as the fire.

The design value of the accidental action that has been mentioned previously does not appear in equation 2.5 because, in case of fire, the fire action is not of the same form as the other actions. It does not consist of some N or some N/m² that could be added to the dead weight or to the wind load. The fire action consists of indirect effects of actions induced in the structure by differential and/or restrained thermal expansion. Whether and how these effects have to be taken into account is discussed in Section 2.

Table A1.3 of Eurocode 0 (EN 1990, 2002) indicates that, for buildings, $\gamma_{GA} = 1.00$.

Table 2-1 given here is from Table A1.1 of Eurocode 0 (EN 1990, 2002) and gives the relevant ψ factors for the fire situation in buildings.

Table 2-1: coefficients for combination ψ for buildings

Action	ψ_1	ψ_2
Imposed load in buildings		
category A: domestic, residential	0.5	0.3
category B: offices	0.5	0.3
category C: congregation areas	0.7	0.6
category D: shopping	0.7	0.6
category E: storage	0.9	0.8
Traffic loads in buildings		
category F: vehicle weight $\leq 30\text{kN}$	0.7	0.6
category G: $30\text{kN} < \text{vehicle weight} < 160\text{kN}$	0.5	0.3
category H: roofs	0.0	0.0
Snow loads for $H < 1000$ m amsl	0.2	0.0
Wind loads	0.5	0.0

The design values of the material properties, $X_{d,fi}$, are described for each material in the Eurocode 2-1-2. The general equation is equation 2.6.

$$X_{d,fi} = \frac{X_k(\Theta)}{\gamma_{M,fi}} \quad (2.6)$$

with $\gamma_{M,fi}$, the partial safety factor for material property in fire design, being normally taken as 1.00.

The rationale for using 1.00 as a partial safety factor for material properties and for the actions lies in the theory of conditional probabilities. Let us assume that the probability of failure at ambient condition meets a particular target value, for example 7.23×10^{-5} . This can be expressed by equation 2.7.

$$P(\text{failure at ambient conditions}) \leq \text{Target Value} \quad (2.7)$$

The probability that the structure ever fails in a fire is the product of two probabilities: the probability that a severe fire occurs and the probability that this fire causes failure, see equation 2.8

$$P(\text{failure in fire condition}) = P(\text{there is a fire}) \times P(\text{failure caused by this fire}) \quad (2.8)$$

This probability has to meet the same target value as the one chosen at ambient temperature, see equation 2.9.

$$P(\text{failure in fire condition}) \leq \text{Target value} \quad (2.9)$$

Equation 2.10 can then be written immediately from 2.8 and 2.9.

$$P(\text{failure caused by the fire}) \leq \text{Target value} / P(\text{there is a fire}) \quad (2.10)$$

Because the probability that there will ever be a fire during the lifetime of a structure is smaller than 1.00, the probability that failure is caused by this fire is allowed to be higher than the probability of failure at ambient temperature (and the probability of failure from a fire during the lifetime of the structure will anyway be the same as the probability of failure at ambient temperature). This is why more favourable values of the partial safety factors are used in the fire situation, as well as in any accidental situation.

Eurocode 2-1-2 also describes how the resistance, $R_{d,fi}$, based on these material properties, is calculated.

2.3.2 Influence of time

When designing a concrete structure at room temperature, the design value of the compressive strength is calculated by dividing the characteristic value by the partial safety factor. A multiplicative factor $\alpha_{cc} \leq 1.0$ may be taken into account (0.85 for example) to calculate the value of the compressive strength that will finally be used, see equation 2.11 (EN1992-1-1, 2004).

$$f_{c,d} = \alpha_{cc} f_{c,k} = \alpha_{cc} \frac{f_{c,k}}{\gamma_M} \quad (2.11)$$

This factor accounts for the fact that the characteristic value of the compressive strength is determined from tests that are made within a certain time scale, usually some seconds or minutes, that is by several orders of magnitude smaller than the normal lifetime of concrete structures, typically several decades. If the compressive loading on the concrete specimen

could be maintained for a sufficiently long duration (but this would be unrealistic), the resistance would be smaller, in the order of 0 to 20% lower.

Because the duration of a typical fire, several minutes or a few hours, is closer to the duration of experimental tests than to the lifetime of the structure, this factor α_{cc} is not taken into account for a design in the fire situation.

The same would not hold, for concrete structures that are submitted to elevated temperatures during a very long duration such as, for example, industrial furnaces. For such a situation, it would be wise to determine experimentally the value of the factor α_{cc} at elevated temperature and certainly not assume that it keeps the same value as at room temperature.

The age of the structure that has to be considered for the fire design depends on the objectives of the design. If the objective is to save the life of the occupants, the design must be made at the date of first occupancy. This will cover any situation later in time because the strength of concrete has a tendency to increase with time, whereas the moisture content that may trigger the phenomenon of spalling usually decreases with time. Verifications at a later stage might be envisaged for prestressed structures.

If the objective is to protect the worker on the construction site, verification at an early age can be done, but a lower safety level could be taken into account owing to the temporary duration of the risk.

References

EN 1990: "Eurocode – Basis of structural design", 2002.

EN1991-1-2: "Eurocode 1: Actions on structures. Part 1-2: General Actions – Actions on structures exposed to fire", December 2004.

EN1992-1-1: "Eurocode 2: Design of Concrete Structures - Part 1-1: General rules and rules for buildings", December 2004, 225 pp.

EN1992-1-2: "Eurocode 2: Design of Concrete Structures - Part 1-2: General rules – Structural Fire Design", December 2004, 97 pp.

fib Bulletin 38: "Fire Design of Concrete Structures – Materials, Structures, and Modelling." *fédération internationale du béton*, Lausanne, Switzerland, 2007, 106pp.

Haksever, A. and Walger, R. "Dehnbehinderte Stahlbeton Plattenstreifen und TT-platten im Brandfall". Sonderforschungsbereich 148, Teil 1 Arbeitsbericht 1978-1981. TU Braunschweig, 1980.

Fellinger, J.H.H.: "Shear and Anchorage Behaviour of Fire Exposed Hollow Core Slabs", PhD Thesis, DUP Science, Delft Universe Press, ISBN 90-407-2482-2, Delft, The Netherlands, 2004, 261 pp.

Schneider U. ed.: "Properties of materials at high temperatures. Concrete", Gesamthochschul Kassel, 1985.

3 Sectional analysis*

3.1 Introduction

Sectional analysis of R/C members subjected to a fire is of fundamental importance to quantify the safety level of any given structural member without - or with limited - redistribution capacity. However, in spite of the many studies devoted so far to sectional analysis, a number of topics are still open to discussion. For instance, whether nonlinear analysis with proper strain limitations can be extended to a fire situation, and whether the eigenstresses due to the thermal gradients can be neglected are two questions still to be answered. In this chapter, four issues are addressed: (a) the use of nonlinear analysis implemented with simplified constitutive laws, as an alternative to realistic (but more complex) laws, at room temperature (EN 1992-1-1); (b) the use of incremental-iterative procedures ("exact" method) and nonlinear analysis in fire conditions; (c) the validity of the well-known 500°C-isotherm method (EN 1992-1-2), in fire conditions, under an eccentric axial force; and (d) the relevance of the eigenstresses generated by the thermal gradients.

3.2 Nonlinear analysis applied to R/C sections under fire

The bearing capacity of R/C sections subjected to a fire is usually evaluated by means of different approaches:

- by using tabulated data (first-level method, see EN 1992-1-2, Section 5);
- by using the 500°C-isotherm method (see EN 1992-1-2, Annex B1) or the zone method (see EN 1992-1-2, Annex B2), both being second-level methods;
- by using stress-strain, temperature-dependent laws, such as those proposed in EN 1992-1-2, Section 3.2.2.1, within the framework of an incremental-iterative procedure (third-level method).

In all cases, the thermal analysis is carried out before the mechanical analysis, since the thermal properties are hardly affected by the load-induced stresses. Moreover, the effects that the reinforcement has on the thermal field are neglected in most cases since the high thermal diffusivity of the steel always guarantees the thermal equilibrium between the bars and the surrounding concrete ($T_{\text{bar}} = T_{\text{concrete}}$). However, in some cases with densely-spaced reinforcement and rather small covers the heat transfer through the bars cannot be disregarded.

When applying the 500°C isotherm or the zone methods, since the thermal field is not explicitly introduced in each point of the section, the thermal strains ensuing from the heating process are neglected. On the contrary, when using advanced methods (EN 1992-1-2, 4.3.3), where the temperature in each point is explicitly introduced, it is possible to account for the free thermal strains and the resulting eigenstresses.

3.2.1 Tabulated data

The tabulated data are based on past experience and on the theoretical evaluation of tests (Naranayan and Beeby, 2005). These data provide a set of admissible values for the main geometric parameters of a section, including the cover of the reinforcement, as a function of the fire duration that the element is required to withstand. This approach allows the designer

* by Patrick Bamonte and Alberto Meda

to give a quick response in many practical cases with well-defined boundary conditions; on the other hand, this approach does not allow the designer to refer to materials' properties and fire scenarios other than ordinary concrete and the standard ISO834 Fire Curve. Neither the mechanical, nor the thermal aspects of the problem are explicitly addressed by this approach.

3.2.2 Reference-isotherm method (500°C isotherm)

The "reference-isotherm" method (or "effective-section" method) is based on the assumption that concrete is fully damaged above the temperature of 500°C, while it is fully effective (fully undamaged) for temperatures below 500°C (Fig. 3-1). On the contrary, the mechanical decay of the reinforcing steel is explicitly introduced.

This method can be applied within the context of nonlinear analysis, by assuming the parabola-rectangle stress-strain curve at ambient temperature for the concrete, with the usual strain limitations, and by considering only the undamaged part of the concrete section (the "effective section", which is the part of the section enveloped by the isothermal line 500°C, Fig. 3-1, Anderberg and Thelandersson, 1976).

This handy method is based on reasonable assumptions, and was originally devised for R/C sections subjected to pure bending, where the failure is generally controlled by the yielding of the tensile reinforcement. The possible extension to sections subjected to an eccentric axial force is still under discussion (Bamonte and Meda, 2005) and has to be considered carefully.

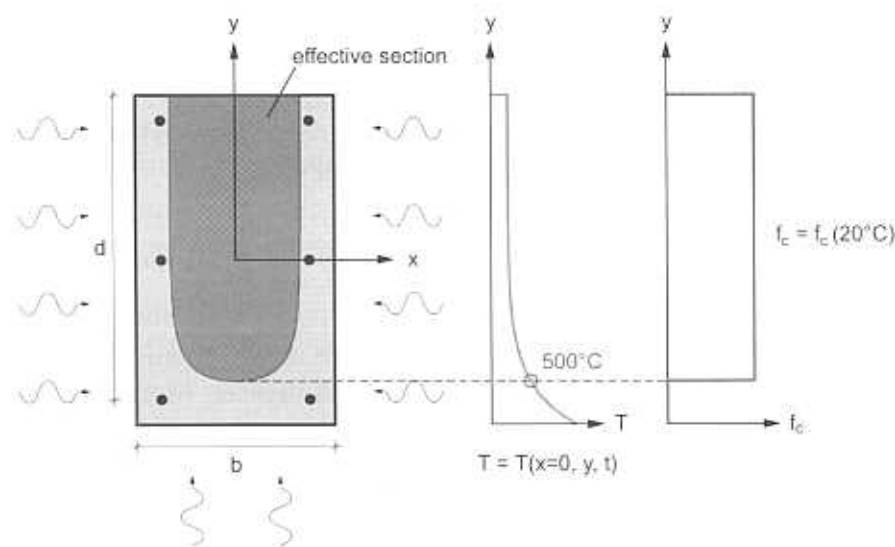


Fig. 3-1: The 500°C isotherm method applied to a rectangular section heated on three sides

3.2.3 Zone method

The zone method retains the philosophy of the 500°C-isotherm method, but considers a more complex and realistic reduced section, whose dimensions depend on the temperature distribution (Fig. 3-2). Also the characteristics of the concrete in the reduced section (compressive strength and Young's modulus) depend on the temperature distribution. In order to perform the calculation, the section is divided into a finite number of zones (≥ 3). The temperature is determined in the centroid of each zone, on the basis of the thermal analysis. The method is more complex than the 500°C-isotherm method, but yields better results (Naranayan and Beeby, 2005), especially in the case of pure compression. Moreover, the method allows to consider second order effects, by introducing proper correction coefficients.

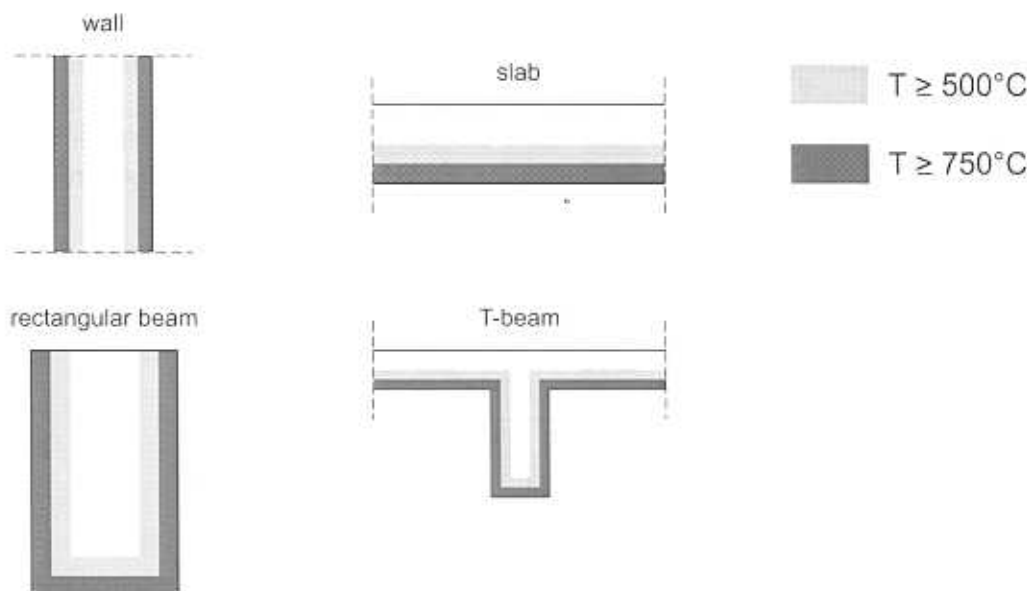


Fig. 3-2: Some applications of the zone method

3.2.4 Exact method – Incremental-iterative procedure

The incremental-iterative procedure is based on the temperature-dependent stress-strain curves, such as those proposed in EN 1992-1-2, Section 3.2.2.1. At first a thermal analysis is performed in order to determine the temperature distribution in the section, and thus the level of the thermal damage at each point, for any given fire duration. The mechanical properties of concrete and steel in each point can then be related to the maximum temperature reached locally (see for instance the decay suggested in EN 1992-1-2 for different types of concrete and steel), by means of the temperature-dependent stress-strain curves. In this way, the section is considered as a composite section, consisting of many different materials, whose properties and spatial distribution are related to the thermal field.

The next step is to determine the maximum value of the bending moment M_u for any given value N_u of the axial force, on the basis of the moment-curvature diagram of the section in question. This calculation is performed for different values of N_u ; the resulting couples of values $(N_u; M_u)$ identify as many points in the M - N domain and the interaction envelope is obtained by connecting these points.

An example of this procedure is shown in Figs. 3-3a and 3-3b, where the bearing capacity of a square section (300×300 mm, $4\phi 16$ rebars; $f_c = 30$ MPa, $f_y = 500$ MPa, siliceous aggregates) is evaluated at room temperature (Fig. 3-3a), starting from the moment-curvature diagrams corresponding to as many values of the axial force acting on the section. In Fig. 3-3b, the bearing capacity is evaluated for different values of the fire duration, starting from the moment-curvature diagrams corresponding to the same value of the axial force ($N_u = 500$ kN). In working out the thermal fields of the section, reference was made to the mean value between the upper and the lower limit curves suggested for the conductivity λ by EN 1992-1-2 (EN 1992-1-2, 3.3.3). The other values of the thermal properties were taken according to Table 3-1 (see EN 1992-1-2, 3.3).

Table 3-1: Materials properties assumed in the thermal analysis

T [°C]	ρ [kg/m ³]	c [kJ/K]	λ [W/mK]
20	2400	900	1.64
100	2400	900	1.50
200	2352	1000	1.33
300	2316	1050	1.18
400	2280	1100	1.05
500	2259	1100	0.93
600	2238	1100	0.83
700	2217	1100	0.75
800	2196	1100	0.68
900	2175	1100	0.63
1000	2154	1100	0.59
1100	2133	1100	0.58
1200	2112	1100	0.57

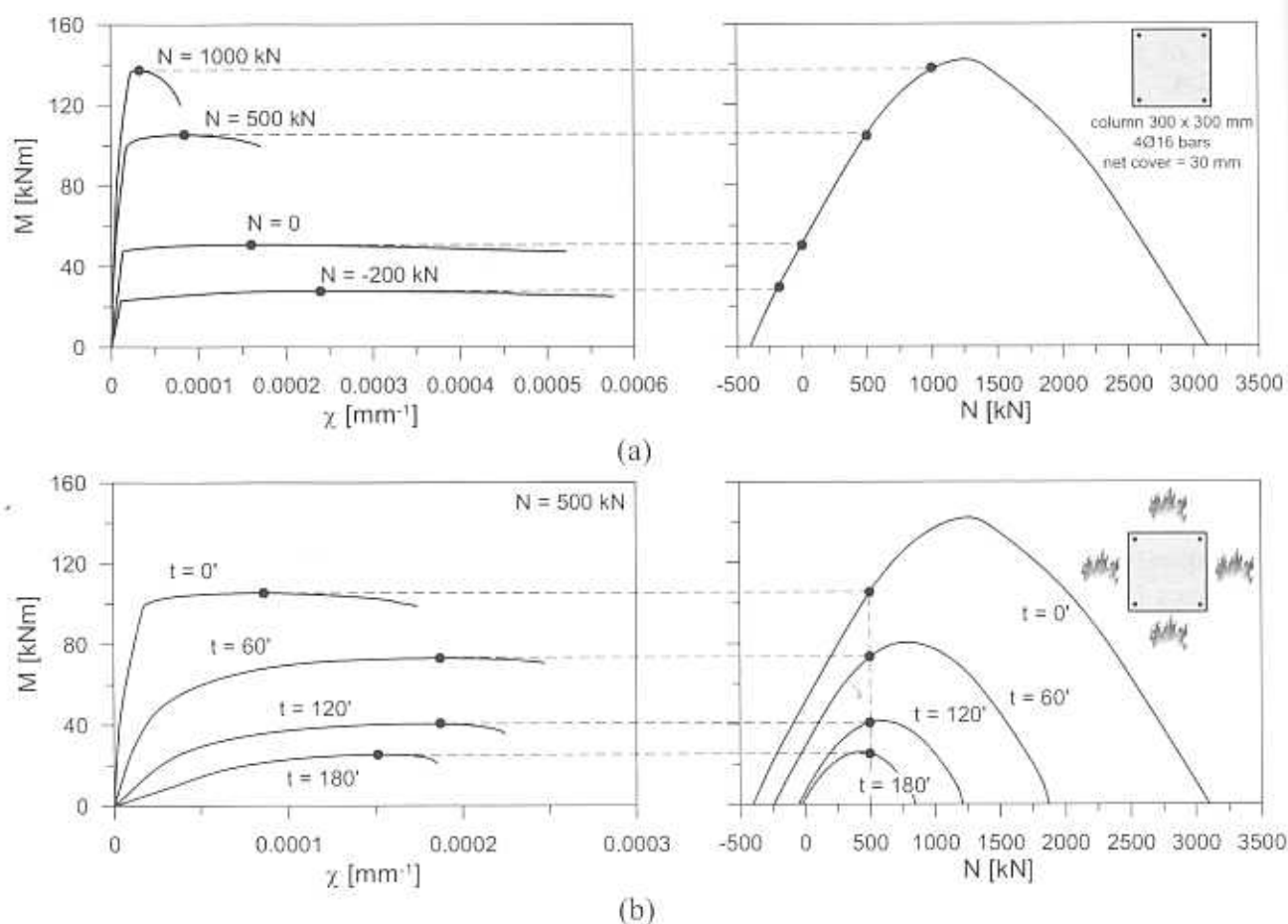


Fig. 3-3: Examples of the application of the incremental-iterative procedure at room temperature (a); and for different values of the fire duration (b)

This procedure is rather time consuming and burdensome for practical design, when compared with the previous methods (500°C-isotherm and zone methods). Nevertheless, nonlinear analysis based on strain limitations cannot be used with the stress-strain curve proposed in EN 1992-1-2, Section 3.2.2.1 (Fig. 3-4, dash-dotted curve, with the softening

branch characterized by large strains and decreasing strength), because the attainment of the ultimate strains in one of the two materials does not correspond - in general - to the attainment of the ultimate bearing capacity (= maximum load-bearing capacity). In fact, when the ultimate strain is reached in the top concrete fiber, most of the section has already undergone unloading and exhibits stress values which are lower than the peak stress f_c .

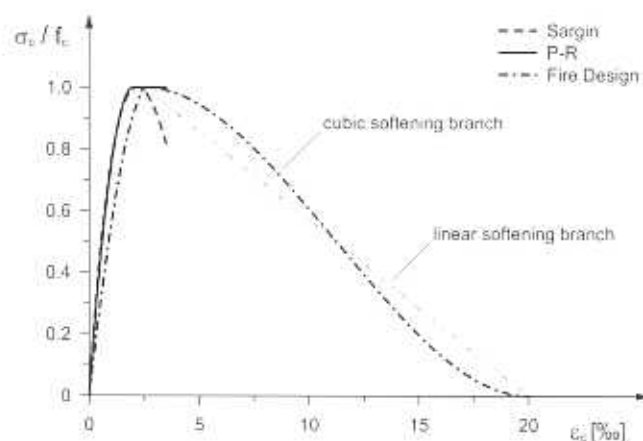


Fig. 3-4: Stress-strain diagrams for concrete at room temperature proposed in EN 1992-1-1 (P-R = Parabola-Rectangle and Sargin) and in EN 1992-1-2 (Fire Design)

Fig. 3-5 shows the three different M-N interaction envelopes, obtained by means of nonlinear analysis (i.e. by imposing strain limitations on both steel and concrete), according to the three constitutive laws plotted in Fig. 3-4 (parabola-rectangle diagram = continuous curve; Sargin diagram = dashed curve; EN 1992-1-2 Diagram = dash-dotted curve). It clearly appears that the results are completely different, and that the bearing capacity of the section is greatly underestimated when a constitutive law characterized by a fully-extended softening branch is used.

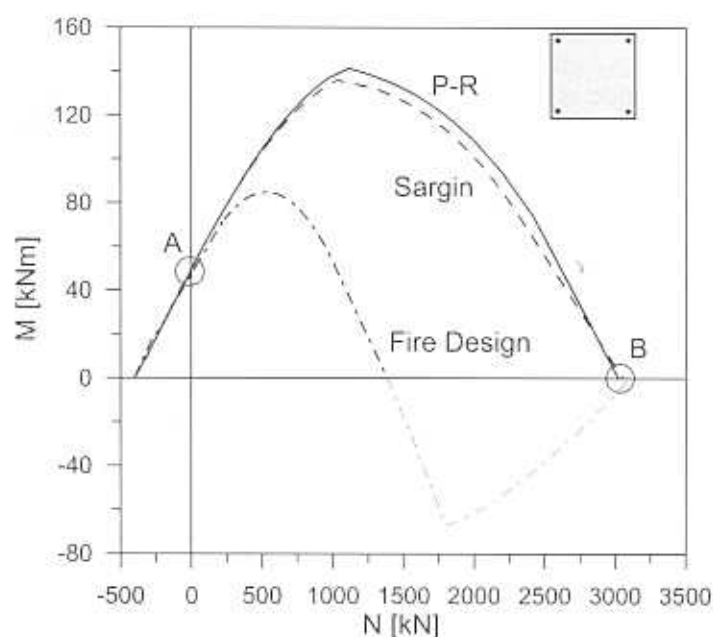


Fig. 3-5: Interaction envelopes for a square section (300 x 300 mm, 4Ø16 rebars; $f_c = 30$ MPa; $f_y = 500$ MPa; net cover = 30 mm, see Fig. 3-3a) obtained by using nonlinear analysis, with three different stress-strain relationships for concrete (P-R = Parabola-Rectangle)

Although the bearing capacity in pure bending (point A) and in pure compression (point B) are almost the same in all cases (the slight differences in point B depend on the steel stress at ultimate), it is worth noting that the negative part of the dash-dotted curve (negative bending moment) is meaningless. Of course, the underevaluation of the sectional capacity is avoided if the EC2-Fire curve is used properly, i.e. by adopting the afore-mentioned incremental-iterative procedure.

It seems that in order to achieve a reliable result with non-linear analyses, the incremental iterative procedure will have to be followed.

3.3 Reference-isotherm method versus exact method

In the following, a comparison between the 500°C-isotherm and the exact method is carried out, by considering different R/C sections (Fig. 3-6), with the sides totally or partially exposed to the fire. The effects of the thermal strains (and ensuing eigenstresses) are neglected in all cases, in order to allow a consistent comparison and to limit the number of the parameters coming into play.

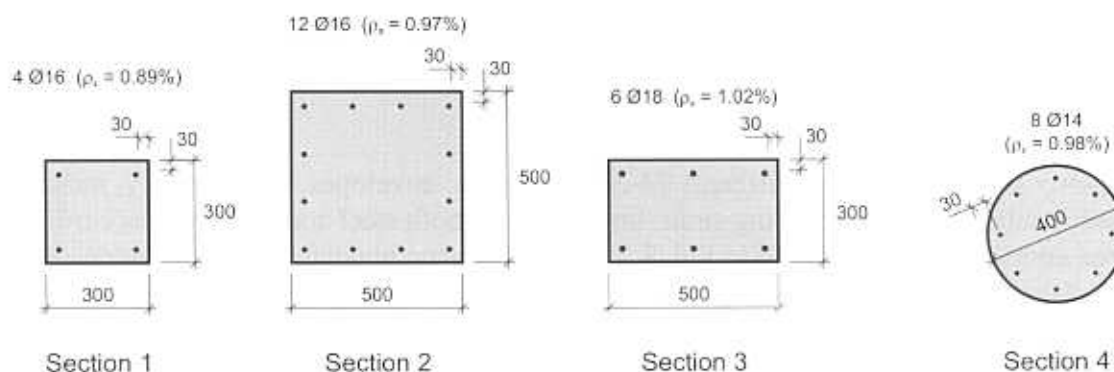


Fig.3-6: Reference sections considered in this study

To this purpose, reference is made to an ordinary concrete ($f_c = 30$ MPa) and to an equally ordinary steel ($f_y = 500$ MPa); furthermore, all sections have almost the same steel ratio ($\rho_s \approx 1\%$). Since the zone method is a sort of extension of the 500°C-isotherm method, only the latter is considered in the following.

The envelopes of Fig. 3-7 show that the larger the fire duration, the worse the agreement between the 500°C-isotherm method and the exact method, particularly under large axial forces. Of course the two methods give almost the same results in the case of large sections, since these sections are less temperature-sensitive. However the bearing capacity in pure compression is still somewhat overestimated by the isotherm method. In Fig. 3-7 the sides exposed to the fire are indicated by means of dashed lines in the inserts.

3.4 An alternative method based on strain limitations

The findings of the previous paragraphs clearly show that by introducing suitable temperature-dependent stress-strain curves (devoid of any softening branch or with a limited softening branch, Figs. 3-4 and 3-5) it is possible to use the simple nonlinear analysis approach instead of the more complex incremental-iterative procedure. These curves (see for instance the P-R curve in Fig. 3-4) do not represent the actual behavior of the concrete and

have to be validated against either the incremental-iterative procedure or appropriate test results, as was originally done with the parabola-rectangle curve (Eibl, 1995).

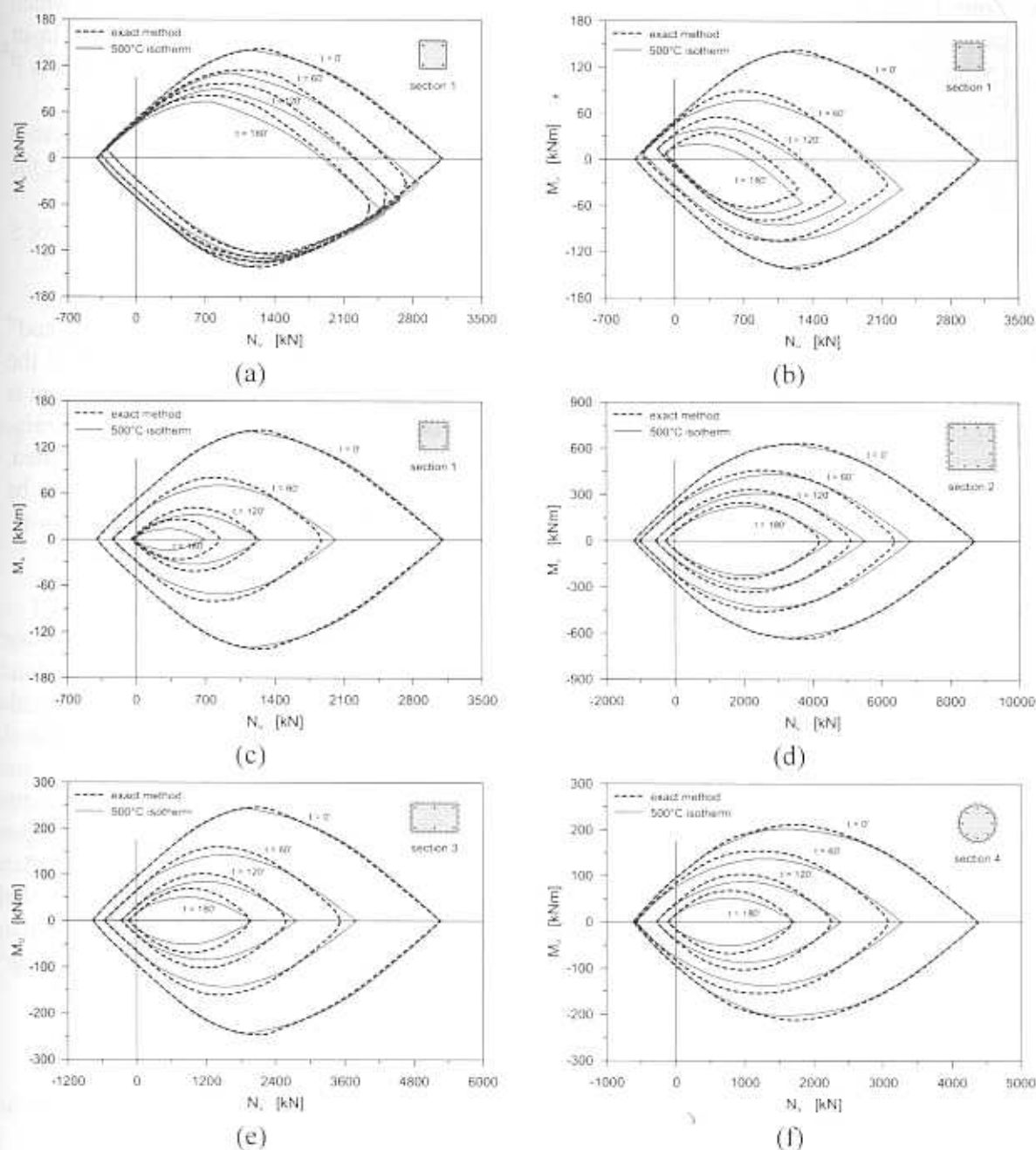


Fig. 3-7: Comparison between the 500°C isotherm and the exact method for different fire exposures (a,b,c) and for different section geometries (d,e,f). The heated sides of the sections are indicated with dashed lines.

In the following, an extension of the well-known method based on strain limitations (commonly used at room temperature) is proposed. For each fire duration, a number of linear strain profiles, all respectful of the ultimate strain in the reinforcement and in the concrete (ϵ_{cu}^T and ϵ_{su}^T), is considered. Since the concrete becomes more ductile with increasing temperature, it is sufficient to respect the ultimate strains along the coldest (and thus less ductile) chord to guarantee the respect of the ultimate strain in any other point. In the case of square sections heated on four sides the coldest chord is the mean chord (Fig. 3-8a). As in the

case of nonlinear analysis at room temperature, the strain profiles belong to three zones (Fig. 3-8b):

- **Zone 1**, ranging from the line l_0 , which represents the failure in pure tension, to l_1 , which corresponds to the simultaneous attainment of the ultimate strain in the bottom steel layer (point P) and in the top concrete fiber (point R): the fixed point of the strain profiles is P (= pivot);
- **Zone 2**, ranging from l_1 to l_2 , where the fixed point is R, until the strain profile becomes tangent to the ultimate-strain profile of the concrete; the strain profiles rotate around the pivot R;
- **Zone 3**, ranging from l_2 to l_3 : there is no fixed point for the strain profiles, but the pivot S moves along the ultimate strain profile, from R to Q.

Since in Zone 3 the pivot moves, this method has been called "mobile-pivot method" (Meda et al., 2002). It is worth noting that - as long as the temperature gradient is low at the periphery of the section - Zone 2 does exist, but it disappears when the temperature gradient is high (generally the gradient grows with the environmental temperature). For any linear strain-profile along the mean chord, the strain at each point of the section can be evaluated, assuming that plane sections remain plane. As a consequence, the stress distribution can be determined by using the local temperature-dependent σ_c - ϵ_c curves, and N_u and M_u can be evaluated by integrating the stresses.

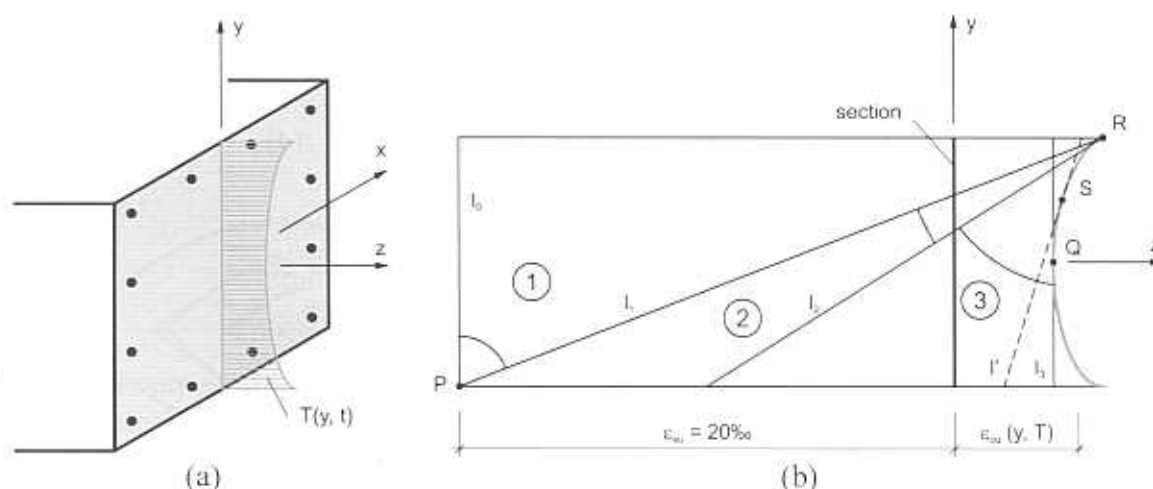


Fig. 3-8: (a) Temperature profile along the coldest chord in a square section heated on four sides; and (b) various strain profiles on the section, corresponding to as many ultimate situations

The temperature-dependent stress-strain curves, used in the following within the framework of nonlinear analysis, have been worked out on the basis of a best-fitting procedure applied to the N_u - M_u interaction envelopes previously calculated by means of the exact method, for different values of the fire duration: for each stress-strain curve the two parameters determined by means of the best-fitting analysis are the peak stress f_c^T and the ultimate strain ϵ_{cu}^T , while the strain ϵ_{c1}^T corresponding to the peak stress f_c^T was given the values suggested in EN 1992-1-2. Reference was made to different sections (square, rectangular and circular, Fig. 3-6), always heated on all sides, in order to simplify the problem in terms of symmetry conditions. The resulting stress-strain diagrams (Fig. 3-9) are compared with the curves exhibiting a complete softening branch proposed in EN 1992-1-2. It is worth

noting that - since the monotonous curves have a horizontal plateau corresponding to the peak stress - the ultimate strains are smaller.

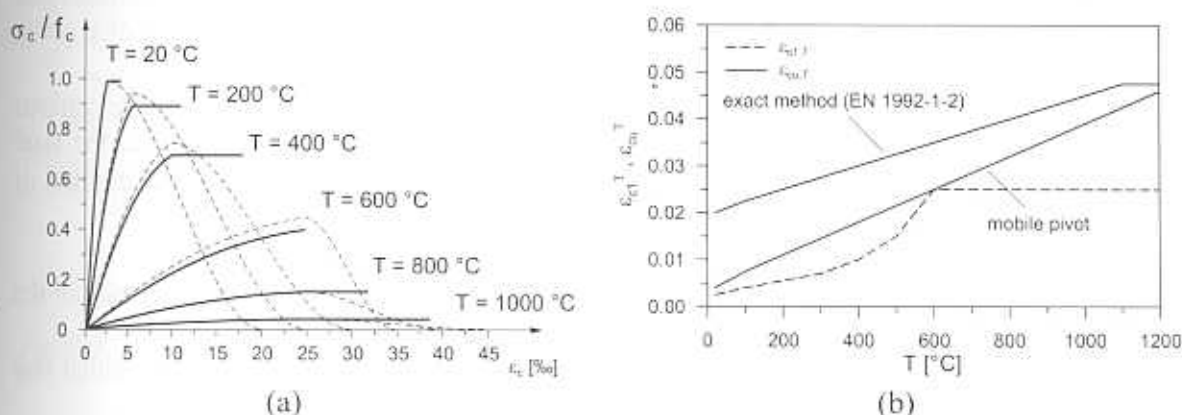


Fig. 3-9: (a) Temperature-dependent stress-strain diagrams for the concrete, to be used with strain limitatin analysis (continuous curves), and stress-strain diagrams proposed in EN 1992-1-2 (dashed curves), to be used with an incremental-iterative procedure (exact method); (b) peak and ultimate strains of the stress-strain curves on the left

The results obtained with the proposed method in the case of Section 1 are compared with those obtained with the exact method in Fig. 3-10a. Especially for the largest values of the fire duration, there is a very good agreement between the exact method and the proposed method, whereas the “effective section” method markedly underestimates the bearing capacity, as already stressed in the previous paragraphs. Similar results (Fig. 3-10b) were obtained in the case of a larger square section (Section 2 in Fig. 3-6). The agreement is still very good particularly when the fire duration exceeds 120'; however, it is fair to say that also the 500°C-isotherm method is very accurate in this case, but this is nothing new, since the larger the section, the smaller the sensitivity to the fire. The advantage of the mobile-pivot method is that, for each value of the fire duration, it is on the safe side, whereas the 500°C-isotherm method overestimates the bearing capacity in pure compression, as mentioned before (see Fig. 3-7d).

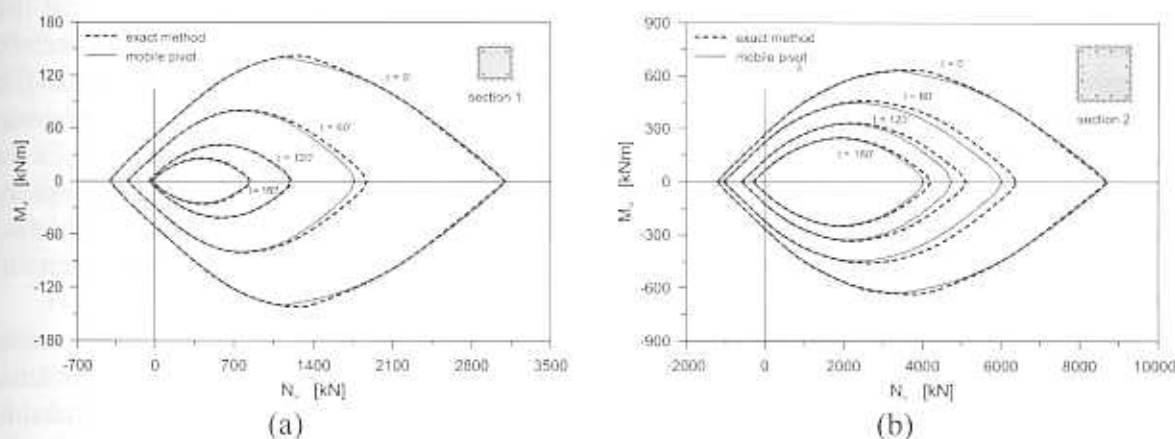


Fig. 3-10: N_u - M_u interaction envelopes for square sections heated on four sides: continuous curves = mobile pivot; dashed curves = “exact” method

3.5 The role of the thermal strains

Finally, some words should be devoted to the influence of the thermal deformations and whether they should be taken into account when evaluating the bearing capacity of a given section.

Including the effect of the thermal strains requires a clear understanding of the various assumptions concerning the different strain components acting on the section, i.e. the total strain ϵ_{tot} , the thermal strain ϵ_{th} and the stress-induced strain ϵ_{σ} , the last one being the part of the deformation associated with the state of stress. More specifically

1. the hypothesis that plane sections remain plane is still valid, but with reference to the total strain ϵ_{tot} ;
2. the usual nonlinear analysis with strain limitations can still be applied, provided that the limitations are referred to the stress-induced strain ϵ_{σ} .

In the case of moment-curvature diagrams the following procedure is applied:

- a reference value of the curvature χ is introduced;
- a tentative deformation ϵ_0 is assumed in the centroid of the section and the corresponding total deformation ϵ_{tot} is determined in each point of the section;
- the corresponding stress-induced deformation ϵ_{σ} is determined at each point of the section:

$$\epsilon_{\sigma} = \epsilon_{tot} - \epsilon_{th}$$

- the value of the stress σ is determined at each point, by using the constitutive laws given in the EN 1992-1-2;
- the internal actions N and M are calculated;
- the procedure is repeated until equilibrium in the axial direction is achieved within the required tolerance and within the limits concerning the stress-induced strains.

The incremental-iterative analysis including the thermal deformations was applied to the reference square section of Fig. 3-6a, for different values of the fire duration and for various exposures. The corresponding results are shown in Fig. 3-11 (dotted curves), where they are compared with the previous results, obtained by neglecting the thermal deformations (continuous curves, exact method). It is clear that the highest influence is played by the fire exposure: if the number of the exposed sides increases, the differences tend to become negligible (Fig. 3-11a), whereas they are no longer marginal if only one side is exposed to fire (Fig. 3-11b). On the contrary, the fire duration does not seem to play a major role. Moreover, it is worth observing that, with the exception of a small part of the interaction envelopes for one-side exposure, the M-N domains obtained by neglecting the thermal deformations are always contained inside the dotted curves. Summing up, it is confirmed that the thermal strains and the ensuing eigenstresses can be neglected when evaluating the bearing capacity of a R/C section in fire.

For the sake of completeness, in Fig. 3-12 the temperatures and the eigenstresses acting along the median and diagonal lines of the square section of Fig. 3-6 are plotted for three values of the fire duration (60', 120' and 180'). In all cases, the stress peaks are extended to very limited zones and do not exceed 2.5 MPa in compression for a 60' fire exposure (Fig. 3-12a) and 2 MPa for 120' and 180' (Fig. 3-12b).

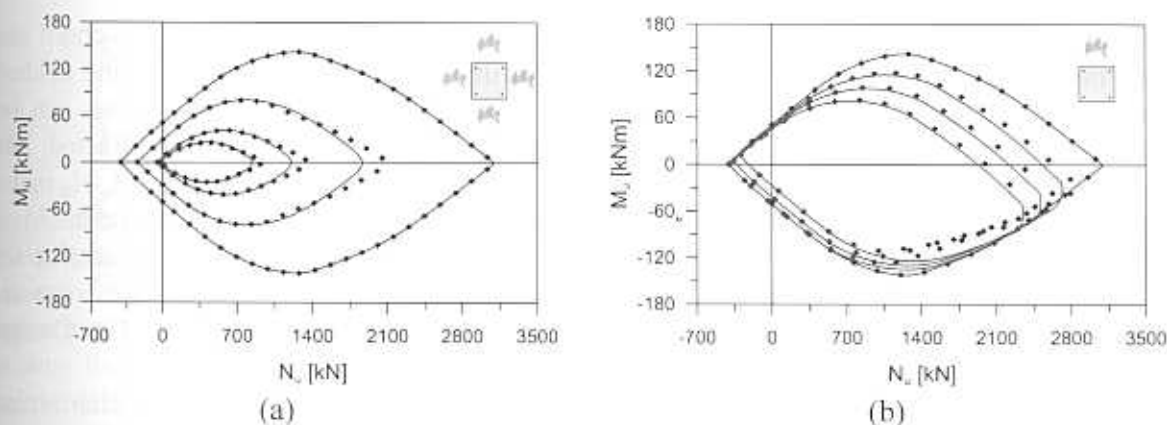


Fig. 3-11: Influence of the thermal strains on the bearing capacity: N_u - M_u interaction envelopes for square sections heated on four sides (a); and same section heated on one side.

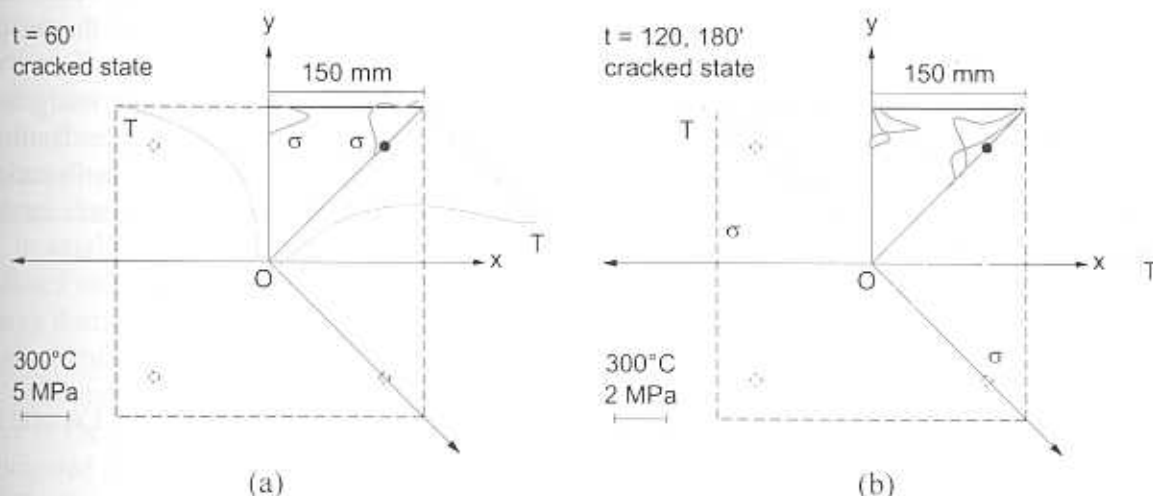


Fig. 3-12: Eigenstresses acting on a square section heated on four sides for different values of the fire duration: (a) 60'; and (b) 120', 180'.

3.6 Conclusions

Sectional analysis in fire conditions can be performed in different ways, by using the "reference-isotherm" method (generally based on the isotherm 500°C), the "zone" method, the "exact" method or the "mobile-pivot" method, the last being an extension of the method generally used in the design of R/C sections at room temperature.

The first two methods are based on rather crude assumptions, that make them simple and designer-friendly. However, these methods are either too conservative under an eccentric axial force and for large values of the fire duration, or even unsafe (in the case of small eccentricities).

The exact method requires the introduction of realistic stress-strain curves for the concrete (like the softening curves given in EC2-Fire Design) and the evaluation of the moment-curvature diagrams of the given section. Then, by means of a rather time-consuming and burdensome iterative procedure, the M_u value can be worked out for any given value of N_u .

The strain-limitation method is based on simple monotonous stress-strain curves, that are the extension of the well-known parabola-rectangle curve used in the design at room temperature. Each curve is valid for a given value of the temperature. After (a) the thermal analysis of the section has been performed for a given value of the fire duration; (b) the

“coldest” chord of the section has been identified; (c) the ultimate strain along this chord has been plotted; and (d) a number of linear strain profiles have been plotted along the coldest chord, so as to respect the ultimate strain in each point of the section, the stresses can be worked out for each strain profile, and – by integration – M_u and N_u can be evaluated. For each strain profile there is a single couple of values for M_u and N_u . The couples M_u - N_u make it possible to draw the envelope for the given value-of the fire duration. This method does not require iterative procedures and is as reliable as the exact method, provided that a suitable set of monotonous, temperature-dependent stress-strain curves is available for the concrete. These curves can be easily worked out from the softening curves proposed by the Fire Design Code.

Sectional analysis is sufficient to investigate the fire resistance of statically-determinate structures, but loses most of its significance in redundant structures (Biondini and Nero, 2006), where the redistribution of the internal forces from the sections with the “hot” tension reinforcement at the bottom to the sections with the “cold” tension reinforcement at the top is generally sizable. However, even if a more comprehensive analysis is needed in redundant structures, sectional analysis remains a first necessary step to understand whether the design of the section is sound in terms of fire resistance.

Finally, the thermal strains and the resulting thermal self-stresses have been investigated. The results obtained in the typical case of a square section heated along four sides confirm that the effects of the self-stresses on the M_u - N_u envelopes are minimal. This is a confirmation of the assumption commonly introduced in Fire Design, that the thermal self-stresses can be neglected.

References

- Anderberg Y. and Thelandersson S.: “Stress and Deformation Characteristics of Concrete at High Temperatures”, Lund Institute of Technology, Lund (Sweden), 1976, 84 pp.
- Bamonte P. and Meda A.: “On Fire Behavior of R/C Sections Subjected to an Eccentric Axial Force”, Proceedings of the Workshop “Fire Design of Concrete Structures: What now? What next?”, *fib* TG 4.3 “Fire Design of Concrete Structures”, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, Milan (Italy), 2-3 December 2004, pp.57-61.
- Biondini F. and Nero A.: “Nonlinear Analysis of Concrete Structures Exposed to Fire”, Proceedings of the 2nd International *fib* Congress, Naples (Italy), 5-8 June 2006.
- Eibl J. (editor): “Concrete Structures – Euro-Design Handbook 1994/1996”, Ernst & Sohn Verlag, 1995.
- EN1992-1-1: “Eurocode 2: Design of Concrete Structures - Part 1-1: General rules and rules for buildings”, December 2004, 225 pp.
- EN1992-1-2: “Eurocode 2: Design of Concrete Structures - Part 1-2: General rules – Structural Fire Design”, December 2004, 97 pp.
- Meda A., Gambarova P.G. and Bonomi M.: “High-Performance Concrete in Fire-Exposed Reinforced Concrete Sections”, *ACI Structural Journal*, V.99 (3), 2002, pp.277-287.
- Naranayan R.S. and Beeby A.: “Designer’s Guide to EN1992-1-1 and EN1992-1-2”, Thomas Telford, 2005, 218 pp.

4 Structural behaviour of continuous beams and frames*

4.1 Introduction

Member analysis is the main verification method suggested by Eurocode 2 (EN1992-1-2, 2004). It is in fact stated (point 2.4.1 of the code) that member analysis is sufficient to verify standard fire-resistance criteria. The verification by means of member analysis consists of comparing the design forces (bending moment, axial force and shear force) against the resisting forces, where the former are computed at ambient temperature, while the latter are evaluated by means of simplified methods, that consider the prescribed fire duration, as shown in Chapter 3.

The main objection raised against member analysis is that, by computing the design forces at ambient temperature, indirect actions arising in the structure due to thermal expansion are not taken into consideration, and that the time-dependent response of the structure is neglected. In statically-determinate members this objection appears to be hardly relevant. However, in statically-indeterminate members, member analysis could, in principle, lead to non-conservative results.

In fixed-end beams, the thermal gradient induces a constant bending-moment that generates tension on the side opposite to the heated face of the beam. This moment might in principle lead to an anticipated collapse of beam end sections.

In axially-restrained beams, a compressive axial force is induced in the beam due to the inhibited expansion. The effect of this axial force, combined with the considerable deflection arising during the fire, induces second-order effects, that may be relevant and may anticipate member collapse.

In frames, the continuity of the beams with the columns may induce a non negligible axial force in the beams, that in turn may generate high shear forces in the columns and trigger an anticipated shear collapse, as often observed in real fires.

To overcome these problems, EC2 suggests the possibility of carrying out the analysis of a part or of the complete structure subjected to the fire.

In the present Chapter, the behaviour (a) of a set of fixed-end beams with different sections and variable axial restraints, and (b) of a set of plane frames is discussed. The purpose of the study is: (i) to establish whether the axial restraint and second-order effects play a significant role in the structural response under fire; (ii) to check whether the effects of axial restraints can be safely neglected in carrying out the strength verifications of a beam under fire (criterion R); and (iii) to serve as the basis for the development of simplified plastic-verification procedures, that may be a valuable substitute to the analysis of a structural sub-assembly.

The analyses have been performed by using the materials models suggested in EC 2 and by assuming a standard fire scenario (ISO-834).

4.2 Modelling

The behaviour of R/C beams and frames exposed to fire is investigated by means of the Finite Element Method, by adopting a fiber-element model similar to those adopted in most FE codes for fire analysis (Franssen 2005, Riva 2005), implemented into the commercial code ABAQUS (HKS 2003).

* by Paolo Riva

In the proposed model (Fig. 4-1), the beams are subdivided into elements having a length equal to the smaller of either the stirrup spacing, or the beam depth. Each beam element is divided into non-linear fiber sub-elements, exhibiting only a monoaxial behaviour. The Navier-Bernoulli hypothesis is enforced on the element end-sections, thus enabling the element to represent both the bi-axial bending and the axial force. Shear and torsion are transferred between two contiguous elements by means of a set of linear springs, representing the elastic shear and torsion stiffnesses. Hence, the axial force and the bending moment are uncoupled from shear and torsion.

Any mechanical and thermal constitutive law may be adopted for the fibre sub-elements representing the concrete and the reinforcing bars.

The structural responses of R/C beams and frames under fire conditions are computed on the basis of the fibre sub-elements temperature history, that is determined by means of an uncoupled FE transient thermal analysis where the mesh coincides with that of the stress analysis (i.e. the number of elements and the position of their centroids coincides in both the thermal and mechanical analyses). Should the fire scenario be constant all over the entire structural element (either a beam or a column, each of constant section), the thermal analysis would be greatly simplified, since 2D modelling can be adopted for the cross-section, and the same temperature distribution holds for all the sections of the structural element.

In the analyses presented in the following, the mechanical properties of concrete and reinforcing steel sub-elements have been taken from EC 2 as a function of the temperature (Fig. 1).

Concerning concrete in compression, one should remember that the EC2 model is a purely phenomenological model, aimed at structural analysis. It does not introduce explicitly LITS (Load-Induced Thermal Strain), which is defined as the sum of the mechanical and total creep-strain, the latter including (a) the transitional thermal creep, (b) the basic creep and (c) the drying creep. Instead, the constitutive law is defined in terms of stress-vs-strain, according to Anderberg's work (Anderberg and Thelandersson 1976). From the definitions, it appears that the EC2 curves may be seen as stress-vs-total strain curves, rather than stress-vs-mechanical strain curves.

To demonstrate this statement, the response of a concrete specimen subjected to constant compression stress up to failure under an ISO 834 fire was studied by using either the EC2 model and Terro's model (Terro 1998), which explicitly introduces LITS and mechanical strains. The results, shown in Fig. 4-2, demonstrate that the strain histories obtained with the EC2 formulation slightly underestimate the total strain (and LITS), even though the overall behaviour is similar. The results confirm that EC2 constitutive law may be adopted in structural analysis, provided that it is treated as a stress-vs-total strain law, its inherent limitation being that the mechanical strains cannot be separated from transient-creep contribution. It follows that EC2 formulation is a viable tool in assessing structural safety, but cannot be used whenever transient creep effects have to be studied in detail. Furthermore, EC2 model is much simpler and can be more easily implemented than the models based on the explicit introduction of transient creep. However, the use of EC2 model is limited to beam analysis. A further limitation of the EC2 model concerns concrete spalling, which is not considered. More general models, applicable to any structure in fire conditions, are described in *fib* Bulletin 38 (2007).

As for concrete behaviour in tension, the post-cracking response has been modelled by means of a cohesive crack model, on the assumption that (a) the crack spacing is equal to the element length (= stirrup spacing), and (b) the decay of the tensile strength with temperature agrees with EC 2. Finally, a small residual tensile strength ($0.05f_{ct}$) has been assumed for numerical-stability purposes. However, it should be observed that concrete behaviour in tension has often a limited influence on the overall bending response of R/C structures, particularly with respect to the ultimate limit state.

The thermal analysis is performed by adopting the material properties (concrete mass per unit volume, specific heat and thermal conductivity) suggested by EC 2 (Figs. 4-3b-d), while the standard ISO-834 temperature-time curve is used (Fig. 4-3a) for the fire scenario. Finally, the presence of the reinforcement has been neglected in the thermal analysis, and the reinforcement temperature has been assumed equal to the concrete temperature in the centroid of each bar.

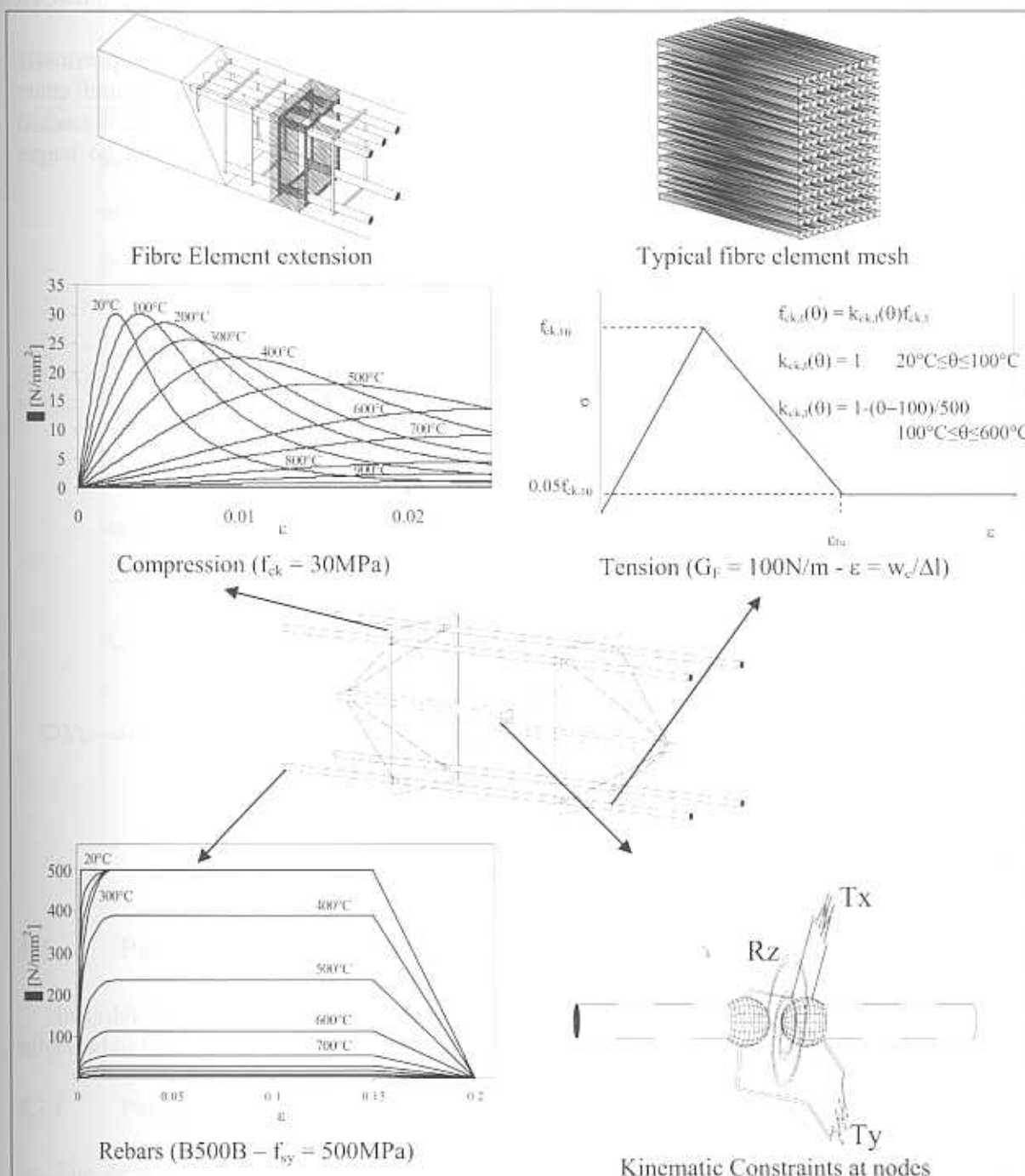


Fig. 4-1: Fibre-Element Model

The numerical procedure is summarized in the following:

1. The FE mesh of the given structure is defined for both the thermal and static analyses;
2. The transient thermal analysis is performed in accordance with the given fire scenario, in order to determine the thermal field in the structure at fixed time intervals (e.g. every 2');;
3. The initial static conditions at ambient temperature (20 °C) are determined by carrying out the static analysis under the applied loads ($\gamma_{GA}G_k + \psi_{1,1}\gamma_{QA}Q_{k,1} = G_k + 0.5Q_{k,1}$, according to EC 1);
4. The structural response under fire conditions is computed, by applying the previously determined temperature history and distribution. The analysis is carried out until either the desired time of fire exposure (for instance 60') or the structural collapse is reached (i.e. when equilibrium conditions are no longer satisfied or the analysis no longer converges).

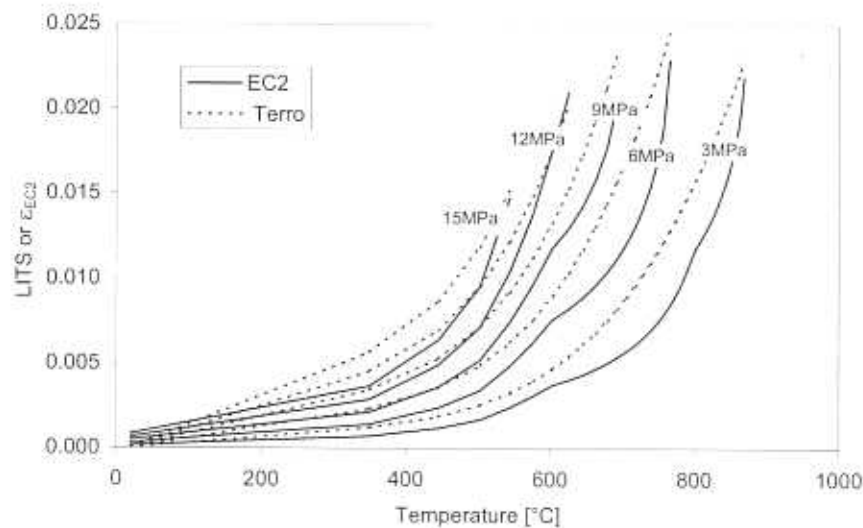


Fig. 4-2: Concrete specimens under constant compression and subjected to ISO 834 fire: comparison of EC2 and Terro's constitutive laws for different stress levels (concrete $f_c=30$ MPa)

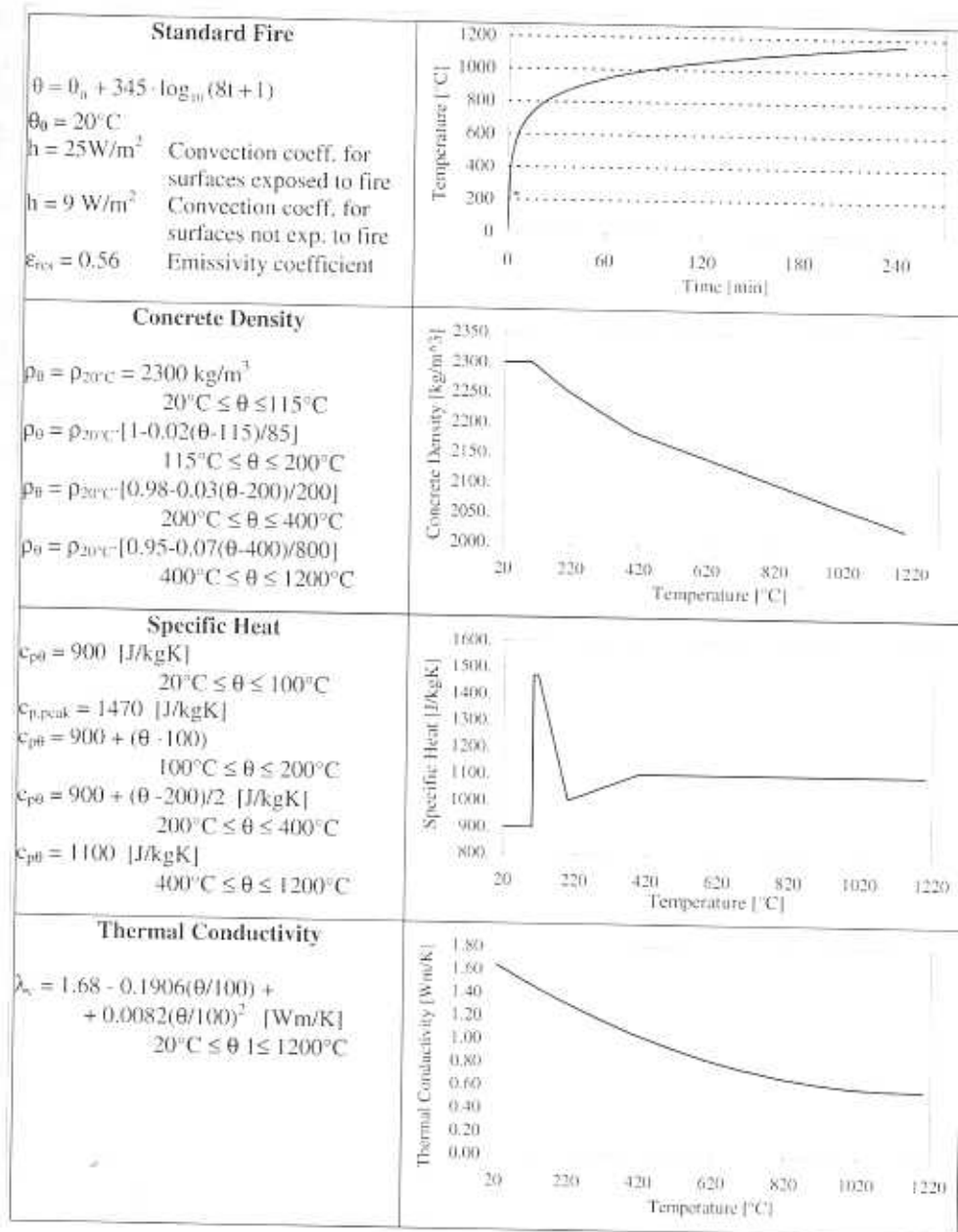


Fig. 4-3: ISO 834 fire and concrete thermal properties (EC2 2005)

4.3 Parametric study

In order to study the effects that the end restraints have on typical beams and frames subjected to ISO-834 fire, a parametric study was carried out as described in the following.

4.3.1 Parametric study of beams

The behaviour of a set of fixed-end beams with a variable axial restraint has been investigated with the proposed model. This investigation was aimed at clarifying the influence of the boundary conditions when a continuous beam is subjected to a fire along one of its spans (the fire compartment), while the remaining spans are in ordinary environmental conditions and behave as an axial and bending restraint of constant stiffness.

The beams have three different sections (T, rectangular and rectangular representing a strip of a one-way slab), two span lengths (6 m and 9 m) and four values of the axial restraint (equal to infinity, EA/L, EA/3L, zero, where A and L are the beam sectional area and the span

length, respectively). The axial restraint stiffness was chosen so as to represent a continuous beam of either one, two, four, or an infinite number of spans, fully restrained at the ends.

The design of the beams was carried out according to EC 2. In order to obtain a reinforcement arrangement representing the inner span of a continuous beam, some degree of redistribution was adopted.

The analyses have been performed by considering large displacements, thus introducing second-order effects, that are directly related to the axial stiffness. The full set of beams is summarized in Fig. 4-4.

The results of the tests carried out on the 6m-beams with rectangular section are here discussed. Some relevant observations concerning also the other sections are reported in the conclusions of the present sub-chapter. The complete set of results of the parametric study are reported in Appendix 4.

The beams have a rectangular section with width and depth $b=350$ mm and $h=500$ mm, respectively. The design was carried out according to EC2, assuming for the loads the values shown in Fig. 4-4. Fire resistance was verified with the tabulated method, by assuming 60' of fire exposure (R60). The reinforcement in the critical sections is shown in Fig. 4-4.

The thermal analysis was carried out considering the fire as acting on three sides, the fourth being in adiabatic conditions (Fig. 4-4), as it would approximately occur when the beam supports a floor. The results of the thermal analysis are illustrated in Fig. 4-5 with reference to three significant values of fire duration. Each beam section was divided in elements having dimensions 10×10 mm or 5×10 mm.

Fig. 4-6 illustrates the results when the axial restraint is equal to EA/L . The following comments can be made:

- the collapse is reached after 180', although the design fire rating was R60, proving that the tabulated method leads to very conservative design solutions, at least when compared to analytical results;
- the displacement after 180' is equal to 34.6 mm, twelve times larger than the initial one (2.78 mm). Though the displacement has undergone a considerable increase, its value ($\cong L/60$) still respects the insulation requirements (I) and the integrity requirements (E), EN1991-1-2 (2004);
- after 30' of fire duration, the bending moment diagram shifts upward, due to the constant negative bending moment generated by the thermal gradient. During this period, no relevant stiffness degradation is observed in either the end- or span-sections, as demonstrated by the moment-curvature ($M-\phi$) diagrams (Figs. 4-5 h, i);
- between 30' and 60', a bending moment reduction at the end sections is observed, together with an increase of the mid-span bending moment. This reduction is related to the damage of the outer concrete layers, that has two effects: (i) a larger stiffness degradation is observed in the end sections, compared to that of the mid-span section; and (ii) the axial force has its centre of thrust located above the beam geometric axis, thus generating a constant positive bending moment in the beam (as will be discussed later in more depth).

MATERIALS:		CROSS SECTIONS		
Concrete C30/37		RECTANGULAR SECTION	ONE WAY SLAB	RECTANGULAR SECTION + SLAB
Reinforcing steel B500B				
STRUCTURAL MODELS	TRANSLATIONAL STIFFNESS k			
LOADS: Rectangular section: $DL = 36.4 \text{ kN/m}$ $LL = 12.4 \text{ kN/m}$ Rectangular section + slab: $DL = 36.4 \text{ kN/m}$ $LL = 12.4 \text{ kN/m}$ One way slab: $DL = 7.25 \text{ kN/m}^2$ $LL = 4.4 \text{ kN/m}^2$	0	$B = 35 \text{ cm}$ $H = 50 \text{ cm}$ SECTION A-A $4\phi 20 + 2\phi 20$ SECTION B-B $2\phi 20 + 2\phi 20$	$B = 125 \text{ cm}$ $H = 25 \text{ cm}$ SECTION A-A $6\phi 12$ $6\phi 12$ SECTION B-B $6\phi 12$ $6\phi 12$	$B = 100 \text{ cm}$ $h_s = 10 \text{ cm}$ $H = 40 \text{ cm}$ $h_b = 30 \text{ cm}$ SECTION A-A $5\phi 20 + 2\phi 20$ SECTION B-B $2\phi 20 + 3\phi 20$
LOADS: Rectangular section: $DL = 54.4 \text{ kN/m}$ $LL = 18.4 \text{ kN/m}$ Rectangular section + slab: $DL = 54.4 \text{ kN/m}$ $LL = 18.4 \text{ kN/m}$ One way slab: $DL = 9.75 \text{ kN/m}^2$ $LL = 4.4 \text{ kN/m}^2$	0	$B = 55 \text{ cm}$ $H = 75 \text{ cm}$ SECTION A-A $7\phi 20 + 3\phi 20$ SECTION B-B $2\phi 20 + 5\phi 20$	$B = 140 \text{ cm}$ $H = 35 \text{ cm}$ SECTION A-A $8\phi 16$ $6\phi 16$ SECTION B-B $6\phi 16$ $6\phi 16$	$B = 135 \text{ cm}$ $h_s = 15 \text{ cm}$ $H = 75 \text{ cm}$ $h_b = 40 \text{ cm}$ SECTION A-A $7\phi 20 + 3\phi 20$ SECTION B-B $2\phi 20 + 5\phi 20$

Fig. 4-4: Parametric study of the beams.

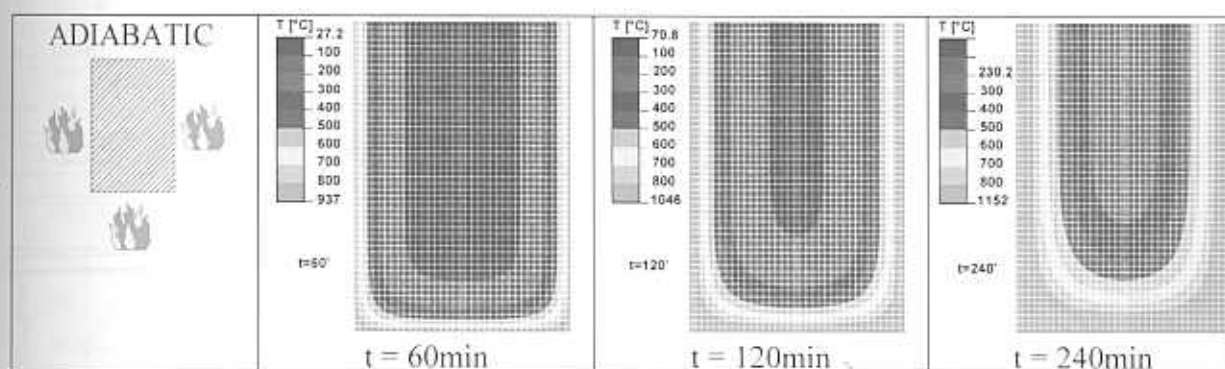


Fig. 4-5: Results of the thermal analysis [$^{\circ}\text{C}$]

- between 60' and 120', due to the progressive damage of the beam and to the increasing contribution of the axial force to the positive bending moment, the bending-moment diagram shifts downward and its values approach the initial ones;
- the collapse is controlled by the end sections, as soon as they reach their ultimate capacity (180');

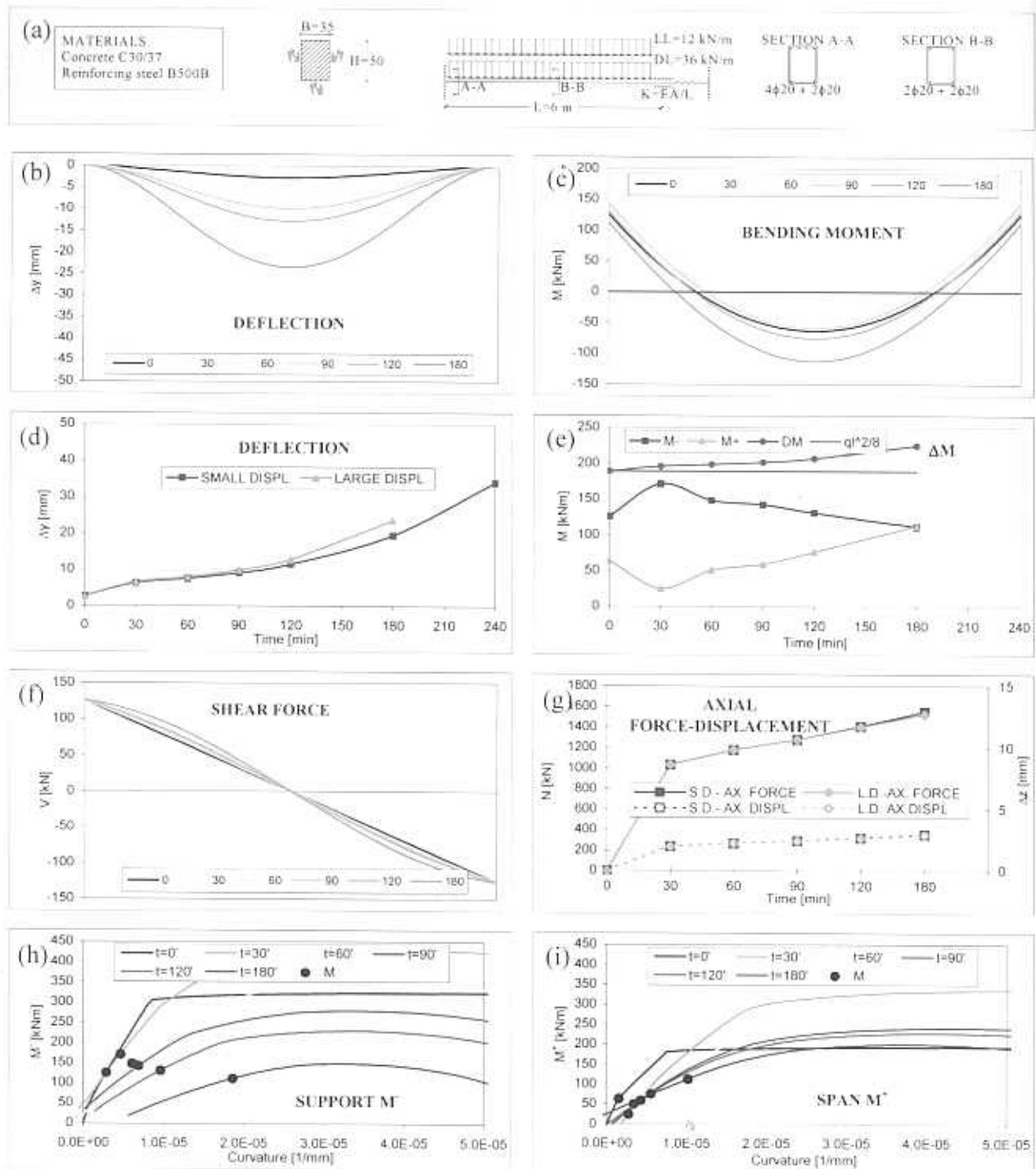


Fig. 4-6: Behaviour of a 6m span rectangular beam with axial restraint of stiffness $k = EA/L$.

- the bending moment due to second-order effects increases almost linearly with time, and after 180' it is approximately 20% higher than the first order bending moment (Fig. 4-6e).
- due to second-order effects, an increase of the shear force in the sections close to the fourths of the span is observed. (Even if these zones are not critical in shear at ambient temperature, they may become critical at high temperature, and the shear reinforcement may be no longer sufficient).
- the axial force increases rapidly in the first 30' of fire duration. Later, its increase is much less significant due to concrete damage. From 30' to 180' the axial-force increase is only 6% of the increase in the first 30'. The axial force tends to move upward with respect to beam axis because of the progression of concrete damage under fire;

- the axial displacement increases rapidly during the first 30' of fire duration. Later it remains almost constant, with rather small values (close to 2.9mm);
- the beam deflection is only marginally affected by second-order effects, while the collapse is anticipated (180' vs. 240' by considering only the first-order effects);
- the collapse occurs when end-section capacities are reached, because of excessive concrete damage.
- due to second-order effects, an increase of the shear force in the sections close to the fourths of the span is observed. (Even if these zones are not critical in shear at ambient temperature, they may become critical at high temperature, and the shear reinforcement may be no longer sufficient).

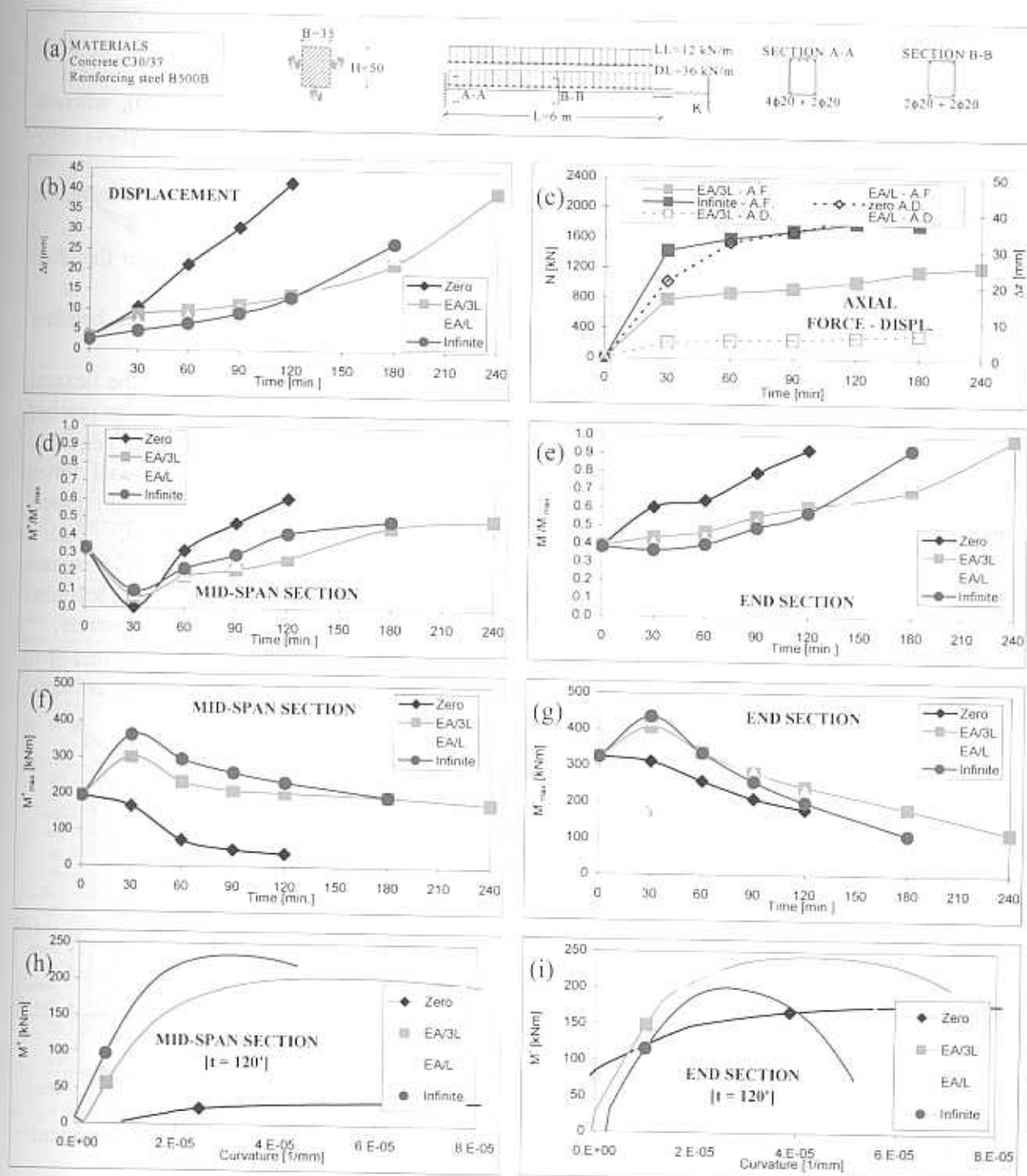


Fig.4-7: Effect of the axial restraint stiffness on the behaviour of a 6m span rectangular beam

- the axial force increases rapidly in the first 30' of fire duration. Later, its increase is much less significant due to concrete damage. From 30' to 180' the axial-force increase is only 6% of the increase in the first 30'. The axial force tends to move upward with respect to beam axis because of the progression of concrete damage under fire;
- the axial displacement increases rapidly during the first 30' of fire duration. Later it remains almost constant, with rather small values (close to 2.9mm);
- the beam deflection is only marginally affected by second-order effects, while the collapse is anticipated (180' vs. 240' by considering only the first-order effects);
- the collapse occurs when end-section capacities are reached, because of excessive concrete damage.

Fig. 4-7 illustrates the influence that the axial restraint has on beam response. Some comments:

- the axially-unrestrained beams exhibit much larger deflections (Fig. 4-7 b), whereas axially-restrained beams exhibit similar displacements, with a few differences only during the first 30', due to the rapid increase of the axial force (Fig. 4-7 c);
- in all cases, the axial force stabilizes after the first 30' and the axial restraint increases beam resistance (R) to fire exposure;
- the axial displacement in unrestrained beams is one order of magnitude larger than that of axially-restrained beams (Fig. 4-7 c);
- Figs. 4-7 d,e show that end sections reach their flexural capacity when the bending moment of the mid-span section is still far from the flexural capacity (50%);
- Fig. 4-7 f shows that, with the exception of the unrestrained beams (where the flexural capacity tends to zero after 120'), the flexural capacity of the mid-span section reaches its maximum value after 30', after which it tends to an almost constant value, equal to the value at ambient temperature. This result is a consequence of the axial force: because of concrete damage, the axial force becomes increasingly eccentric with respect to beam axis and produces a favourable positive bending moment in the beam;
- Fig. 4-7 g shows that in the end sections the bending capacity initially increases (because of the favourable effects of the axial force acting on the undamaged sections) and then progressively decreases, because of the damage in the compressed concrete;
- finally, Figs. 4-7 h,i show the moment-curvature relations after 120' for the mid-span and end sections, respectively. Only the unrestrained beams are close to collapse, whereas the restrained beams still have a considerable safety margin.

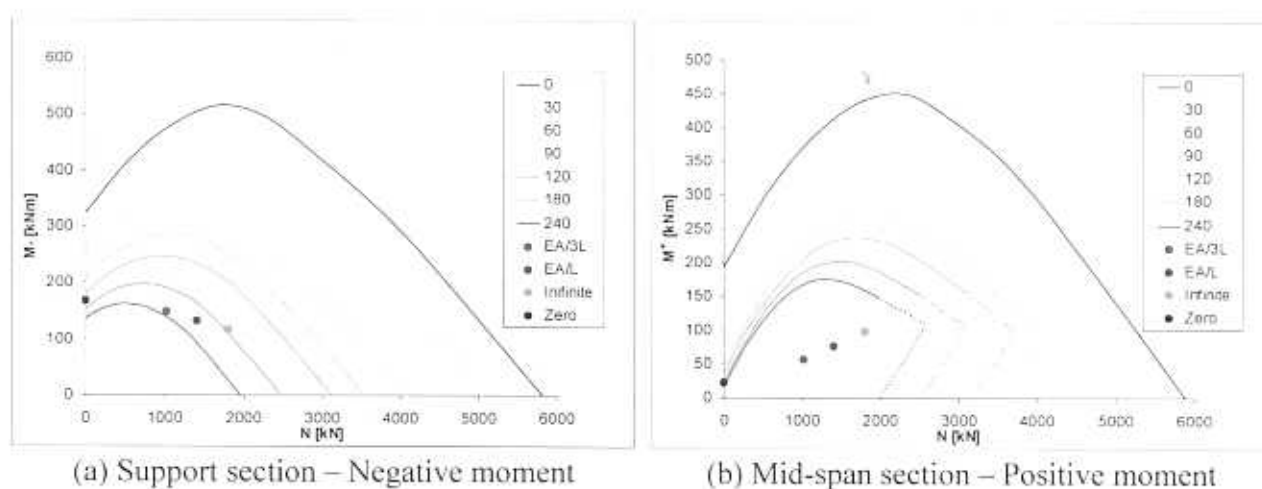


Fig. 4-8: Bending moment – axial force interaction curves

Figs. 4-8a,b show the bending moment-axial force (M-N) interaction curves for end and mid-span sections. In the same figure, the bullets show the values of M and N at 120' for the beams investigated in the parametric analysis. The results clearly demonstrate that the critical sections are always those over the supports.

With reference to the mid-span section (Fig. 4-8 b), the greater the fire exposure, the more the shape of the interaction curves tends to become similar to that of T beams, where the axial capacity is reached for bending-moment values larger than zero, depending on the section shape. For rectangular sections in fire conditions, this behaviour is due to the concrete damage, that progresses from the outer layers toward the centre and upper part of the section.

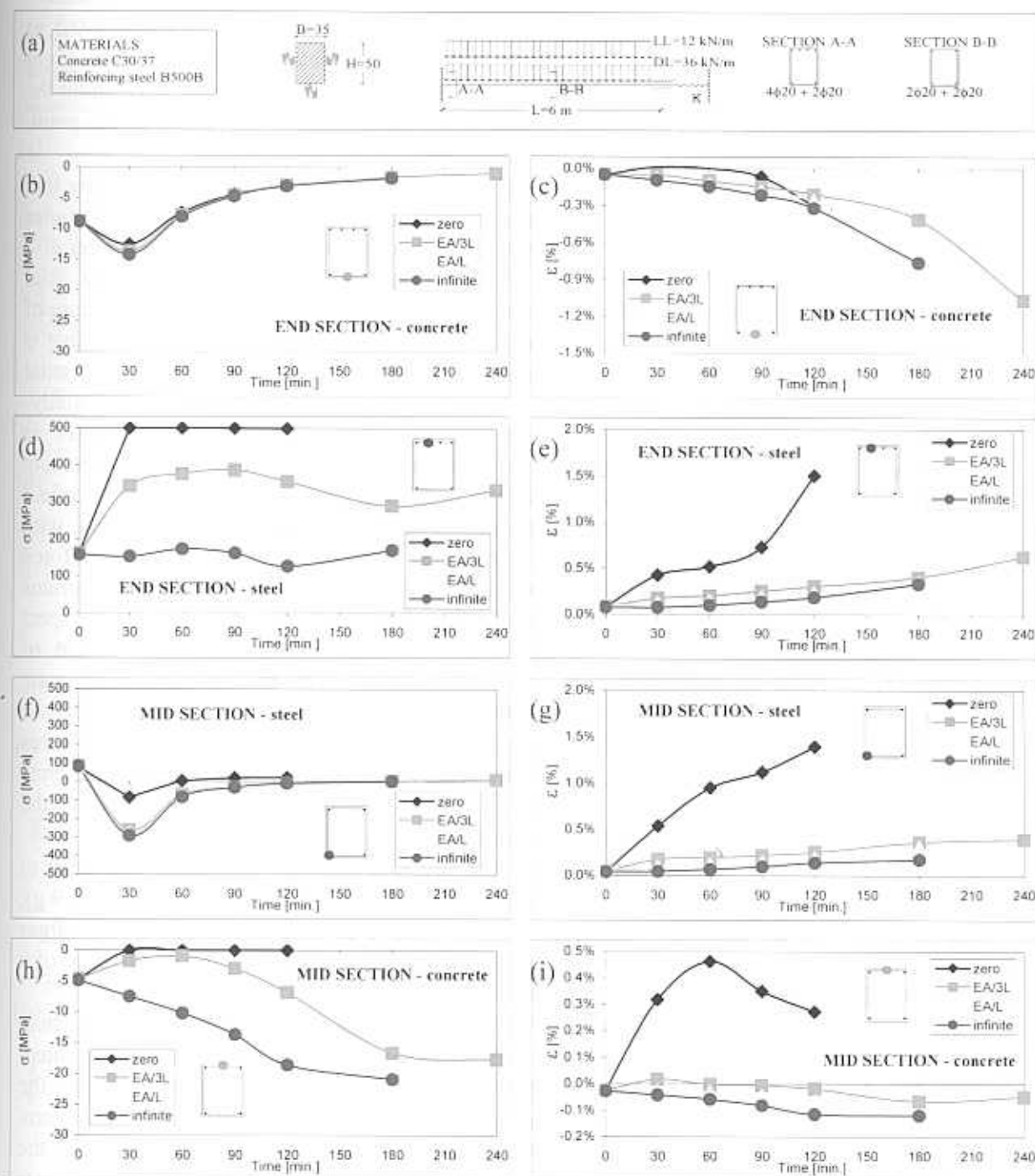


Fig. 4-9: Stress and strain time-histories with different values of the axial restraint stiffness

Fig. 4-9 illustrates the effects that the different restraint stiffnesses have on the stress and strain time-histories of a few significant points of the end and mid-span sections. Some comments:

- The maximum stress in the bottom fibres of the end sections is reached after 30', when concrete is nearly undamaged. Up to this fire exposure, no relevant differences are observed between the axially-restrained and -unrestrained beams. Later, fire-induced concrete damage leads to a progressive decrease of the stress up to collapse, while the total strain keeps increasing up to values approximately equal to 1% (Figs. 4-9 b, c). This behaviour demonstrates that the effective section depth progressively decreases under fire conditions;
- The stress and strain time-histories of the end-section top reinforcement (Figs. 4-9 d, e) demonstrate that after 30' the reinforcing steel of the unrestrained sections reaches the yield plateau, whereas in axially-restrained beams the steel remains linear elastic throughout the whole load history, due to the beneficial effects of the axial force. With regard to this point, even a small degree of restraint is sufficient to prevent the top bars from yielding;
- The stress-time histories of the bottom reinforcement at mid-span (Fig. 4-9 f) show that initial load-induced tensile stress quickly decreases and then reverts to compression (after less than 15' of fire exposure), the maximum compressive stress being reached after 30'. This behaviour is due to the combined effects of the rotational restraint, of the axial restraint and of the Navier-Bernoulli hypothesis (plane sections remain plane). Because of the rotational end-restraint, the thermal field induces a negative bending moment, that is constant all along the beam, with compressive stresses in the bottom fibres. The axial restraint is responsible for further compressive stresses, that are zero in axially-unrestrained beams (Fig. 4-9 f). Furthermore, the Navier-Bernoulli hypothesis is responsible of a self-equilibrated stress distribution in the beam sections, with compressive stresses in the bottom fibres and tensile stresses in the top fibres. In fact, the thermal field induces in the sections a highly nonlinear strain distribution, that is not compatible with the plane-section hypothesis. Hence, mechanical strains are required to enforce compatibility in the section, or, in other words, to have a linear distribution of the strains in the section. (i.e. total strain linearly distributed over the section) and to respect equilibrium of internal forces. It is interesting to observe that the enforcement of compatibility leads to compression in the bottom fibres even in statically-determinate beams, where the bending moment is constant throughout the loading history. Finally, the maximum value of the compatibility-induced compressive stress in the bottom fibres occurs at a fire duration close to 30', when the concrete is hardly damaged. Later, concrete damage considerably increases in the bottom part of the section, and consequently the mechanical strains required to enforce compatibility yield much lower stresses;
- In all beams, after the first 30' the compressive stress in the steel decreases, and tends to zero the more the closer to collapse. Concerning the steel-strain time histories (Fig. 4-9 g), the total steel strain is always positive (i.e. the steel is always stretched). Much larger strain values are found for the axially unrestrained beams, while even the occurrence of a small axial restraint limits steel elongations to a maximum of 0.4%;
- The stress-time histories of the top concrete fibres in the mid-section (Fig. 4-9 h) show that, in axially-unrestrained beams, the negative bending moment due to the thermal gradient and the enforcement of the Navier-Bernoulli hypothesis lead to a reduction of the compressive stresses, to such an extent that after 30' the compressive stress is almost zero. These effects are mitigated by the existence of an axial restraint. For $k=EA/3L$, the compressive stress decreases up to a value almost equal to zero after 60'. For larger values of k , no reduction of the compressive stress is observed, and the maximum stress keeps increasing up to collapse. Concerning the strain-time histories in the same fibre (Fig. 4-9 i), one may observe that the thermal gradient leads to a tensile total strain in the axially-

unrestrained beam, whereas in axially-restrained beams the axial restraint tends to dominate the response, thus leading to increasing values of the initial total compression strain.

The effects that the section shape has on the structural behaviour in fire become evident if Figs. 4-9 and 4-10 (one-way slab section) and Figs. 4-11 and 4-12 (T-section) are compared with Figs. 4-5 and 4-6 (rectangular section).

The comparison of these figures confirms some previous observations for all section types. Some further comments:

- one way slabs: a full bending-moment inversion is observed in the mid-span section; should no top reinforcement be provided, an anticipated collapse of the mid-span section may occur, with the beam behaving like two separated cantilevers;

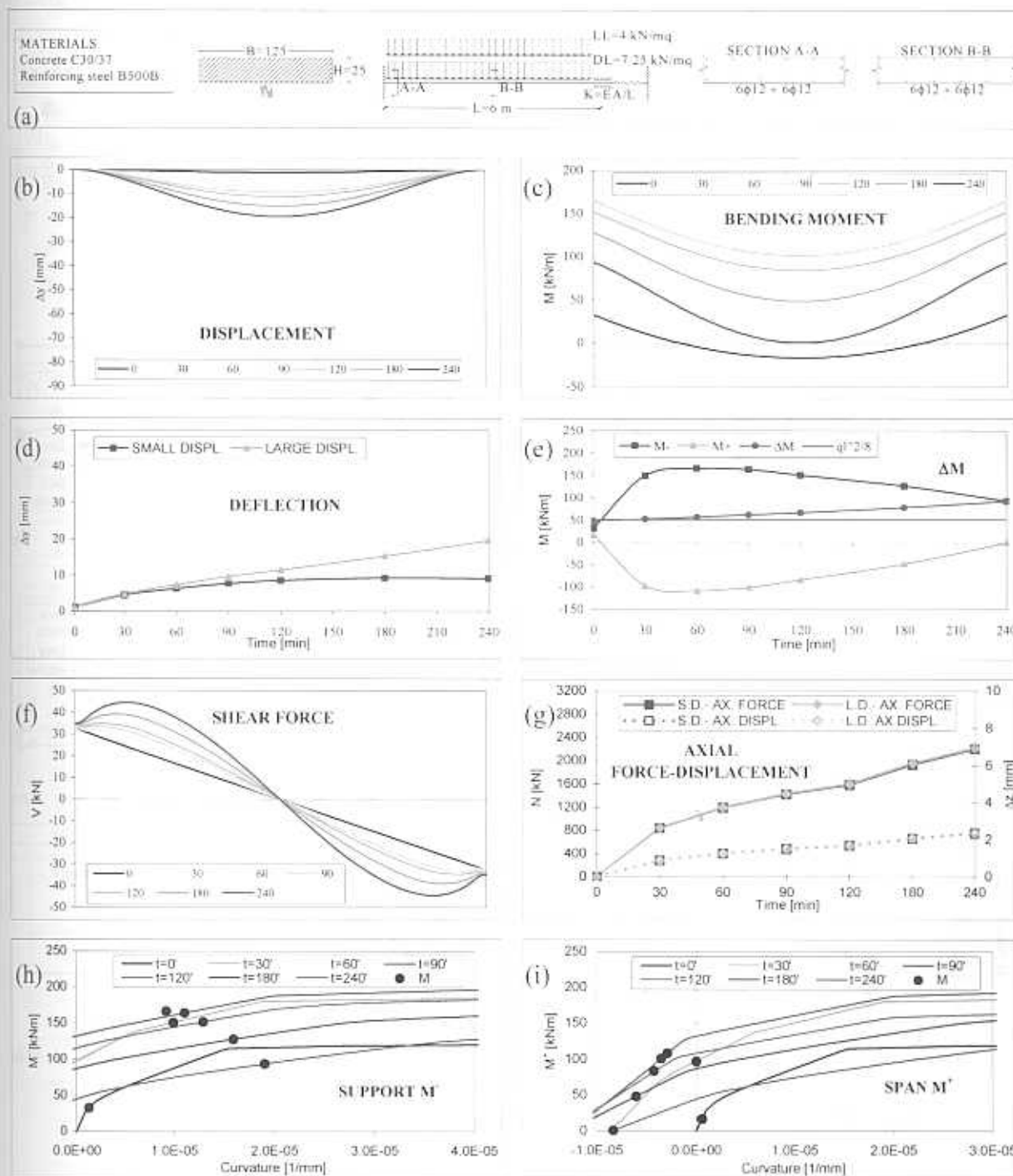


Fig. 4-10: Behaviour of a one-way, 6m-span slab - Axial restraint stiffness $k = EA/L$

- one way slabs: due to second-order effects, the shear force may become higher than that at the end sections, thus leading to a possible anticipated shear failure;
- axially restrained T-beams: these beams exhibit the smallest fire resistance, due to the strength decay of the top slab, which is generally thin.

Finally, the beams with rectangular sections and different span lengths (Figs. 4-6 and 4-13, $L = 6\text{m}$ and 9m) show that the span length has a limited role.

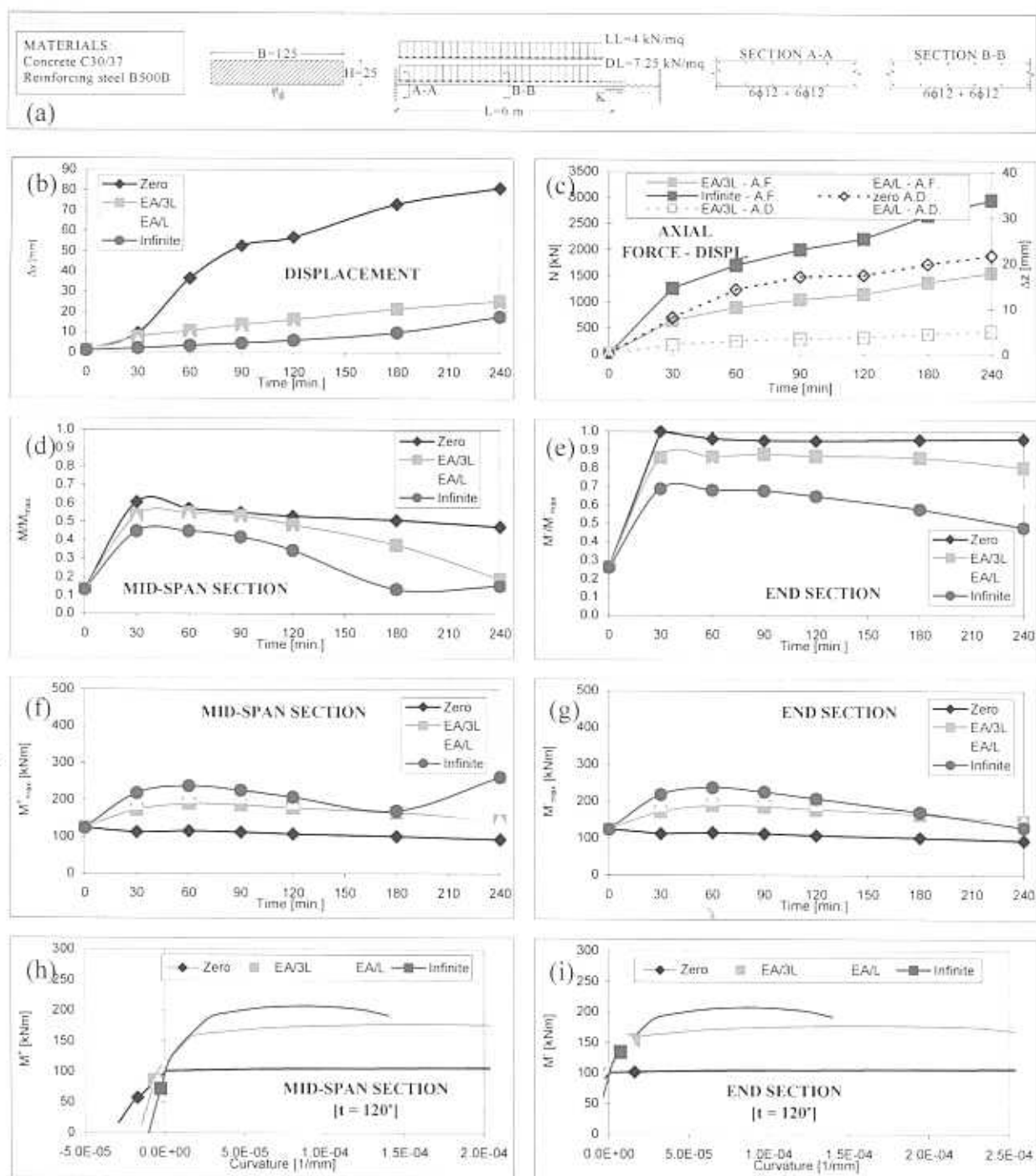


Fig. 4-11: Effect of the axial restraint stiffness on the behaviour of a 6m-span, one-way slab

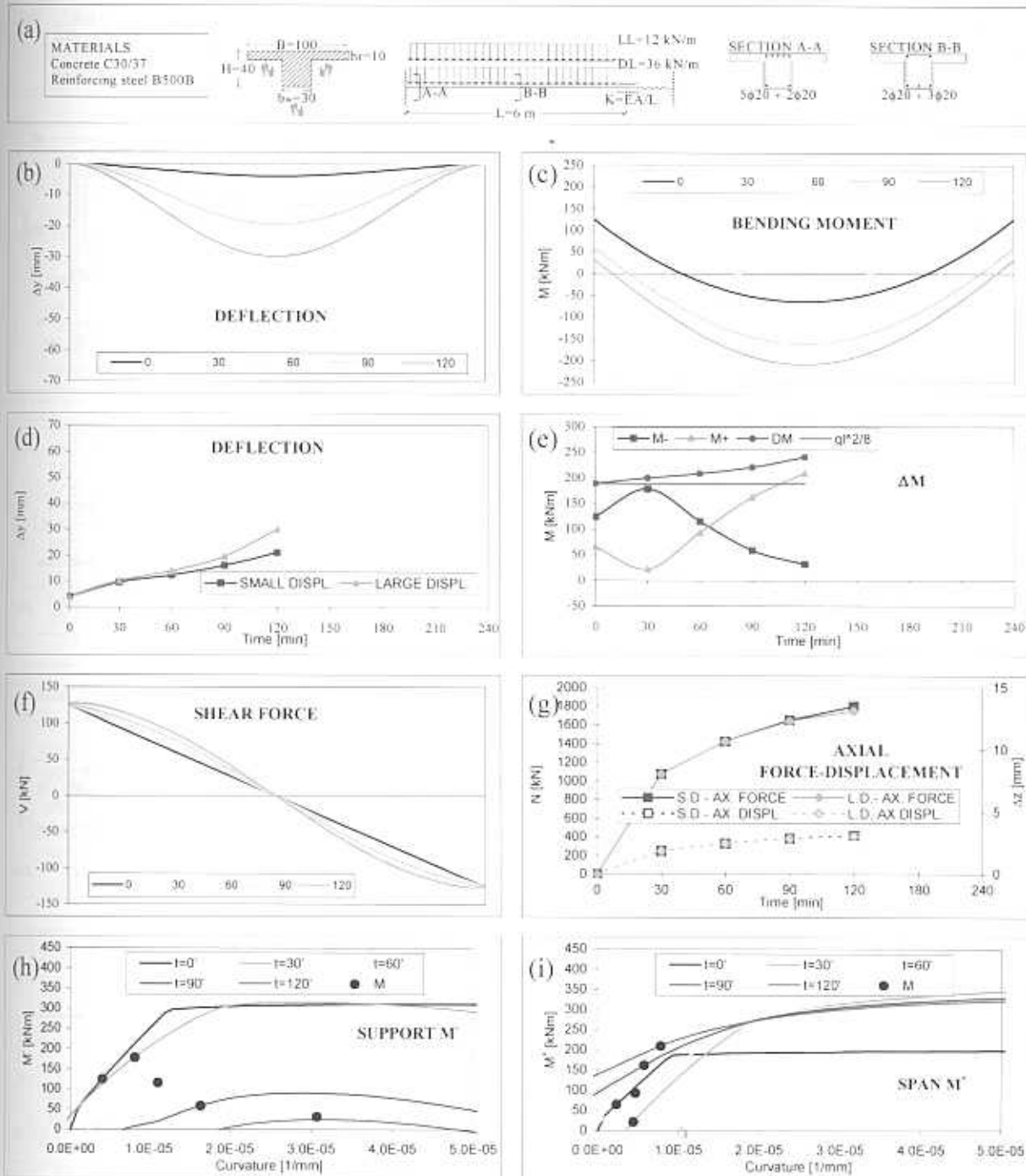


Fig. 4-12: Behaviour of a 6m-span T-beam - Axial-restraint stiffness $k = EA/L$

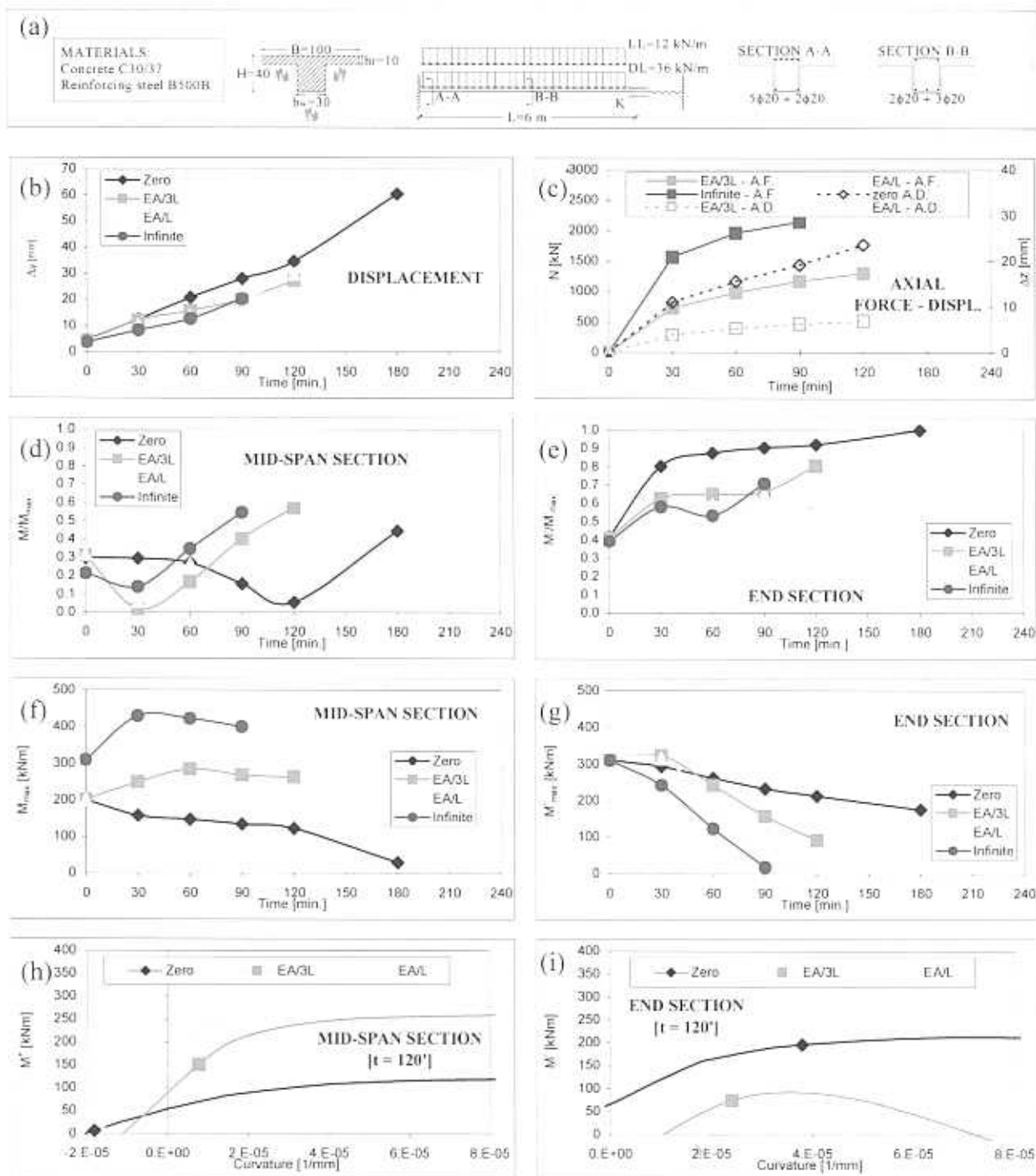


Fig. 4-13: Effect of the axial restraint stiffness on the behaviour of a 6m span T-beam

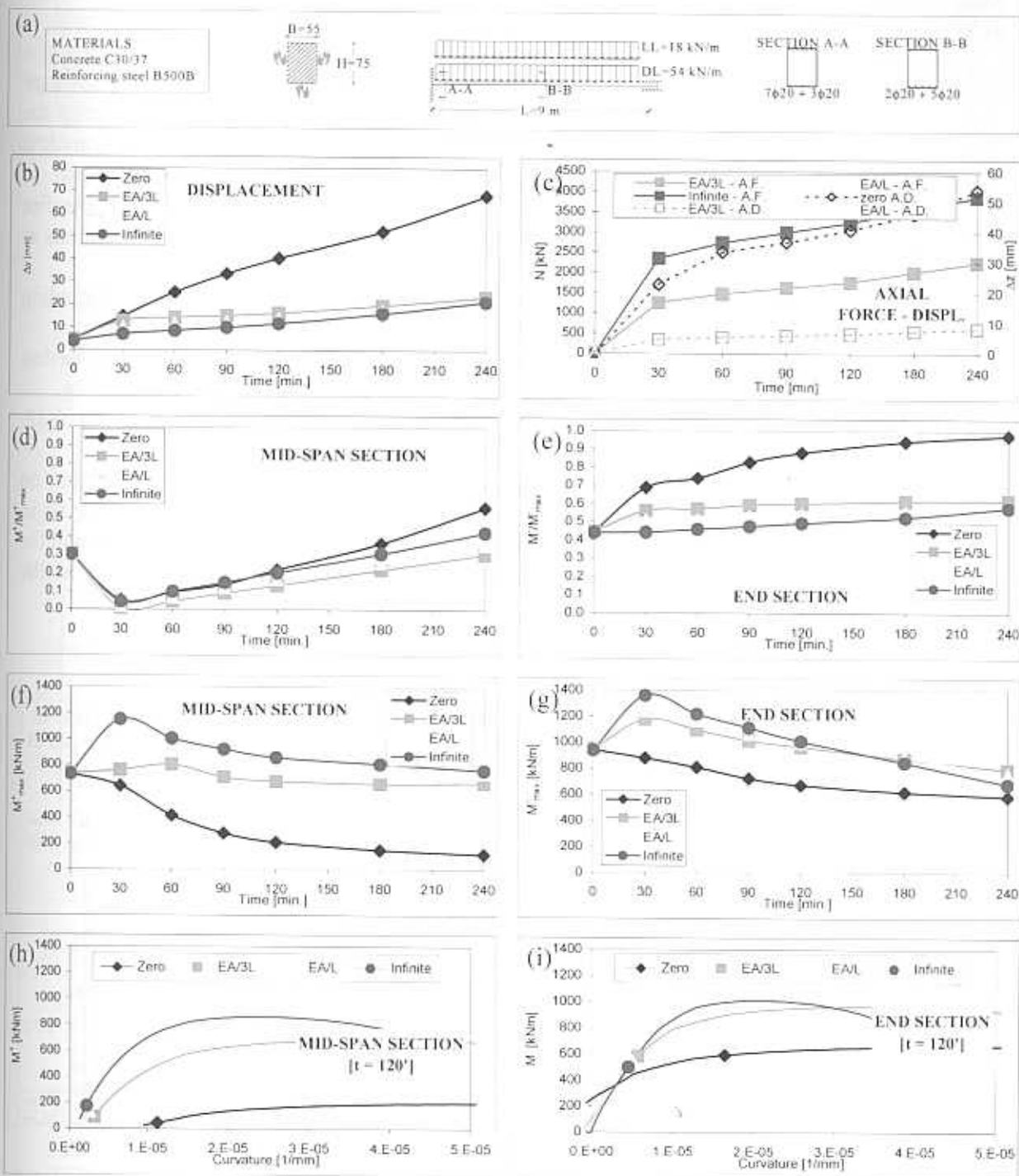


Fig. 4-14: Effect of the axial restraint stiffness on the behaviour of a 9m span-rectangular beam

4.3.2 Parametric study of frames

The behaviour of a set of single-bay 2D fixed-joint frames, belonging to a multi-storey building, has been studied in fire conditions, with the objective of clarifying to what extent the structural response is affected by the section of the beam (rectangular section, T-section, shallow section representing a one-way slab), the span (6m and 9m), and by the fire exposure of the columns. Two different fire exposures were considered for the columns: fire on three sides, with the fourth side at ambient temperature, and fire on one side, with the remaining three sides at ambient temperature. The former case represents a column with the external side

flush with the wall of the compartment in fire, and the latter case a column with the internal side flush with the wall of the compartment.

In the analyses, the existence of the upper floors has been introduced by considering a portion of the columns above the fire compartment, by assuming that the inflection points are located at mid-height between two contiguous storeys, and by applying to the columns the axial forces ($N_{sd} = 100 \text{ kN}$) representing the effects of the upper floors. The dimensionless value adopted for the axial forces acting on the columns is approximately equal to $v_d = N_{sd}/(A_c f_{cd}) \approx 0.30$. The columns belonging to the upper floor, being outside the fire compartment, are assumed to be at ambient temperature. Finally, the boundary conditions at the base of the columns are characterized by full fixity.

The analyses have been performed by considering large displacements, thus including second-order effects. The frames investigated in this study are shown in Fig. 4-15.

The results of the analysis carried out on a frame with a 6 m-span beam having rectangular section and with the column heated on one side are discussed in the following.

The frame geometry and the reinforcement of the critical sections are shown in Fig. 4-15. The fire resistance was 60' (R60), according to the tabular method.

As usual, at first the thermal analysis was carried out, considering the fire as acting on one side in the columns and on three sides in the beam. The results of the thermal analysis in the beam are shown in Fig. 4-5, while the boundary conditions adopted for the column and the results for 3 values of the fire duration are shown in Fig. 4-16. The mesh used in the discretization of the sections was $10 \times 10 \text{ mm}$.

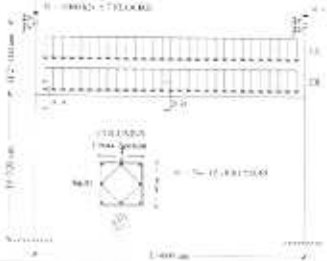
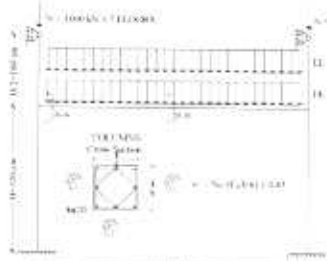
STRUCTURAL MODELS	CROSS SECTIONS		
	RECTANGULAR SECTION	ONE WAY SLAB	RECTANGULAR SECTION + SLAB
MATERIALS: Concrete C30/37 Reinforcing steel B500B REI 60	 B = 35 cm H = 50 cm SECTION A-A 3ø20 + 2ø20 SECTION B-B 2ø20 + 3ø20	 B = 125 cm H = 25 cm SECTION A-A øø12 SECTION B-B øø12 øø12	 B = 100 cm, h = 10 cm H = 40 cm, h _c = 30 cm SECTION A-A 4ø20 + 2ø20 SECTION B-B 2ø20 + 3ø20
LOADS: Rectangular section: DL = 36.45 m, LL = 12.45 m Rectangular section + slab: DL = 36.45 m, LL = 12.45 m One way slab: DL = 12.45 m, LL = 12.45 m	 3.000m x 1.000m	 3.000m x 1.000m	 3.000m x 1.000m
LOADS: Rectangular section: DL = 36.45 m, LL = 12.45 m Rectangular section + slab: DL = 36.45 m, LL = 12.45 m One way slab: DL = 12.45 m, LL = 12.45 m	 3.000m x 1.000m	 3.000m x 1.000m	 3.000m x 1.000m

Fig. 4-15: Parametric study of the frames

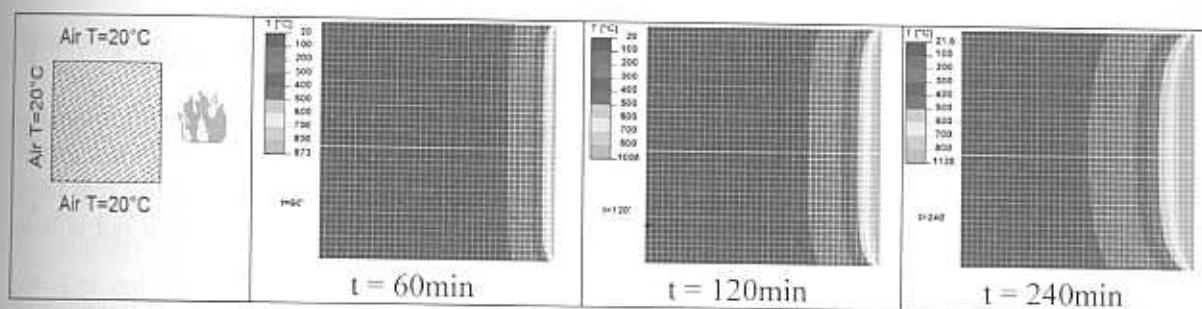


Fig. 4-16: Thermal analysis results for the columns [$^{\circ}\text{C}$]

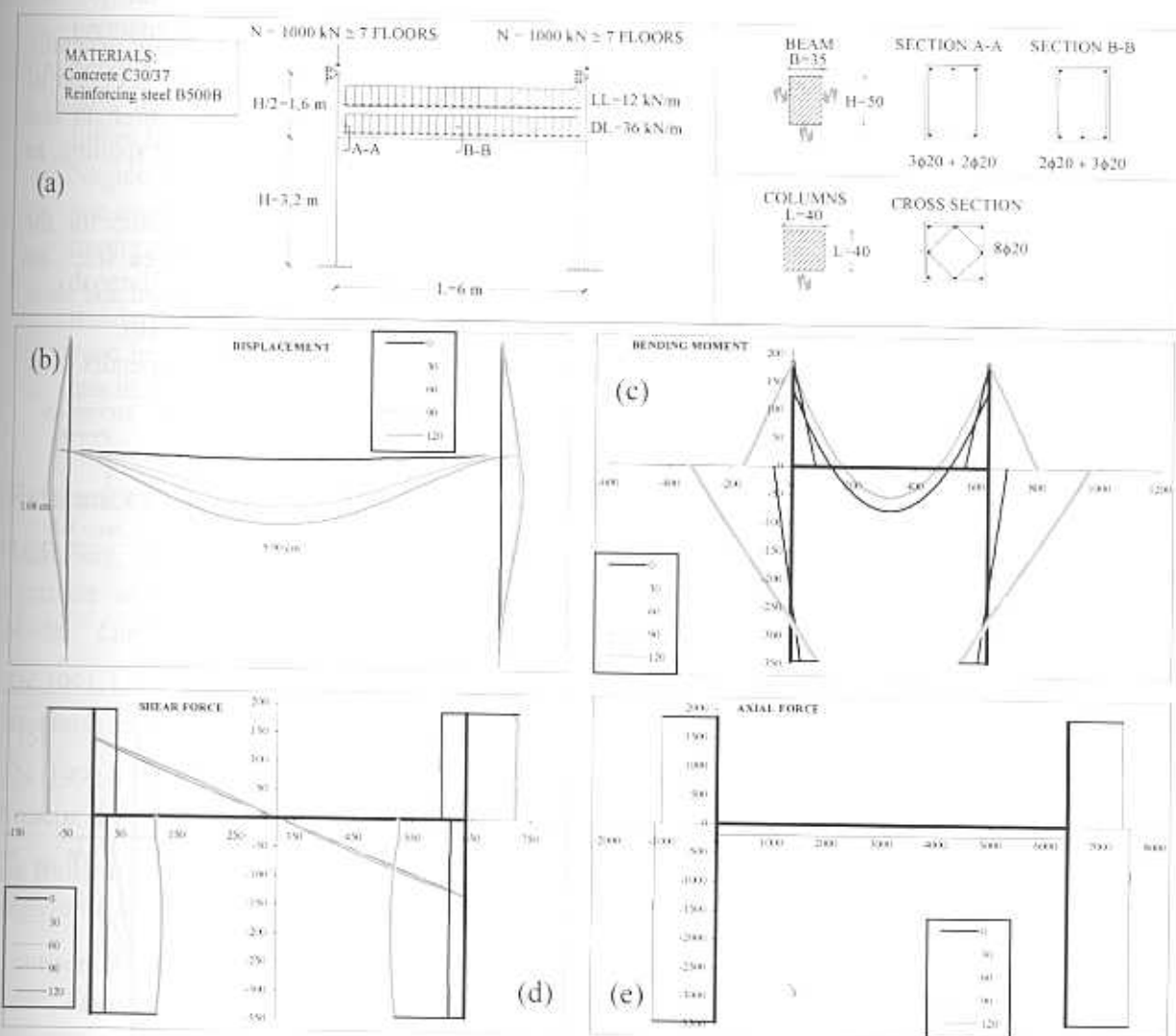


Fig. 4-17: R/C frame with the columns exposed to fire on one side and with a 6m-span beam (rectangular section)

The results of the frame analysis are shown in Fig. 4-17. The following comments can be made:

- Both the bending moment and the shear increase dramatically in the columns in the first 30', because of the heating of the beam; however, no further increase is observed later, because of the progressive damage of the beam, as already discussed in the previous sections;
- The bending moments in the lower columns increase approximately seven times because of the thermal deformations of the beam (elongation and end rotations), while

the bending moments in the upper columns change sign, and their values become more than twice as large as under ambient conditions;

- The shear forces in the lower columns increase approximately four times with respect to ambient conditions; as a result, the columns lightly-reinforced in shear – and under-confined – may exhibit an anticipated shear failure, as often observed during real fires. However, it is fair to observe that the FE model adopted in this study was not able to describe shear failures, because of its inherent limitations;
- As for the beam, no further comments can be added about the different behaviours of the axially-unrestrained and axially-restrained beams.

From a practical point of view, for the columns detailing rules similar to those generally adopted in seismic design seem to be suitable also in fire design. In fact, the adoption of closely-spaced closed stirrups (hoops) is instrumental in improving section strength and ductility in combined bending and axial force, and helps in controlling concrete spalling, as shown by Kodur et al. (2004).

Concerning the remaining beam sections, the same comments already made for the axially-restrained beams still apply. As for the column subjected to fire on three sides, no relevant differences were observed, except that the increase of the bending moment and shear is less pronounced, due to the smaller temperature gradient in the column (Fig. 4-18).

The complete set of results of the parametric study of frames is reported in Appendix 4,

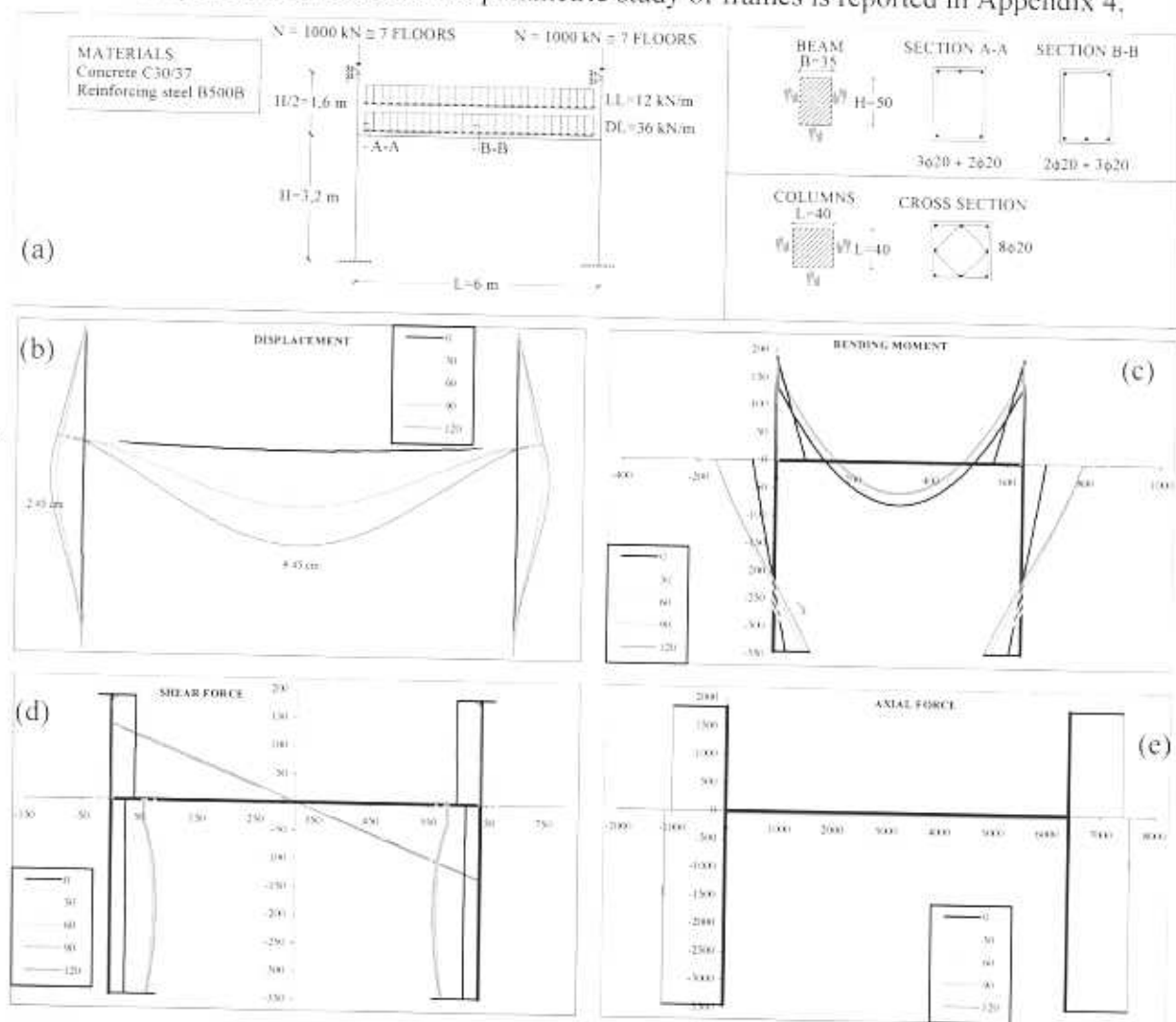


Fig. 4-18: R/C frame with the columns exposed to fire on three sides and with a 6m-span beam (rectangular section)

4.4 Concluding remarks

The following remarks can be drawn from the parametric analysis presented in this study:

- Neglecting the axial force in the design of R/C beams under fire conditions leads to a conservative estimate of the fire resistance.
- The collapse of continuous R/C beams is generally controlled by the support sections, where concrete damage may lead to anticipated section failures;
- Top reinforcement is required in the mid-span section of one-way slabs to avoid a premature collapse of the slab.
- Second-order effects have a marginal influence on R/C members subjected to bending, with the exception of one-way slabs, where second-order effects may induce shear forces larger than those at ambient temperature.
- Neglecting the effects of beam thrust in the design of R/C frames may lead to highly non-conservative results, because of the increasing bending and shear in the columns during the first 30' of fire exposure. However, the thermal effects on bending and shear depend on the type of the foundation adopted in column design, since, for instance, isolated footings provide less rotational stiffness than continuous foundation beams and even less than 2D foundations mats. Consequently, bending and shear in fire increase less in the first case.

References

- Anderberg, Y., and Thelandersson, S. (1976). "Stress and Deformation Characteristics of Concrete at High Temperature - 2. Experimental Investigation and Material Behaviour Model." *Lund Institute of Technology*, August 1976, 84pp.
- EN 1991-1-2 : "Eurocode 1: Actions on structures. Part 1-2 : General Actions – Actions on structures exposed to fire", December 2004.
- EN 1992-1-2: "Eurocode 2: Design of Concrete Structures - Part 1-2: General rules – Structural Fire Design", December 2004.
- fib Bulletin 38. "Fire Design of Concrete Structures – Materials, Structures, and Modelling." *fédération internationale du béton*, 2007, 106pp.
- Franssen, J.-M. (2005). "SAFIR. A Thermal/Structural Program Modelling Structures under Fire", *Engineering Journal, A.I.S.C.*, 42(3), 143-158.
- HKS (2003). "ABAQUS V.6.4 Theory and Users Manuals." Providence, Rhode Island.
- Kodur, V.K.R., McGrath, R., Laroux, P., and Latour, J.C. (2004). "Fire Endurance Experiments on High-Strength Concrete Columns." *National Research Council Canada*, NRC RD Bulletin 138, 147pp.
- Riva, P. (2005). "Nonlinear and Plastic Analysis of RC Concrete Beams." *Proc. Int. Workshop Fire Design of Concrete Structures: What now? What next?*, Milano, 2-4 Dicembre 2004, Starry Link Editore.
- Terro, M.J. (1998). "Numerical Modeling of the Behavior of Concrete Structures." *ACI Structural Journal*, 95(2), 183-193.

5 Plastic analysis of continuous beams*

5.1 Introduction

It is stated in Eurocode 2 (EN 1992-1-2, 2004) that an important question is whether load redistributions between different sections of a member in bending can be accepted in case of fire, these redistributions being allowed by the plastic behaviour of both the reinforcement and the concrete. One of the key conditions for this plastic behaviour is the ductility of the section, i.e. the capacity of the section to keep on developing the plastic bending moment, when the curvature increases to very high values. This seems to be the case according to some numerical examples that show how the ductility of a section tends to increase during a fire. For instance, the moment-curvature diagrams of a 160x400mm rectangular concrete section heated on three sides are plotted in Fig. 5-1. Four curves are presented, namely at time $t = 0'$, 30', 60' and 90' of ISO 834 fire. In each case, a linear (L) and a nonlinear (NL) descending branch has been considered for concrete stress-strain diagram, with hardly any difference between the two; the nonlinear formulation yields a slightly higher ductility than the linear formulation. Note that the ductility increases significantly in a fire situation, as also observed in a 140 mm-thick slab, where ductility increased much less than in beams.

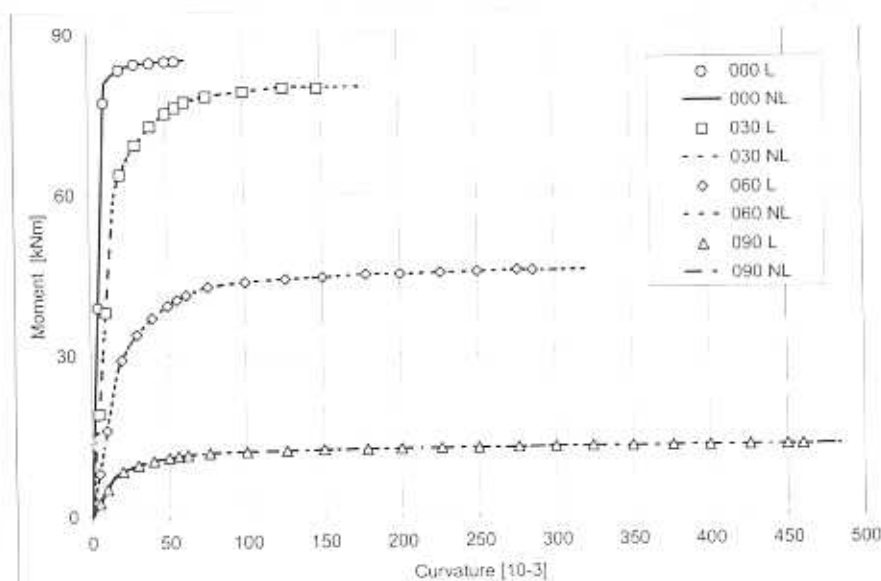


Fig. 5-1: Moment-curvature diagrams of a rectangular section subjected to ISO 834 fire

The main difference between the hot and cold situations is the ratio between the ultimate plastic moment and the first-yielding moment. This ratio is much higher in fire, which means that much higher rotations have to take place before the full plastic moment is reached. This is in no contradiction with what is generally observed during the laboratory tests, where the failure of R/C structures is often accompanied by very large displacements.

The theory of plasticity gives a theoretical validation to the fact that several effects leading to self-equilibrated stress distributions can be neglected in nonlinear numerical analysis. Among these effects, (a) those occurring either in the construction phases or during the service life at room temperature, before the fire starts (due for instance to shrinkage, creep and thermal strains), and (b) those occurring during the fire (due to creep and thermal expansion) should be mentioned. A consequence of neglecting these effects is that the strains,

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stresses and tangent moduli that are computed in any given point of a structure are only approximate – or “mean” – values compared to the “true” values that would be computed if all these effects were taken into account. The computed values are indeed based on the hypothesis of a virgin initial stress distribution, which is far from reality.

Neglecting these self-equilibrated stress distributions is subjected to some limitations, since it is justified as long as the ensuing displacements are small. This is why the effects of thermal expansion during the fire must be taken into account. The thermal strains can indeed reach values up to 1% in steel and up to 1.4 % in concrete, these values being higher or of the same order of magnitude of those occurring at the peak stress, depending on the temperature.

Another strain component that may affect deformations – second-order effects included – in concrete structures submitted to fire is the transient-creep strain. Whether this component has to be taken into account explicitly or implicitly is still a subject of debate. In the stress-strain relationships presented in Eurocode 2, for example, transient-creep strains are incorporated implicitly. Possible reasons why this apparently very simplified model yields reasonable results are:

- the behaviour of a concrete structure is mainly dictated by the behaviour of the steel bars, not by the behaviour of concrete;
- transient creep is not absent from the afore-mentioned simplified model, since it is introduced in an implicit manner;
- an explicit transient-creep model leads to different predictions compared to a simple implicit model only when the material exhibits strain reversals or - more importantly - when the temperature decreases.

These, plus the advantage of utilising a single, widely adopted general model, are probably the reasons why the simple concrete model of Eurocode 2 is so popular among the designers in spite of its many limits.

Nevertheless, it should be noted that the evolution of the strain at the peak stress proposed in the ENV version of Eurocode 2 has been found to be too stiff for representing transient creep. In the revised EN version, the concrete model has been made somehow softer, in order to improve the prediction of the deformations.

5.2 Use of plastic analysis

The objective of plastic analysis is generally to evaluate the load-carrying capacity of a beam. In the following, the fire resistance (R) of the beams analysed in Chapter 3 (concerning the parametric study of continuous beams) is verified by means of Plastic Analysis for a 120' fire duration.

With reference to Fig. 5-2, according to plastic analysis the verification is positive if the ultimate load at the requested fire duration is larger than the applied load, at the onset of beam collapse because of the formation of a suitable number of plastic hinges.

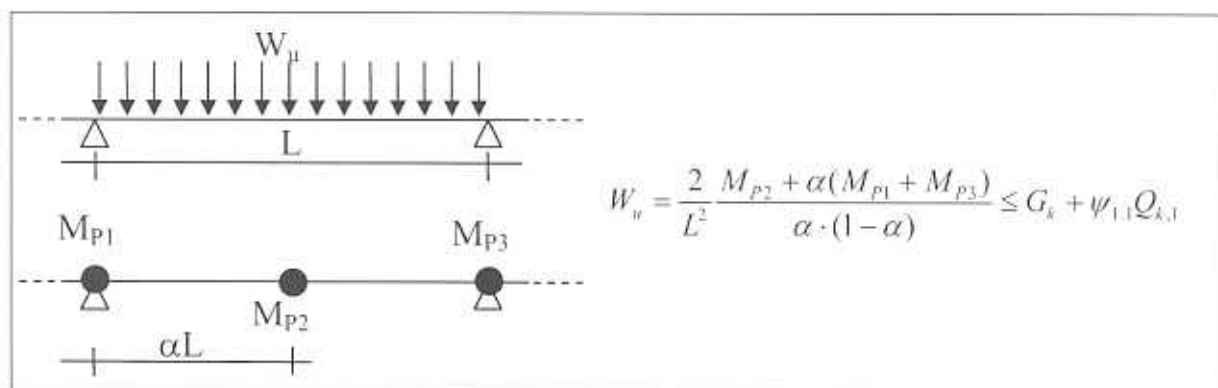


Fig. 5-2: Verification by means of Plastic Analysis

The plastic (e.g. ultimate) moments at critical sections may be determined according to various sectional-analysis method, such as the constant-isothermal method or the zone method, both suggested by EC2.

In axially-unrestrained beams, the application of plastic analysis is straightforward and generally leads to a conservative estimate of the ultimate load.

In case of axially-restrained beams, the ultimate bending-moment value depends on the axial force developed during the fire. Accordingly, the axial-force value has to be estimated prior to sectional analysis, based on fire effects and on the actual axial restraint of the beam.

An approximate estimate of the axial force, to be adopted in plastic analysis, can be performed via the simplified procedure outlined below and shown in Fig. 5-3 (Riva, 2005):

- based on the results of the thermal analysis after a time t of exposure to the fire, the average temperature distribution at each level along the section is determined as shown in Fig. 5-3a and the average temperature in the section is found, as shown in Fig. 5-3b;
- the axial force ensuing from the restrained thermal elongation under a constant temperature distribution is evaluated by computing the normal stress σ_{th} arising in a beam of stiffness $K_{beam} = (E_{\Delta T,ave} A)/l$, axially restrained by a spring of stiffness $K = k(E_{C,20^\circ C} A)/l$, as a consequence of an average thermal elongation $\epsilon_{T,Ave}$, and multiplying such a stress by $0.30A$, A being the cross-section area of the beam (Figs. 5-3c and d). The value of the axial force determined in this way has been checked against the results of the nonlinear parametric analysis (Chapter 3) and has been found to be - in most cases - acceptable for design purposes.
- the plastic (i.e. ultimate) moments of the critical sections are determined by means of either the $500^\circ C$ -isotherm method or the zone method (CEN 2005, Anderberg and Thelandersson 1976), considering also the axial force.

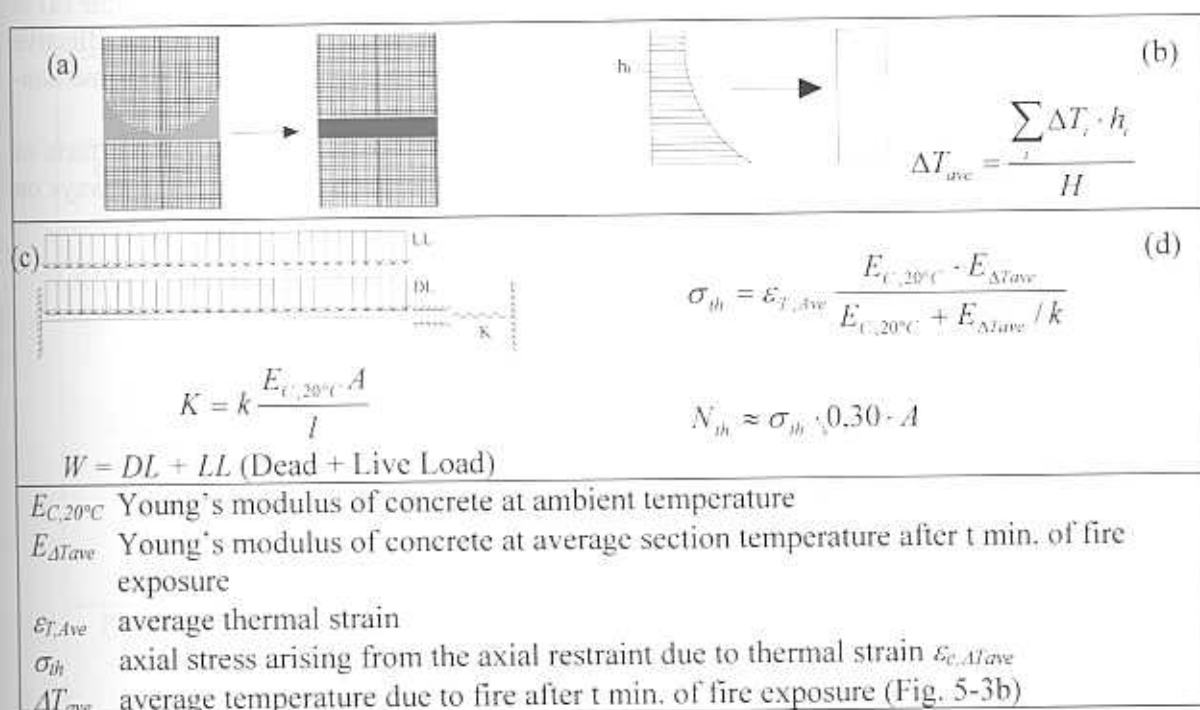


Fig. 5-3: Evaluation of the axial force in axially-restrained beams

The results of the plastic analysis verification after 120' of exposure to a standard ISO 834 fire for the same set of single-span, clamped-end beams analysed in the parametric study is presented in Tables 1 to 6, where reference is made also to the results obtained by means of nonlinear analysis (Chapter 4). In the plastic analysis, the section capacity has been determined by means of the 500°C isothermal method. On the basis of the results shown in the afore-mentioned tables, the following comments can be made:

- in the case of no axial restraint ($K=0$), the ultimate bending capacity of the critical sections at any given fire duration is generally underestimated. As a result, plastic analysis underestimates the ultimate load-carrying capacity of a beam, thus leading to conservative results;
- the assumed axial force in axially-restrained beams results in a lower-bound estimate of the actual force (as given by nonlinear analysis). Hence, the proposed method underestimates the effects of the axial restraint;
- in most cases, the ultimate bending moment of fully axially-restrained beams is overestimated, particularly close to the end sections. As a result, plastic analysis leads - in most cases - to a non-conservative estimate of the ultimate load-carrying capacity. In the case of the fully-restrained 6m-span T-beam, plastic analysis leads to the conclusion that, after a 120' fire duration, the beam is still able to carry a load equal to 90.3 kN/m, while the collapse occurs earlier on the basis of nonlinear analysis.
- for partially-restrained beams, plastic analysis leads in most cases to acceptable results, i.e. to conservative results or to slightly non-conservative results.

Though the results of plastic analysis demonstrate that the safety of a beam at a given fire duration can be assessed rather easily also in axially-restrained beams, one should observe that the proposed method is affected by some rather crude approximations, for example (a) in the estimate of the axial force due to the axial restraint, and (b) in the choice of the effective section (enveloped by the 500°C-isothermal line). These assumptions may lead to some non-conservative results.

However, it is observed that in plastic analysis, by completely neglecting the effects of the axial restraint, the estimate of the ultimate load-bearing capacity of a beam is always on the safe side.

Table 5-1: Results of the plastic-analysis for a 6m beam with rectangular section

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 383.9^{\circ}\text{C}$							
B [mm]	H [mm]	$E_{c,20^{\circ}\text{C}}$ [MPa]	A_{s1} [mm ²]	T_{As1} [°C]	A_{s2} [mm ²]	T_{As2} [°C]	
350	500	18 000	628	492.0	628	108.7	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	A_{sinf} [mm ²]	TA_{sinf} [°C]		$\epsilon_{T,Ave}$	
295	445	5 982.7	628	746.9		4.58E-03	

Mid-Span: $A_{s2} = 0$

Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)										
Restraint	σ [MPa]	N_{th} [kN]	N_{an} [kN]	Error [%]	M_{pl} [kNm]	M_{an} [kNm]	Error [%]	M_{pl} [kNm]	M_{an} [kNm]	Error [%]
$K=0$	0.0	0.0	0.0	0.0	14.4	36.6	-60.5	186.4	180.6	3.2
$K=EA/3L$	13.71	719.7	1021.0	-29.5	185.8	203.6	-8.8	298.4	244.1	22.2
$K=EA/L$	20.55	1078.7	1406.0	-23.3	238.4	229.1	4.0	286.1	231.5	23.6
$K=\infty$	27.38	1437.2	1803.0	-20.3	254.0	234.6	8.3	247.7	201.2	23.1

Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 42 \text{ kNm}\}$			
$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 44.6 \text{ kNm}$	$W_u = 107.6 \text{ kNm}$	$W_u = 116.6 \text{ kNm}$	$W_u = 111.5 \text{ kNm}$
$W_{u,NLA} = 48.3 \text{ kNm}$	$W_{u,NLA} = 99.5 \text{ kNm}$	$W_{u,NLA} = 102.3 \text{ kNm}$	$W_{u,NLA} = 96.8 \text{ kNm}$

Table 5-2: Plastic analysis data and results for the 6m span T-beam

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 442.5^{\circ}\text{C}$							
B [mm]	H [mm]	$E_{c,20^{\circ}\text{C}}$ [MPa]	A_{ssup} [mm ²]	T_{Assup} [°C]	A_{sinf} [mm ²]	T_{As1inf} [°C]	
1000	400	18 000	1570	95.0	628	690.0	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	A_{s2inf} [mm ²]	TA_{s2inf} [°C]		$\epsilon_{T,Ave}$	
1000	349	4 385.7	314	620.0		5.80E-03	

End Sections:
 $A_{ssup} = 1570\text{mm}^2$
 $A_{sinf} = 628\text{mm}^2$
 Mid-Span:
 $A_{ssup} = 628\text{mm}^2$
 $A_{sinf} = 942\text{mm}^2$

Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)										
Restraint	σ [MPa]	N_{th} [kN]	N_{an} [kN]	Error [%]	M_{pl} [kNm]	M_{an} [kNm]	Error [%]	M_{pl} [kNm]	M_{an} [kNm]	Error [%]
$K=0$	0.0	0.0	0.0	0.0	46.6	54.3	-14.2	201.5	212.8	-5.3
$K=EA/3L$	14.7	837.0	1295.0	-35.4	210.1	241.8	-13.1	164.0	111.3	47.4
$K=EA/L$	20.4	1164.9	1753.0	-33.5	266.8	300.2	-11.1	129.3	53.9	139.8
$K=\infty$	25.4	1448.8	-	-	312.9	-	-	93.7	-	-

Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 42 \text{ kNm}\}$			
$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 55.1 \text{ kNm}$	$W_u = 83.1 \text{ kNm}$	$W_u = 88.0 \text{ kNm}$	$W_u = 90.4 \text{ kNm}$
$W_{u,NLA} = 59.4 \text{ kNm}$	$W_{u,NLA} = 78.5 \text{ kNm}$	$W_{u,NLA} = 78.7 \text{ kNm}$	$W_{u,NLA} = \text{Collapsed}$

Table 5-3: Results of the plastic analysis for a 6m one-way slab

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 229.0^{\circ}\text{C}$							
B [mm]	H [mm]	$E_{c,20^{\circ}\text{C}}$ [MPa]	$A_{ss,end}$ [mm ²]	$A_{sl,end}$ [mm ²]	T_{As1} [°C]	T_{Assup} [°C]	
1250	250	18 000	678	6784	463.0	29.0	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	$A_{ss,mid}$ [mm ²]	$A_{sl,mid}$ [mm ²]		$\epsilon_{T,Ave}$	
1250	220	10 222.0	678	678		2.16E-03	

End Sections

$A_{s,sup} = 678 \text{ mm}^2$
 $A_{s,inf} = 678 \text{ mm}^2$

Mid-Span

$A_{s,sup} = 678 \text{ mm}^2$
 $A_{s,inf} = 678 \text{ mm}^2$

Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)											
Restraint	σ	N_{th}	N_{an}	Error	M_{pl}	M_{an}	Error	M_{pl}	M_{an}	Error	
	[MPa]	[kN]	[kN]	[%]	[kNm]	[kNm]	[%]	[kNm]	[kNm]	[%]	
$K=0$	0.0	0.0	0.0	0.0	48.5	70.8	-31.5	59.9	107.6	-44.3	
$K=EA/3L$	8.2	764.4	1148.0	-33.4	143.7	179.7	-20.1	159.6	178.1	-10.4	
$K=EA/L$	14.1	1318.2	1610.0	-18.1	182.6	215.8	-15.4	199.7	195.3	2.2	
$K=\infty$	22.0	2066.7	2222.0	-7.0	213.4	252.2	-15.4	228.0	208.9	9.1	

Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 11.56 \text{ kNm}\}$

$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 24.1 \text{ kNm}$	$W_u = 67.4 \text{ kNm}$	$W_u = 85.0 \text{ kNm}$	$W_u = 98.1 \text{ kNm}$
$W_{u,NLA} = 39.6 \text{ kNm}$	$W_{u,NLA} = 79.5 \text{ kNm}$	$W_{u,NLA} = 91.4 \text{ kNm}$	$W_{u,NLA} = 102.5 \text{ kNm}$

Table 5-4: Results of the plastic analysis for a 9m beam with rectangular section

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 250.2^{\circ}\text{C}$							
B [mm]	H [mm]	$E_{c,20^{\circ}\text{C}}$ [MPa]	A_{s1} [mm ²]	T_{As1} [°C]	A_{s2} [mm ²]	T_{As2} [°C]	
550	700	18 000	628	463.0	1570	98.0	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	A_{sinf} [mm ²]	T_{Asinf} [°C]		$\epsilon_{T,Ave}$	
494	698	9 591.7	628/942	702.9/472.0		2.43E-03	

Mid-Span: $A_{s2} = 0$
 $A_{sinf} = 1570 \text{ mm}^2$

Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)											
Restraint	σ	N_{th}	N_{an}	Error	M_{pl}	M_{an}	Error	M_{pl}	M_{an}	Error	
	[MPa]	[kN]	[kN]	[%]	[kNm]	[kNm]	[%]	[kNm]	[kNm]	[%]	
$K=0$	0.0	0.0	0.0	0.0	282.4	209.2	35.0	606.3	673.6	-10.0	
$K=EA/3L$	9.0	1110.7	1762.0	-37.0	539.2	677.4	-20.4	892.9	1688.8	-47.1	
$K=EA/L$	15.2	1883.0	2418.0	-22.1	726.7	782.3	-7.1	1039.0	1314.7	-21.0	
$K=\infty$	23.3	2886.4	3235.0	-10.8	908.7	867.4	4.8	1164.0	1234.2	-5.7	

Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 63 \text{ kNm}\}$

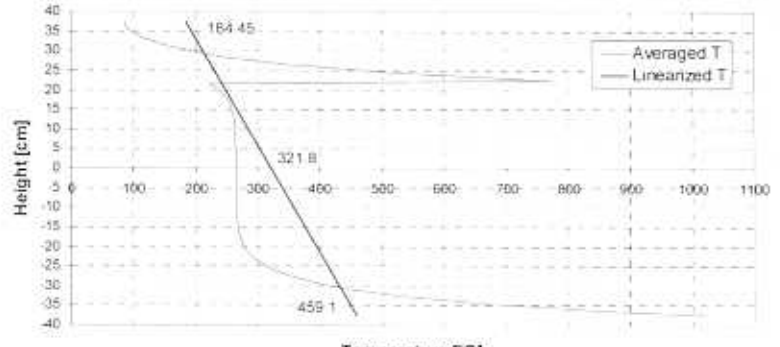
$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 80.4 \text{ kNm}$	$W_u = 123.7 \text{ kNm}$	$W_u = 172.6 \text{ kNm}$	$W_u = 221.3 \text{ kNm}$
$W_{u,NLA} = 87.2 \text{ kNm}$	$W_{u,NLA} = 233.7 \text{ kNm}$	$W_{u,NLA} = 207.1 \text{ kNm}$	$W_{u,NLA} = 207.6 \text{ kNm}$

Table 5-5: Results of the plastic analysis for a 9m T-beam

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 321.8^\circ\text{C}$							
B [mm]	H [mm]	$E_{c,20^\circ\text{C}}$ [MPa]	A_{ssup} [mm ²]	T_{Assup} [°C]	A_{sinf} [mm ²]	T_{Asinf} [°C]	
1350	750	18 000	2198	95.0	628	690.0	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	A_{s2inf} [mm ²]	T_{As2inf} [°C]			$\epsilon_{T,Ave}$
1350	693	7 017.0	942	620.0			3.48E-03



End Sections
 $A_{ssup} = 2198\text{mm}^2$
 $A_{sinf} = 942\text{mm}^2$
 Mid-Span:
 $A_{ssup} = 628\text{mm}^2$
 $A_{sinf} = 1570\text{mm}^2$

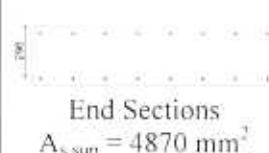


Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)											
Restraint	σ	N_{th}	N_{an}	Error	M_{pl}	M_{an}	Error	M_{pl}	M_{an}	Error	
	[MPa]	[kN]	[kN]	[%]	[kNm]	[kNm]	[%]	[kNm]	[kNm]	[%]	
$K=0$	0.0	0.0	0.0	0.0	212.1	243.0	-12.7	640.5	809.0	-20.8	
$K=EA/3L$	11.3	1495.2	2258.0	-33.8	659.1	979.8	-32.7	675.4	867.4	-22.1	
$K=EA/L$	17.6	2334.0	3127.0	-25.4	887.4	1236.8	-28.3	537.6	752.6	-28.6	
$K=\infty$	24.4	3243.9	4221.0	-23.1	1109.0	1529.3	-27.5	365.1	497.1	-26.6	

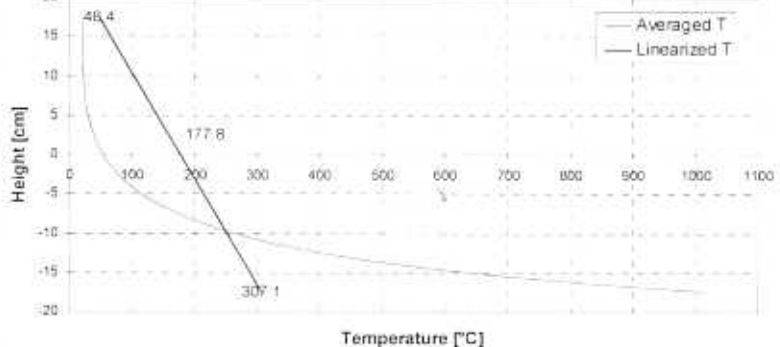
Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 63 \text{ kNm}\}$			
$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 84.2 \text{ kNm}$	$W_u = 131.8 \text{ kNm}$	$W_u = 140.7 \text{ kNm}$	$W_u = 145.6 \text{ kNm}$
$W_{u,NLA} = 103.9 \text{ kNm}$	$W_{u,NLA} = 182.4 \text{ kNm}$	$W_{u,NLA} = 196.5 \text{ kNm}$	$W_{u,NLA} = 200.1 \text{ kNm}$

Table 5-6: Plastic analysis data and results for the 9m span one-way slab

Time $t=120'$ – Average Temperature $\Delta T_{ave} = 177.8^\circ\text{C}$							
B [mm]	H [mm]	$E_{c,20^\circ\text{C}}$ [MPa]	$A_{ss,end}$ [mm ²]	$A_{sl,end}$ [mm ²]	T_{Assup} [°C]	T_{Assup} [°C]	
1400	350	18 000	1608	1206	473.0	23.0	
B_{red} [mm]	H_{red} [mm]	$E_{c,120'}$ [MPa]	$A_{ss,mid}$ [mm ²]	$A_{sl,mid}$ [mm ²]			$\epsilon_{T,Ave}$
1400	296	12 037.7	1206	1206			1.55E-03



End Sections
 $A_{s,sup} = 4870 \text{ mm}^2$
 $A_{s,inf} = 1963 \text{ mm}^2$
 Mid-Span
 $A_{s,sup} = 452 \text{ mm}^2$
 $A_{s,inf} = 3927 \text{ mm}^2$



Plastic Moments at Critical Sections (Anderberg's method vs. Non-Linear Analysis)											
Restraint	σ	N_{th}	N_{an}	Error	M_{Nth}	M_{pl}	M_{an}	Error	M_{pl}	M_{an}	Error
	[MPa]	[kN]	[kN]	[%]	[kNm]	[kNm]	[kNm]	[%]	[kNm]	[kNm]	[%]
$K=0$	0.0	0.0	0.0	0.0	0.0	111.5	120.3	-7.3	222.0	320.5	-30.7
$K=EA/3L$	6.2	911.8	1290.0	-29.3	111.7	349.9	308.0	13.6	454.4	439.7	3.3
$K=EA/L$	11.2	1642.5	1792.0	-8.3	201.2	433.4	373.2	16.1	533.7	473.0	12.8
$K=\infty$	18.6	2741.0	2575.0	6.4	335.8	603.3	465.1	29.7	667.8	516.6	29.3

Plastic Analysis – $\{W_u = (M_{pl}^+ + M_{pl}^-) \cdot 8/L^2 \geq W = 16.45 \text{ kNm}\}$			
$K=0$	$K=EA/3L$	$K=EA/L$	$K=\infty$
$W_u = 32.9 \text{ kNm}$	$W_u = 79.4 \text{ kNm}$	$W_u = 95.5 \text{ kNm}$	$W_u = 125.5 \text{ kNm}$
$W_{u,NLA} = 43.5 \text{ kNm}$	$W_{u,NLA} = 73.8 \text{ kNm}$	$W_{u,NLA} = 83.6 \text{ kNm}$	$W_{u,NLA} = 97.0 \text{ kNm}$

5.3 Conclusions

The results obtained by means of plastic analysis lead to the following concluding remarks:

- Plastic analysis is a simple and straightforward method, that is very sensitive to the evaluation
 - (a) of the plastic moments at the supports, and
 - (b) of the effects of the axial restraints.
- Neglecting the effects of the axial restraints always leads to a conservative estimate of the ultimate load-carrying capacity in statically-redundant beams.

The latter observation lead to the conclusion that designing a continuous beam subjected to fire, using plastic analysis, is on the safe side, if the effects of the axial restraints are ignored.

References

- Anderberg, Y., and Thelandersson, S. (1976). "Stress and Deformation Characteristics of Concrete at High Temperature – 2. Experimental Investigation and Material Behaviour Model." *Lund Institute of Technology*, August 1976, 84pp.
- EN 1992-1-2: "Eurocode 2: Design of Concrete Structures – Part 1-2: General rules – Structural Fire Design", December 2004, 97 pp.
- Riva, P. (2005). "Nonlinear and Plastic Analysis of RC Concrete Beams." *Proc. Int. Workshop Fire Design of Concrete Structures: What now? What next?*, Milano, 2-4 Dicembre 2004, Starry Link Editore.

Notation

A_s	reinforcing steel area
B	cross section width
B_{red}	reduced cross section width, based on 500°C isothermal method
$E_{c,20^\circ C}$	concrete Young's modulus at 20°C
$E_{c,120'}$	concrete Young's modulus after 120' of fire duration
H	cross section height
H_{red}	reduced cross section height, based on 500°C isothermal method
K	axial restraint of beam
M_{pl}	plastic moment resulting from 500°C isothermal method
M_{an}	ultimate moment resulting from parametric study (Chapter 4)
N_{th}	axial force due to thermal elongation evaluated as shown in Fig. 5-2
N_{an}	axial force due to thermal elongation resulting from parametric study (Chapter 4)
T_{As}	reinforcing steel temperature
W	load applied to the beam, sum of dead and live load
W_u	ultimate load resulting from plastic analysis
$W_{u,NL,A}$	ultimate load resulting from nonlinear analysis (Chapter 4)
ΔT_{ave}	average temperature due to fire after t min. of fire exposure
$\epsilon_{T,ave}$	average thermal elongation
σ	normal stress due to thermal elongation and axial restraint

6 Expertise and assessment of materials and structures after fire*

6.1 Residual material characteristics

6.1.1 Introduction

A few recent and dramatic incidents in tunnels (Fig. 6-1) have brought the safety of R/C structures subjected to fire back to the scene, with reference to both ordinary concrete (normal-strength = NSC) and high performance/high-strength concrete (HPC/HSC). In most of the cases temperatures as high as 800-1100°C – and more – were reached (Demorieux and Levy, 1998; Khoury, 2000).

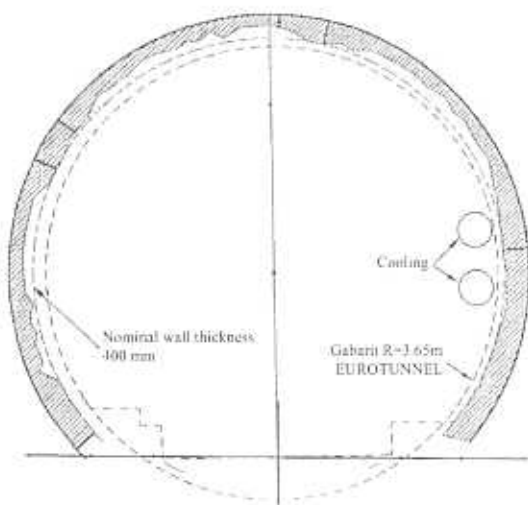


Fig. 6-1: Fire-induced damage in a typical section of the HPC lining of the railway tunnel across the English Channel after the fire of November 1996 (Demorieux and Levy, 1998)

Besides the tunnels, many other structures and infrastructures are at a risk from fire, such as bridges and viaducts, high-rise buildings (FEMA; 2002), covered parkings, off-shore platforms and containment shells (in nuclear power plants and petrochemical plants), not to mention more specific structures like – for instance – airport runways (Hironaka and Malvar, 1998).

In all cases, the load-bearing capacity of the structure in the actual fire conditions, with rapidly-increasing temperatures, is of primary importance for the evacuation of people and things, as well as for the safety of the rescue teams. However, since all fires have a finite duration and in most cases R/C structures do not collapse, the residual capacity past the fire also has to be taken into account and assessed, since tearing down and rebuilding a structure, or strengthening and rehabilitating the damaged members have huge economical implications. To make the proper choice, the knowledge of the residual properties of the various cementitious composites and of the reinforcement is necessary, even more today since innovative materials (like HPC/HSC, HPLWC and UHPC) are increasingly used. This Chapter is divided into two parts. With the first part (6.1) focused on the residual behaviour of concrete-like materials and the second part (6.2) aimed to describe the various non-destructive methods that are available nowadays in order to evaluate the residual strength and – more generally – the level of the damage of the materials in fire-damaged structures.

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6.1.2 Reinforcement

Hot-rolled bars keep most of their mechanical properties up to 400°C, with no strength reduction, but with a continuous and sizeable reduction of the elastic modulus over 100°C, that is accompanied by the disappearance of the yield plateau over 200°C (Fig. 6-2, Harmathy, 1993, see also Buchanan, 2001; Malhotra, 1982 and Takeuchi et al., 1993). At higher temperatures the strength reduction becomes very pronounced, and only 20% of the original strength is left at 650°C. As for cold-drawn bars, wires and strands, the unfavorable effects of high temperature start at lower temperatures, and the strength decrease is close to 50% at 400°C, while less than 10% is left at 650°C (Fig. 6-3, Harmathy, 1993).

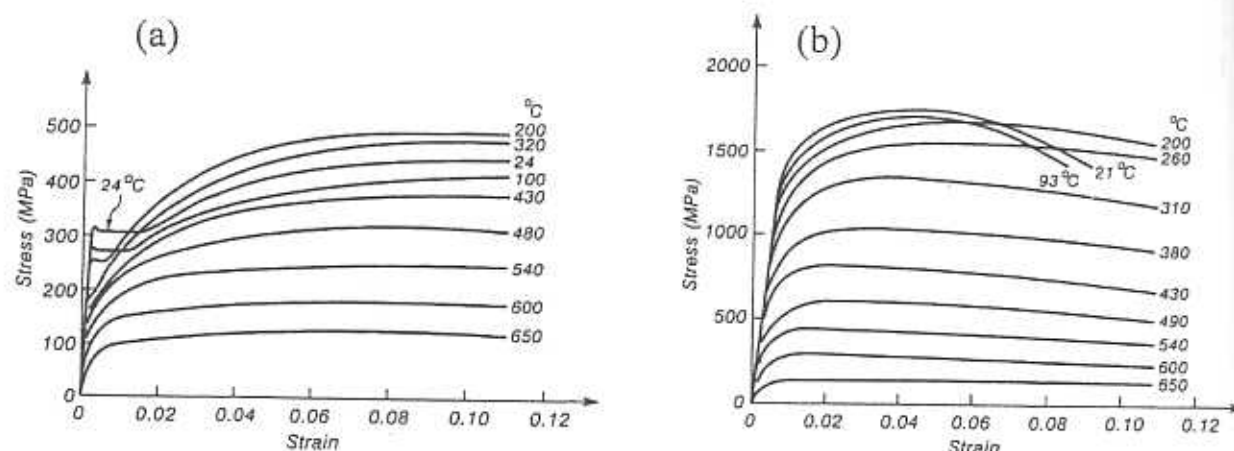


Fig. 6-2: Stress-strain curves for typical hot-rolled steel (a) and prestressing steel (b) at elevated temperature (Harmathy, 1993, see Buchanan, 2001)

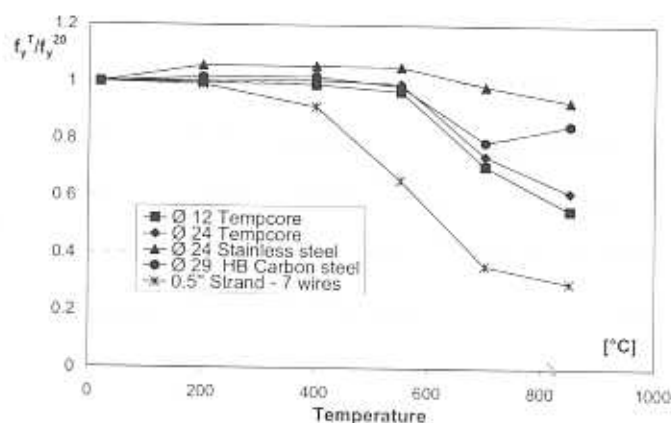


Fig. 6-3: Residual yield strength versus temperature for typical deformed quenched and self-tempered bars ($f_{yk} = 500$ MPa, hot-rolled bars) and for a 0.5" 7-wire strand ($f_{yk} = 1600$ MPa, cold-drawn wires), from Felicetti and Meda (2005). Tempcore = steel, quenched in sprayed water.

Contrary to the hot-state properties, the residual properties of both ordinary and high-strength reinforcement (for R/C and P/C respectively) have received little attention so far, and the not much data on hot-rolled bars are accompanied by a nearly total lack of data on cold-drawn, high strength reinforcement. A possible explanation is that ordinary, hot-rolled bars tend to recover completely after a thermal cycle at 500°C, and lose 20-30% after a cycle at 650-850°C, while the strength decay of cold-drawn steel is so pronounced at high temperature ($T > 550^\circ\text{C}$) that the possible strength recovery after a fire is of no use, since the structure is

too heavily damaged, as a result of the pre-stressing loss during the fire. It is worth noting that hot-rolled bars still keep a well-defined yield plateau even after a high-temperature cycle.

With regard to the residual properties, some data has been recently obtained in Milan from a preliminary project on the residual behavior of a series of typical deformed bars (quenched and self-tempered bars with $\varnothing = 12, 16, 20$ and 24 mm, $f_{yk} = 500$ MPa) and of a typical strand ($1/2$ in., 7 wires, $f_{yk} = 1600$ MPa). All specimens (except those to be tested in the virgin conditions) were heated up to $200, 400, 550, 700$ and 850°C , and rested at the nominal temperature for one hour, prior to being slowly cooled to room temperature. Then all specimens were stored at room temperature for 90-120 days, prior to testing (3 tests for each case). The deformed bars exhibited a sizeable decay above 550°C (Fig. 6-3), with a loss of 40-45% and 30% in terms of yield and ultimate strength respectively, after being heated to 850°C . As expected, the mechanical decay of the strands was definitely more pronounced (up to -70%, Fig. 6-3, Felicetti and Meda, 2005).

6.1.3 Concrete

The residual behavior of ordinary concrete after a high-temperature cycle has been studied extensively in the past, since high-temperature effects were investigated for many years not at high temperature, but after the specimens had cooled down to room temperature, because of the much simpler test modalities required in the latter case.

As well indicated in the still topical RILEM's Report on "Properties of Materials at High Temperatures – Concrete" (edited by Schneider, 1985), the main concrete properties investigated in the past were: (a) residual compressive and tensile strength; (b) residual modulus of elasticity; (c) strength recovery; and (d) residual strain at ultimate, not to cite some other properties that received less attention, like the residual bond strength and the residual thermal diffusivity (which is a combination of thermal conductivity, specific heat and specific mass). However, in the last ten-to-fifteen years the fracture properties of the concrete (both hot and residual) have appeared on the scene, and much work has been done on fracture parameters (see Bazant and Kaplan, 1996 for NSC), with several recent contributions to high-performance and special cementitious composites (Felicetti and Gambarova, 1999; Zhang et al., 2000a,b).

6.1.3.1 Chemo-physical issues

It is well known that most of the damage in the concrete exposed to high temperature comes from the mostly irreversible processes occurring in the heating phase and during the rest period at high temperature. With regard to this point the following factors can be cited: loss of evaporable water in the cement paste (40 - 105°C) and in the aggregates (beyond 200°C), reduction of chemically-combined water in the hydration products (105 - 850°C , with the dehydration and breakdown of the gel structure), crystalline transformation of siliceous aggregates (from α - to β -quartz between 500 and 650°C , with a peak at 575°C and significant expansion), dissociation of calcium hydroxide (into lime and water, at 350 - 500°C) and decarbonation of calcareous aggregates (from calcium carbonate to lime and carbon dioxide at 600 - 900°C). These factors, together with the different thermal expansion of the hardened cement paste and of the aggregates, cause micro-strains and micro-cracking, to the detriment of the integrity of concrete microstructure. Basically the hardened cement paste expands up to 150 - 200°C and then shrinks, while the aggregates expand continuously with the temperature, with a maximum for granite and a minimum for basalt (among the most usual aggregates, see Bazant and Kaplan, 1996). However, the differential strains caused by the thermal

incompatibility of the cement paste and aggregates are, to a large extent, reduced by the "transitional thermal creep" (Khouri et al., 1985; Khouri, 2000), which ensues from the large increase in the rate of creep during the initial transient heating.

Water evaporation in the cement paste and in the aggregates is accompanied and/or caused by moisture migration (liquid and gaseous phases) in the capillary pores of the cement paste, the porosity being in turn increased by the progressive breakdown of the gel structure as dehydration proceeds. It is therefore obvious that moisture content can not only markedly affect concrete physical and mechanical properties, but can also induce some "structural" effects (like thermal spalling, at 250-400°C), which may overshadow concrete constitutive behavior during a test at high temperature.

Thermal spalling, in various explosive and non-explosive forms, is mainly related to the pressure of the steam in the pores, this pressure being in turn a function of temperature, heating rate and time, pore size, as well as of specimen shape and size. Concrete spalling is also favored by the compressive thermal stresses acting in the layers closest to the surface (Fig. 6-4). However, pore pressure is the dominant factor, since the ensuing tensile stresses might have the same order of magnitude of concrete tensile strength, which decreases sharply at high temperature, especially in high-performance siliceous concrete (Felicetti and Gambarova, 1999).

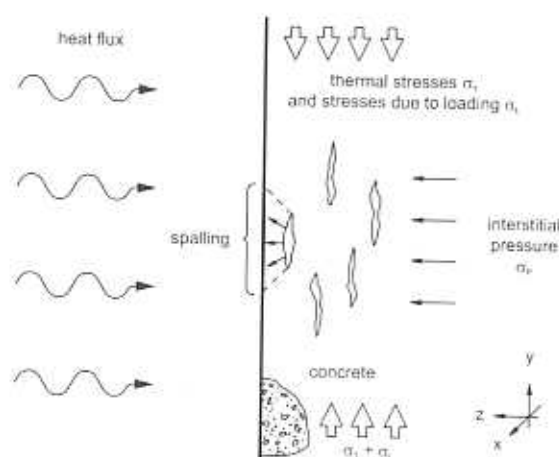


Fig. 6-4: Forces acting in heated concrete (Zhukov, 1975, cited by Khouri, 2000)

Although both normal-strength and high-performance silica-fume composites are spalling-prone, the latter are more sensitive, because of their reduced and disconnected porosity (Jahren, 1989), and of the large content of expansive silica fume in certain ultra high-performance composites.

However, to prevent HPC from spalling, the introduction of polymeric fibers has been proposed, and there is plenty of evidence that even small volumetric contents (0.15–0.30 %) can improve the material's behavior, with marginal or no effects on both the stress-strain relationship and Young's modulus (Hoff et al., 2000).

Compared to the many factors causing concrete damage during the heating process and at high temperature, there are relatively few factors influencing concrete properties during the cooling process, down to room temperature. Since the transitional thermal creep is no longer active, the thermal incompatibility of the cement paste and aggregates creates further damages, this being the major reason why concrete strength after cooling is lower than at high temperature, more in ordinary than in high-performance mixes (RILEM, 1985; Felicetti and Gambarova, 1998, Fig. 6-5). The subsequent, partial rehydration of the cement paste has two conflicting effects, a further strength decrease in the short term (one or two months) because of the formation of calcium hydroxide from lime, with volume expansion, and a partial or

even a total strength recovery (compared to the strength at high temperature) in one-two years, because of the rehydration of the gel and of the unhydrated cement grains (Fig. 6-5). The initial strength loss exhibited by the test specimens also has a "structural" nature, since moisture absorption by the surface layers produce an expansion, which is restrained by the inner region (Khouri, 1992), with compressive stresses close and parallel to the surface, and tensile stresses in the core. Anyway, both the strength loss and the strength recovery are more pronounced in ordinary than in high-performance concrete (Fig. 6-5, Felicetti and Gambarova, 1998).

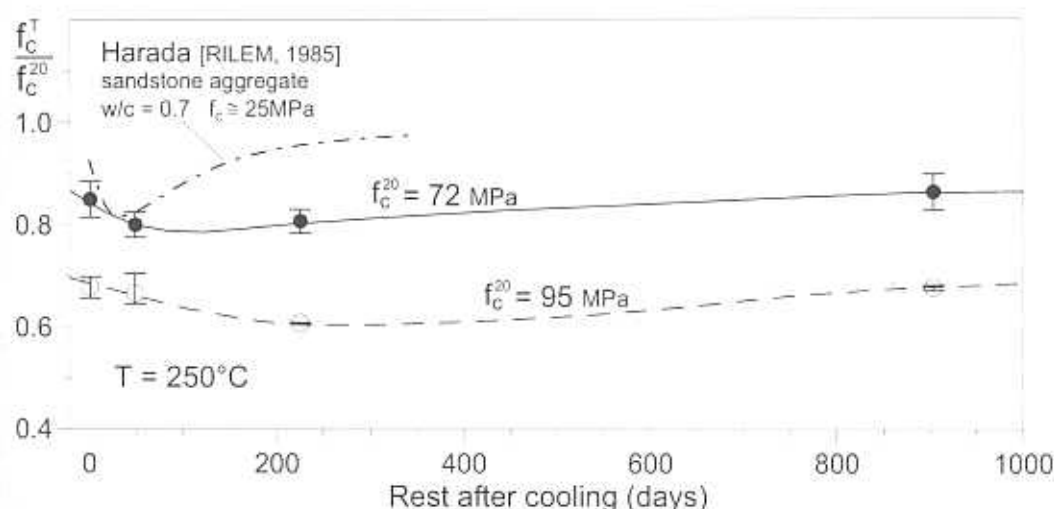


Fig. 6-5: Strength recovery of heated concrete past a cycle at 250°C (Felicetti and Gambarova, 1998; Harada, 1972, see RILEM, 1985)

6.1.3.2 Experimental issues

The measurement of concrete residual properties is generally performed after a thermal cycle at high temperature, with/without pre-loading during the heating and cooling processes (Fig. 6-6). Since the typical parameters characterizing concrete constitutive behavior are to be measured, all possible structural effects should be avoided or reduced to a minimum. To this end, the hygro-thermal conditions inside the specimens during the heating and cooling processes should be as uniform as possible (quasi-steady conditions), since both temperature and humidity are the controlling parameters of all chemo-physical processes and - in the end - of concrete mechanical decay.

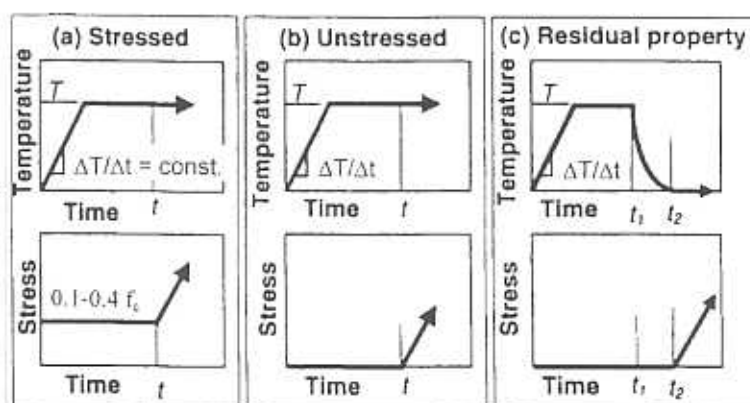


Fig. 6-6: Temperature and loading histories for different test conditions (Phan and Carino, 2002)

As a rule, the thermal ramp in the furnace should be very gentle during the heating process in order to limit temperature gradients ($< 10^{\circ}\text{C}/\text{cm}$) and to control the tensile stresses inside the specimen ($< f_{ct}^T$). The lowest values found in the literature are $0.2\text{--}0.5^{\circ}\text{C}/\text{min}$ (as an example, $0.35^{\circ}\text{C}/\text{min}$ in Hoff et al., 2000; $0.50^{\circ}\text{C}/\text{min}$ in Felicetti and Gambarova, 1998, where the max. differential temperature was $10\text{--}25^{\circ}\text{C}$ at $105\text{--}500^{\circ}\text{C}$ between the surface and the axis of 100 mm HPC cylinders), while the highest values range from 2 to $5^{\circ}\text{C}/\text{min}$ (Cheyrezy, 2001; Phan and Carino, 2002) and up to more than $10^{\circ}\text{C}/\text{min}$ (Papayanni and Valiasis, 1991). Of course, the larger the diameter of the specimens, the greater the differential temperature (up to $70\text{--}110^{\circ}\text{C}$ at $T = 300^{\circ}\text{C}$, $\Delta T/\Delta t = 1^{\circ}\text{C}/\text{min}$, $\varnothing = 150\text{ mm}$, $f_c = 70\text{--}72\text{ MPa}$, see Noumowe and Debicki, 2002).

After the heating process, the specimens are left to rest at the nominal temperature for a certain length of time, which is generally between 1 and 2 hours (up to 3–5 hours in Takeuchi et al., 1993, Phan and Carino, 2002, and up to 12 hours in Felicetti and Gambarova, 1998).

During the cooling process, the thermal ramps should be even gentler, since there is no thermal transient creep to soften the lack of compliance between the thermal strains in the cement paste and those in the aggregate. Values ranging from 0.2 to $1.0^{\circ}\text{C}/\text{min}$ are often found in the literature (Noumowe and Debicki, 2002).

Because of the friendly environment (room temperature), specimen instrumentation poses no problems, and the tests can be easily displacement-controlled, to measure the entire stress-strain curve. In compression, at medium temperatures ($T < 400\text{--}500^{\circ}\text{C}$) the most usual control parameter is the hoop elongation in the specimen midsection (Fig. 6-7a), while at higher temperatures the loss of stiffness in the concrete makes it possible to control the test through the relative displacement of the press platens. In tension, plane or cylindrical notched specimens are the rule, and the control parameter is the displacement astride the mouth of the notch; the cylindrical specimens can be either blocked at their ends (Fig. 6-7b: the specimen end-sections are glued to the press platens and cannot rotate, to the advantage of test control) or hinged (Rosati and Sora, 2001). Though testing concrete in tension at room temperature (residual strength) is tricky, it is much easier than testing at high temperature, and for this reason most of the results found in the literature refer to the residual tensile strength. At high temperature, since gluing the specimen to press platens is no longer possible, special pulling devices are requested, to get hold of specimen extremities; the specimens should have a special shape as well (for instance, a dumbbell shape, as in Felicetti et al., 2000).

Returning to the residual tests in tension, the depth of the notch should be fairly large (for instance $\varnothing_{\text{notched section}}/\varnothing_{\text{nominal}} = 0.6\text{--}0.7$), since the temperature-induced micro-cracks often produce notch effects as large as those associated with the notch itself, or even larger. As a consequence, the crack may localize outside the notched zone, as often occurs in steel fiber-reinforced concrete with high fiber contents (Felicetti et al., 2000).

6.1.3.3 Compressive strength

The residual compressive behavior of normal-strength concrete has been investigated for a long time, starting from the early Sixties (see the contributions by Zoldners, Dougill, Harmathy, Crook, Kasami et al., Schneider and Diederichs, all quoted in RILEM, 1985). The attention was mostly focused on the residual compressive strength as such, on the strength recovery in time and on the residual strain. The main features investigated were:

- (a) the type of cement : no significant differences between pozzolanic and blast-furnace slag cements in terms of residual strength after cooling down to room temperature, but lower strength recovery in the former case, for specimens stored in water;

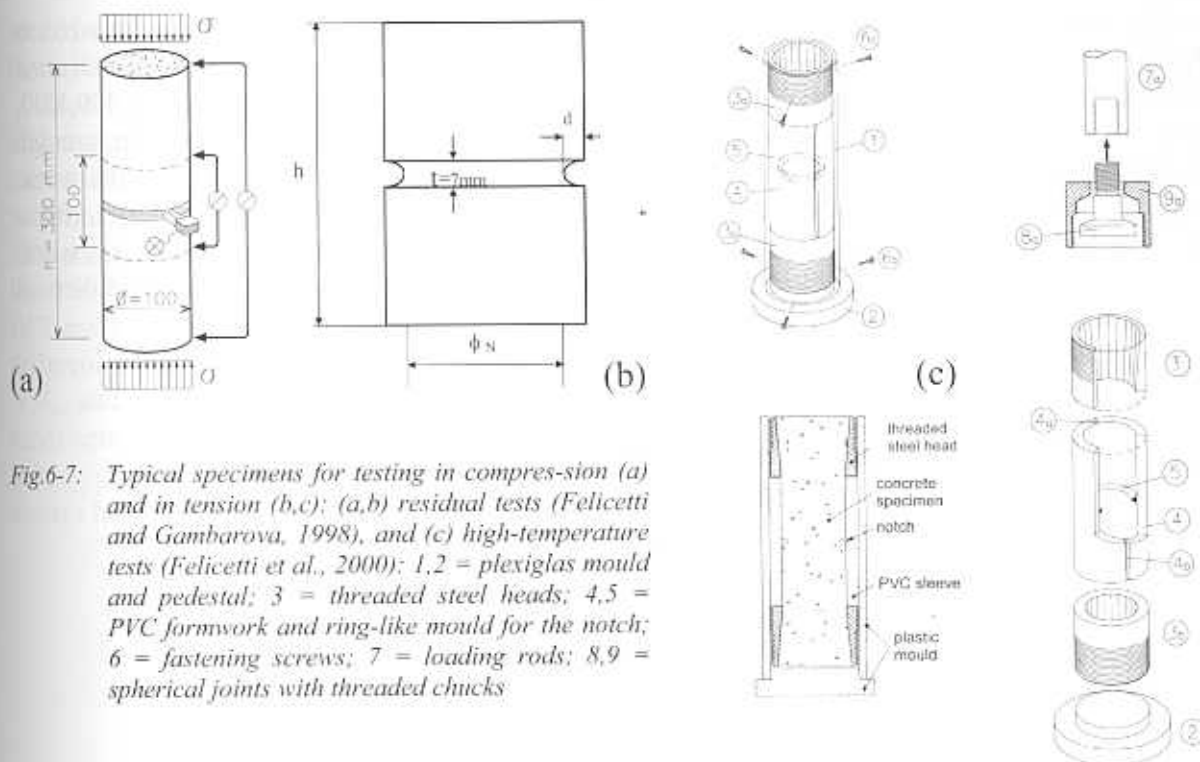


Fig.6-7: Typical specimens for testing in compression (a) and in tension (b,c); (a,b) residual tests (Felicetti and Gambarova, 1998), and (c) high-temperature tests (Felicetti et al., 2000); 1,2 = plexiglas mould and pedestal; 3 = threaded steel heads; 4,5 = PVC formwork and ring-like mould for the notch; 6 = fastening screws; 7 = loading rods; 8,9 = spherical joints with threaded chucks

- (b) the type of aggregate : light-weight concrete exhibits a lower strength loss at high temperature and a lower strength recovery after cooling, compared to ordinary-aggregate concrete; siliceous aggregate gives the concrete a marginally-lower residual strength, compared to calcareous aggregate (Fig. 6-8, $T = 300-600^{\circ}\text{C}$);
- (c) the concrete age : relatively-young concrete with incomplete hydration of the cement may exhibit a strength increase for $T < 400^{\circ}\text{C}$, due to accelerated hydration effects;
- (d) the loading conditions during the thermal cycle : a compressive load applied during heating and cooling (pre-loading = α times the failure load in virgin conditions) may increase by 20-40% the residual compressive strength, for $T = 300-500^{\circ}\text{C}$ ($\alpha = 0.0-0.4$, see Fig. 6-9: in the following, the terms "stressed"/"unstressed" will indicate the tests with/without pre-loading);
- (e) the heat treatment during the cooling process : quenching hot specimens in water from 400°C may decrease the residual strength by 40%, compared to slow cooling, Fig. 6-8;
- (f) the storage conditions after cooling : storage in water decreases the strength loss after cooling, but the results are very limited and often contradictory; in concrete with sandstone aggregate the strength has a minimum 1-2 months past the cooling process, but then a 3-8 month period is sufficient for the strength to recover up to – or even above – the residual strength immediately after cooling, depending on the max. temperature of the thermal cycle (Fig. 6-5, $T = 250^{\circ}\text{C}$; see also Papayanni and Valiasis, 1991).
- (g) the residual strain after a single thermal cycle : limited shrinkage up to $300-450^{\circ}\text{C}$, followed by a marked strain increase up to 6.5-8.5‰ at $700-750^{\circ}\text{C}$ in both calcareous and siliceous mixes; at all temperatures, very limited but continuous shrinkage in expanded-clay mixes.

Since the late eighties, many research projects have been devoted to high-performance/high-strength concrete, special cementitious composites included (LWC, HPLWC and UHPC), with specific reference to the hot and residual properties (see for instance Diederichs et al., 1989, and Castillo and Durrani, 1990).

As indicated by the tests performed by Chan et al. (1996), it appears that compared to NSC (OPC, granite aggregate, $w/c = 0.57$, $f_c = 57$ MPa at 90 days) various HPC mixes (same cement and aggregate, no silica fume, $w/c = 0.35-0.28$, $f_c = 84-118$ MPa) heated to 400, 600, 800, 1000 and 1200°C and then cooled down to room temperature, exhibited similar strength losses, with a marginally-better behavior for HPC up to 400°C, and a definitely worse behavior at higher temperatures (HPC: residual values close to 50-55% and 25-30% after a cycle at 600 and 800°C respectively, compared to 70 and 50% for NSC, Fig. 6-10). It is worth noting that the rest period at high temperature was one hour, and during heating the thermal ramp was 1.7, 2.5 and 5°C/min for $T > 900^\circ\text{C}$, $> 600^\circ\text{C}$ and $< 600^\circ\text{C}$ respectively.

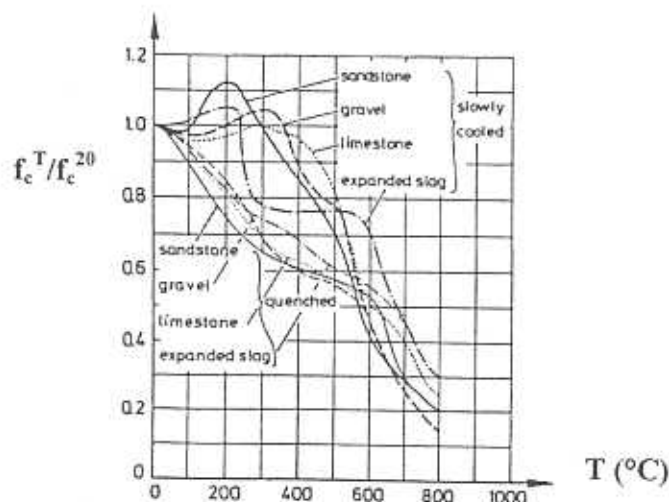


Fig. 6-8: Residual compressive strength of different aggregates, after slow cooling or quenching in water for 5 minutes and drying at 50°C for 18 hours (NSC, $f_c = 27$ MPa from Zoldners, 1960, cited in RILEM, 1985)

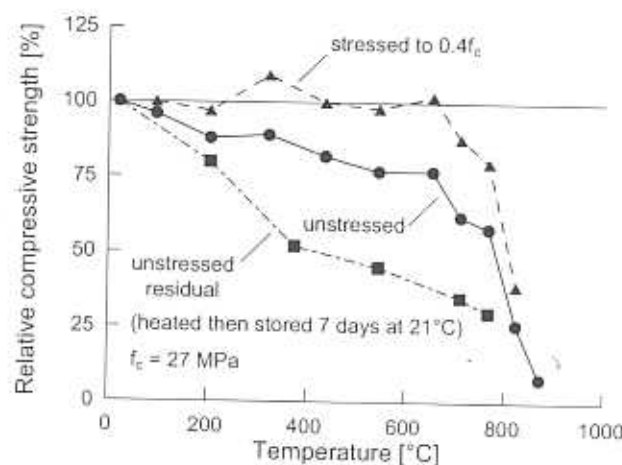


Fig. 6-9: Compressive strength of calcareous concrete (Abrams, 1971, cited by Khoury, 1992): (a) hot tests on preloaded specimens (stressed to $0.40 f_c$ during heating); and (b),(c) hot tests/ residual tests without pre-loading (unstressed specimens)

Even more than in the tests of Chan et al. (1996), the tests performed by Felicetti and Gambarova (1998) on HPC (OPC + 10% silica fume, highly-siliceous aggregate, $w/b = 0.43$ and 0.30 , $f_c = 72$ and 95 MPa, $T = 105, 250, 400$ and 500°C) showed an impressive strength loss, with no differences between the two materials above 300°C and less than 10% of the original strength after a cycle at 500°C (Fig. 6-11). Possible explanations may be the prolonged heating (thermal ramp $0.5^\circ\text{C}/\text{min}$, rest at high temperature 12 hours), the disruptive

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loss of the zeolitical water contained in the flint aggregate and the time elapsed between the thermal process and the tests (1-2 months).

Most of the results obtained in the tests carried out in the Nineties on concrete behavior at high temperature are summarized in NIST S.P. 919 (1997) and in the state-of-the-art paper by Phan and Carino (1998), where normal-strength/normal weight, high-strength/high-performance and light-weight cementitious composites are examined. The behavior at high temperature (stressed and unstressed tests) and after cooling (residual behavior, unstressed tests) is discussed in terms of compressive strength and elastic modulus, with reference also to EC2 and CEB-FIP design curves (Fig. 6-12). In spite of the scattering of the test results by the different authors, it appears that up to 200°C the strength loss in HSC/HPC is smaller than in NSC, and HSC/HPC may even grow stronger, while above 300°C HSC/HPC appears to be more sensitive to the temperature, especially in the case of siliceous aggregate (Fig. 6-12a, lowest curves). However, in the case of light aggregate (Fig. 6-12b), HSLWC/HPLWC always has an edge over LWC.

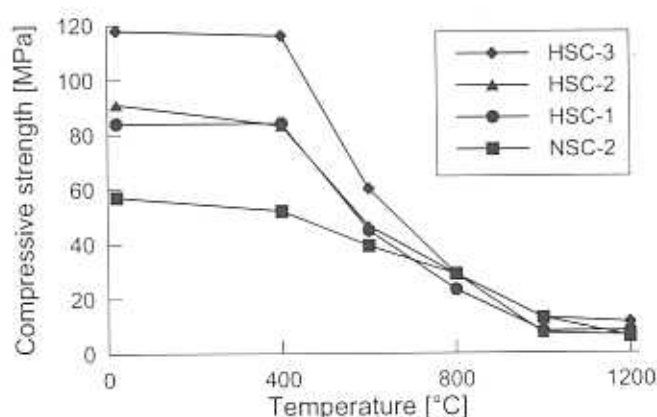


Fig. 6-10: Residual compressive strength of 4 types of concrete subjected to elevated temperatures from 400 to 1200°C (Chan et al., 1996); $f_c = 57, 84, 91$ and 118 MPa at 20°C, age 90 days; cement: OPC; aggregate: crushed granite; no silica fume

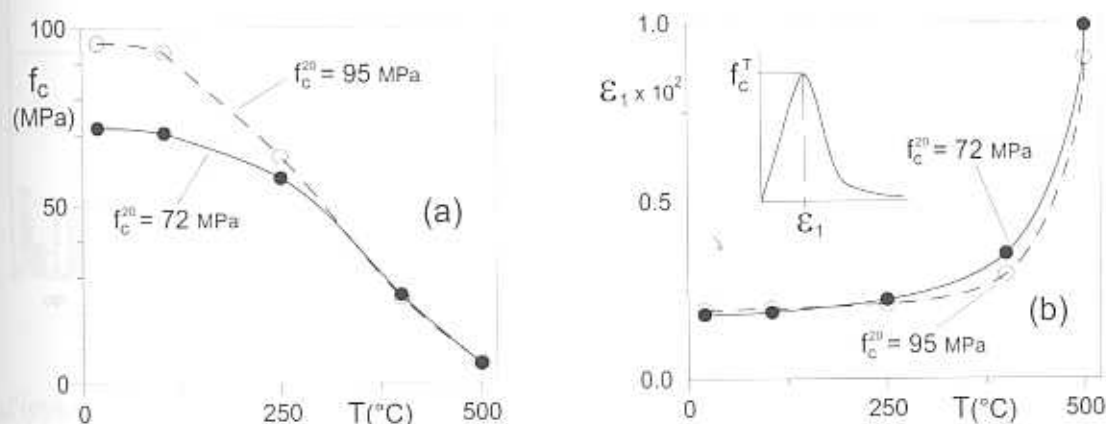


Fig. 6-11: Strength decay (a) and peak-strain enhancement (b) for two high-performance, highly-siliceous mixes heated up to 500°C (Felicetti and Gambarova, 1998)

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More recently, in a paper focused on high-strength composites with and without polypropylene fibers, Hoff et al. (2000) investigated the residual behavior in compression of 12 types of concrete heated up to 1100°C (heating rate 0.35°C/min, rest at high temperature ≈ 2 hours). All the mixtures contained OPC and silica fume ($w/b = 0.32$), but the coarse aggregate was different, since 4 mixtures had crushed limestone or granite, 4 expanded slate

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or slag, and 4 had a mix with 55% of limestone/granite and 45% of expanded slate. Fibrillated polypropylene fibers were added to 6 mixtures ($L = 20$ mm, $v_f = 0.15\%$ by volume) to investigate the spalling tendency of the materials (however, spalling never occurred during the tests, most probably because of the very low heating rate). As demonstrated by the diagrams of Figs. 6-13a,b, the conclusions were that HSC/HPC and HSLWC/HPLWC are no worse than NSC/LWC after a thermal cycle at high temperature, and that the favorable effects of polypropylene fibers towards spalling are not accompanied by possible unfavorable effects on concrete strength, because of fiber-induced porosity. However, a loss of 5% on concrete cubic strength (virgin specimens) was observed by Lennon and Clayton (1999) for fiber contents equal to 0.3% by volume.

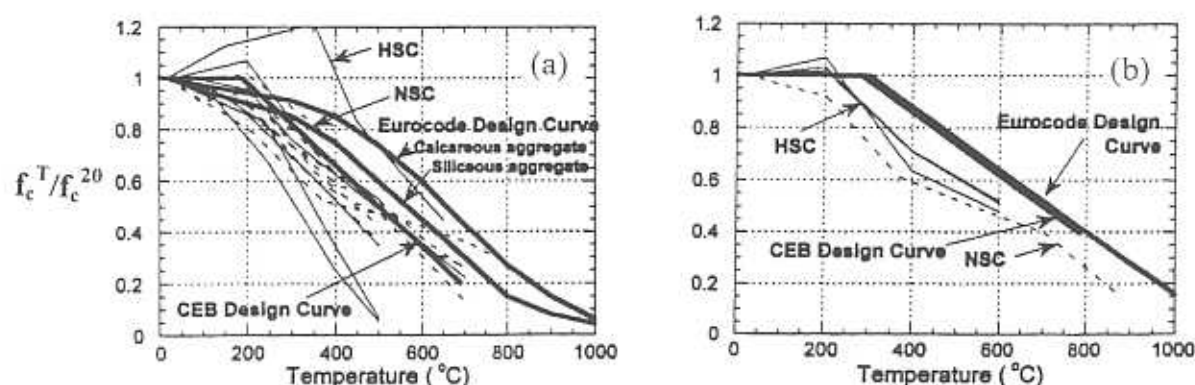


Fig. 6-12: Test results and recommended design curves for the residual compressive strength of NSC and HSC/HPC with ordinary aggregate (a), and with light-weight aggregate (b). Unstressed tests (Phan and Carino, 1998)

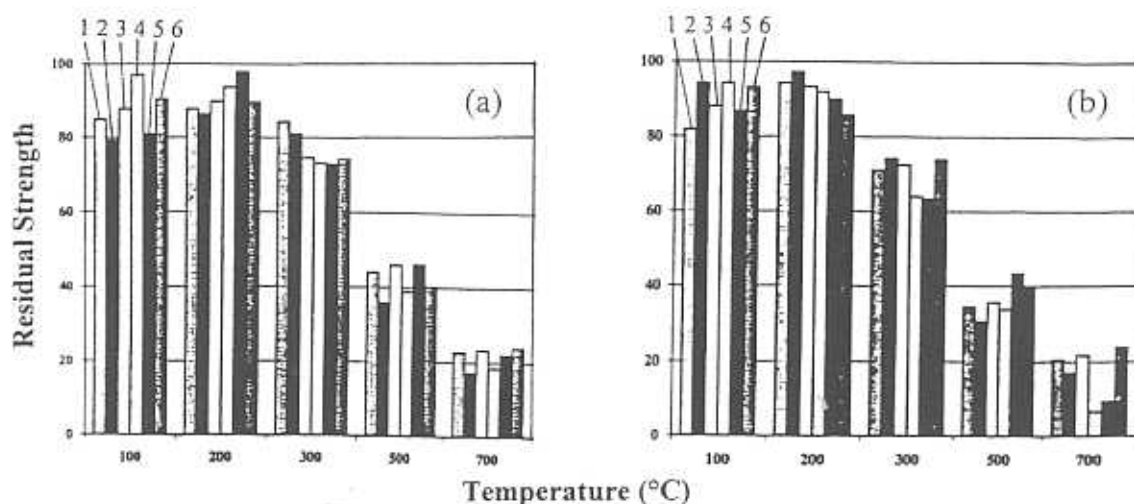


Fig. 6-13: Residual strength in compression for different types of concrete: (a) with no fibers, and (b) with fibers. 1 = Normal-Density, Granite; 2 = Normal-Density, Limestone; 3 = Normal-Density, Granite + Expanded Slate; 4 = Normal-Density, Limestone + Expanded Slate; 5 = Light-Weight, Expanded Slate; 6 = Light-Weight, Slag (Hoff et al., 2000)

The practical irrelevance of polypropylene fibers with regard to the residual compressive strength was confirmed also by the tests carried out by Chan et al. (2000) on 4 different mixtures, all with OPC, crushed granite and fly ash (f.a./c = 33-38%). The water/binder ratio was 0.6 in the NSC ($f_c = 34.9$ MPa) and 0.32 in the three HSCs (all with silica fume = 0.15 c, $f_c = 97.3, 113.5$ – with steel fibers, $v_f = 1\%$ – and 99.1 MPa – with polypropylene fibers, $v_f = 0.2\%$). After a single cycle at 800°C ($\Delta T/\Delta t = 5-7^\circ\text{C}/\text{min}$, rest at 800°C 1 hour), the residual

strength of the 3 HPC mixtures was 24 and 26% (plain concrete and concrete with polypropylene fibers), and 34% (concrete with steel fibers) compared to 45% for the NSC mixture (no fibers).

The conclusions were that silica-fume mixtures seem to be more temperature-sensitive, and that moderately-large steel fiber contents help in terms of residual strength. Such favorable effect is enhanced by the use of high contents of steel micro-fibers, as shown in the study by Felicetti et al. (2000) concerning the hot and residual behaviors of 3 mixtures (HSC with hyposiliceous aggregate, s.f./c = 0.10, w/b = 0.29, $f_c = 92$ MPa; CRC with s.f./c = 0.30, w/b = 0.16, steel micro-fibers with $v_f = 6\%$ by volume, $f_c = 158$ MPa; RPC with s.f./c = 0.25, w/b = 0.14, steel + polypropylene micro-fibers 2% + 2% by volume, $f_c = 165$ MPa, BRITE-EURAM HITECO III). After a cycle at 600°C both the CRC and RPC still retained a little less than 70% of their initial strength, while the HPC had only 44%.

Silica-fume concrete sensitiveness to high temperature was confirmed by the many tests carried out within the HITECO European Research Program (1996-99, see Cheyrezy, 2001), where 4 high-performance mixtures were studied ($f_c = 60, 60, 75$ and 90 MPa, with w/b = 0.36-0.29), at high temperature ($T = 100-700^\circ\text{C}$, $\Delta T/\Delta t = 2^\circ\text{C}/\text{min}$, 1 hour-rest at T) and after cooling down to room temperature, with/without silica fume, with/without pre-loading ($\sigma_c = 0.2, 0.3 f_c$). While the silica-fume specimens tended to concentrate in the lower portion of the scatter limits (Fig. 6-14), the scatter in itself was found to be very close to that of conventional mixtures, and the strength values obtained in stressed/unstressed conditions, at high temperature and after cooling, turned out to be pretty much the same.

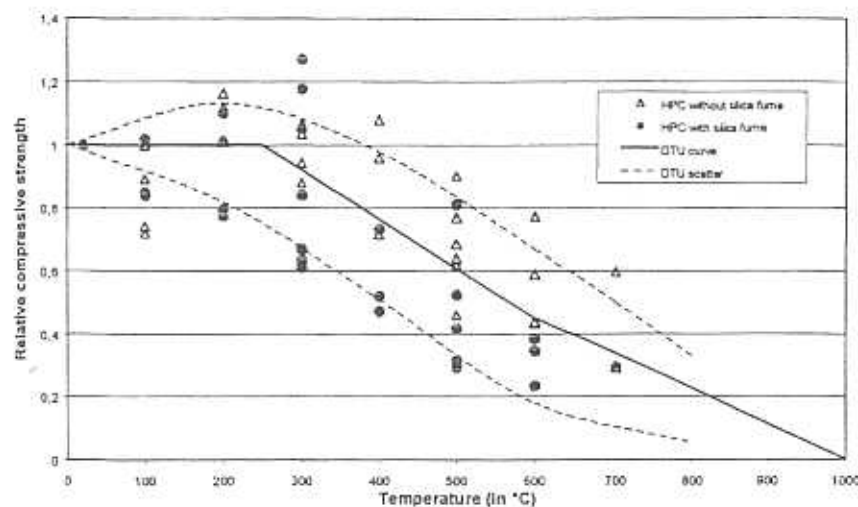


Fig. 6-14: Summary of the compression tests performed within the HITECO Program (1996-99) with reference to silica-fume effects (all tests, at high temperature and after cooling, Cheyrezy, 2001); DTU = Document Technique Unifié and relative scatter

New information not only on the mechanical properties, but also on the spalling tendency of different high-performance composites is given by the very recent investigation by Phan and Carino (2002) on HPC/HSC behavior at high temperature and after cooling, with/without silica fume, with/without pre-loading during the thermal process ($\sigma_c^0 = 0.40 f_c^{20}$). Four mixtures (2 with s.f. = 11% cement, $f_c = 98$ and 88 MPa, w/b = 0.22 and 0.33, and 2 without s.f., $f_c = 75$ and 50 MPa, w/b = 0.33 and 0.57, all with crushed limestone as coarse aggregate) were heated up to 600°C, with 4 intermediate temperature levels ($T = 100, 200, 300$ and 450°C). The heating rate was $5^\circ\text{C}/\text{min}$, and the rest period at the nominal temperature T was never less than 2 hours. Since several specimens exhibited spalling-related explosions during the heating up to 600°C (probably because of the relatively-high heating rate and the ensuing

thermal stresses), the data of the “hot” stressed/unstressed tests concerning this temperature are not complete (Figs. 6-15a,b). Furthermore, in the unstressed, residual tests the temperature was limited to 450°C (Fig. 6-15c). The main conclusions were: (a) there are no significant differences between the stressed and unstressed “hot” tests, up to 450°C; (b) the residual strength (unstressed tests) is generally higher than the “hot” strength (stressed/unstressed tests) for low-medium temperatures (up to 200-250°C), while the opposite is true at medium-high temperatures (450°C); (c) on the whole, the HSC mixture with the lowest w/b ratio exhibits the smallest strength loss in all conditions, but especially in the residual conditions (Fig. 6-15c); and (d) the presence of silica fume has no statistical significance; however, in the stressed and unstressed tests at high temperature (Figs. 6-15a,b) the strength loss of the lower strength, silica-fume mixture appears to be the highest.

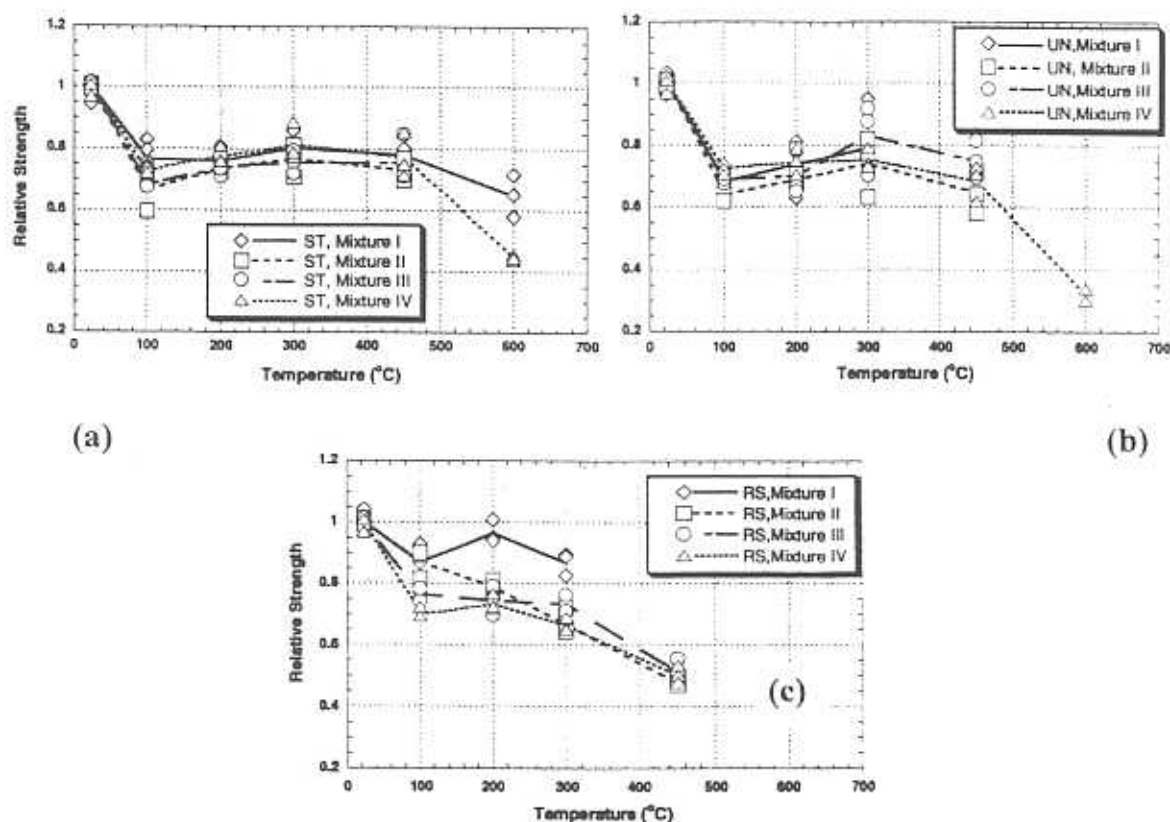


Fig. 6-15: Hot stressed (a), hot unstressed (b) and residual (c) tests by Phan and Carino (2002): compressive strength of different HSC/HPC mixtures (Mixture I, $f_c = 98$ MPa, and Mixture II, $f_c = 88$ MPa, both with silica fume; Mixture III, $f_c = 75$ MPa, and Mixture IV, $f_c = 50$ MPa, both without silica fume)

Finally, some very recent results on special cementitious composites and special loading conditions should be cited.

In Guerrini and Rosati (2003) a high-performance “white” concrete ($f_c = 67.3$ MPa, w/b = 0.40), containing titanium dioxide - enriched cement (350 kg/m³), crushed marble (as coarse aggregate) and metakaolin (10.5% of cement by mass) was heated up to 750°C, in order to measure the residual properties ($\Delta T/\Delta t = 1^\circ\text{C/h}$, 2 hour-rest at the nominal temperature). As shown in Fig. 6-16 (where the Young’s modulus is plotted as well, but will be discussed later), the relative residual strength after being heated to 750°C is as low as 11%, compared to the usual values of 45-50% for conventional concrete and 25-35% for high-performance silica-fume concrete (mixed aggregate). However, it is fair to remember that this costly white concrete is used in monumental structures, where fire is not among the major factors in the design.

Adding metallic microfibers to the mix helps in terms of residual strength, provided that fiber content is sizable ($> 2\%$ by volume), as shown in Fig. 6-17, where the residual cubic strengths of 4 cementitious mixes are plotted versus the temperature (Felicetti et al., 2000). HSC and HSC* are two high-performance concrete (with hyposiliceous and highly-siliceous aggregates, $f_c = 92$ and 95 MPa, respectively), while CRC is a Compact fiber-Reinforced Concrete ($f_c = 158$ MPa, steel microfibers, $v_f = 6\%$ by volume) and RPC is a Reactive-Powder Concrete ($f_c = 165$ MPa, steel microfibers + polymeric fibers, $v_f = 2 + 2\%$ by volume). All mixes – except HSC* – were studied within the HITECO European Project (High Temperature in CONcrete, 1995-99). Because of the favorable effects of the fibers, the residual strength of both CRC and RPC is still 65-70% of the original strength, after a thermal cycle at 600°C . Note the much better behavior of the hyposiliceous HSC compared to the highly-siliceous HSC*.

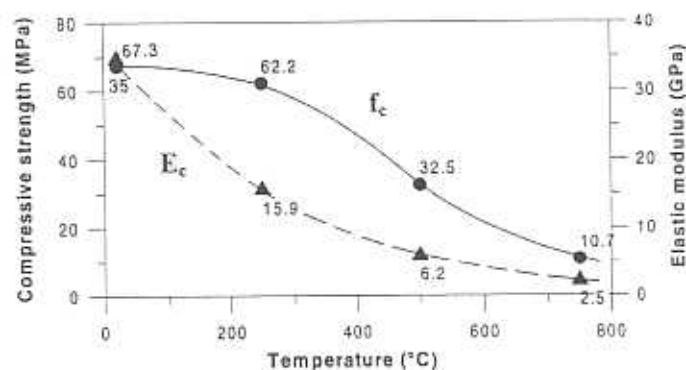


Fig. 6-16: Residual compressive strength and Young's modulus for a white concrete containing titanium-enriched cement and metakaolin (Guerrini and Rosati, 2003): $f_c = 67.3$ MPa, $E_c = 35.4$ GPa (secant modulus)

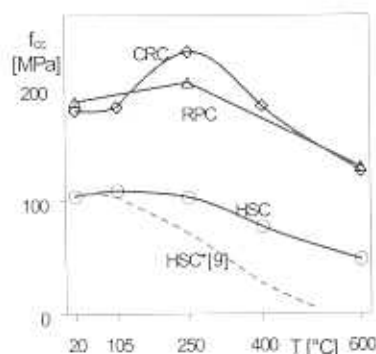


Fig. 6-17: Residual cubic strength of 4 cementitious composite sites: HSC*-highly-siliceous concrete ($c = 415$ kg/m³, $sf/c = 6.7\%$, $w/b = 0.30$); HSC-hyposiliceous concrete ($c = 510$ kg/m³, $sf/c = 10\%$, $w/b = 0.29$); CRC-Compact fiber-Reinforced Concrete ($c = 720$ kg/m³, $sf/c = 30\%$, $w/b = 0.16$); and RPC-Reactive Powder Concrete ($c = 933$ kg/m³, $sf/c = 25\%$, $w/b = 0.14$), see Felicetti et al., 2000

In Zhou and Zhang (2001) the fatigue behavior of a conventional concrete ($f_c = 23$ MPa, $w/c = 0.6$, limestone aggregate, Type I cement) was studied after a thermal cycle at 200 and 300°C , under fatigue loading ($\sigma_{\max}/f_c = 0.6, 0.7, 0.8$; $\sigma_{\min}/f_c = 0.1$; frequency 6 Hz; 2×10^6 cycles). The authors found that - because of the progressive propagation of the thermally-induced micro-cracks, followed by their coalescence into continuous macro-cracks - even temperature as low as 200°C can reduce the fatigue strength by 30%, though the strength under monotonic loading is hardly affected by the temperature up to 300°C .

6.1.3.4 Elastic modulus and Poisson's ratio

Not many studies have been devoted to the modulus of elasticity, be it at high temperature or after cooling, and the interpretation of the data found in the literature is not univocal, since

there are at least three different ways to define this parameter. The modulus of elasticity (Fig. 6-18) can be defined as: (a) the slope at the origin of the stress-strain curve (tangent modulus E_0); (b) the mean slope of the first-loading curve (secant modulus E_c , which requires the introduction of an upper limit αf_c for the stress, with $\alpha = 0.3-0.4$); or (c) the mean slope of the last load cycle past a number of cycles (stabilized modulus E_c^* ; 3 load cycles are proposed in RILEM 2004, Fig. 6-18, with $\alpha = 0.30$, $\beta = 0.05$ and $\gamma = 0.15$). Unfortunately, in the literature it is not always clear what definition has been adopted. This is not a trivial matter, since the first definition regards only the domain of very small stresses and strains, the second definition considers the initial inelastic settlement of the material as a sort of equivalent elastic strain (which is reasonable for the first permanent loads, like the self weight), while the third definition refers to the mostly-elastic behavior of an already stabilized material (as in the case of the variable loads, to be applied after the self weight). In the following, the values of the modulus are those of the tangent modulus, unless otherwise declared, and the symbol E_c is always adopted for simplicity.

With reference to normal-strength concrete, the studies carried out in the Sixties, Seventies and early Eighties (RILEM, 1985) show that the elastic modulus depends mainly on the compressive strength of the material. Consequently, the elastic modulus is affected by most of the factors influencing the strength (type of cement, water-cement ratio, type of aggregate, loading conditions during the thermal cycle, maximum temperature reached during the heating process). However, besides the last parameter, only the type of aggregate and – to a lesser extent – the loading conditions during the heating process have a sizeable influence on the elastic modulus, which is more temperature-sensitive than the compressive strength (both at high temperature and after cooling), because concrete stiffness is directly affected by thermal cracking. Siliceous aggregates (like basalt and quartzite, $E_c^T/E_c^{20} = 0.2-0.3$ at 500°C) make the concrete more temperature-sensitive than calcareous aggregates (like carbonate and sandstone, the latter being partly siliceous, $E_c^T/E_c^{20} = 0.4-0.5$ at 500°C), while light-weight aggregates are close to- or better than calcareous aggregates (at 500°C , E_c^T/E_c^{20} up to $0.70-0.75$ for expanded clay, RILEM, 1985, Fig. 6-19). As for the cement type, in most of the tests performed so far portland cement has been used; consequently, no mention of the cement type is made in the following, unless a different cement type has been used.

With reference to the elastic modulus at high temperature and after a thermal cycle, one has to remember that in both cases the static method (based on the measurements of the displacement at mid-height and of the load) and the dynamic method (based on the measurement of the resonant frequency) are used. However, some tests show that the dynamic method may enhance the thermal dependency of the modulus, at least up to 400°C (RILEM, 1985), but other tests indicate that there is hardly any difference between the two methods (Phan and Carino, 2002).

According to the very recent and already-cited test results by Phan and Carino (2002), at high temperature and after cooling, in the stressed and unstressed states, with and without silica fume (Figs. 6-20a,b,c, see also Figs. 6-15a,b,c), there is hardly any difference at high temperature, be the specimens in the stressed state (Fig. 6-20a) or in the unstressed state (Fig. 6-20b), and there is not much difference either between the former results and those after cooling (residual values, Fig. 6-20c), even if the latter are marginally better up to 150°C and marginally worse above 300°C . Mixture IV ($f_c = 50$ MPa, no silica fume) is always the worst, while Mixture III ($f_c = 75$ MPa, with silica fume) is the best, whatever the test conditions may be. This conclusion is at odds with the results of other tests like those by Papayianni and Valiasis (1991), which show a worse behavior for the mixes containing pozzolanic materials, like fly ash (however, it should be noted that in these tests the high thermal rates and the ensuing sizable thermal gradients might have played a non marginal role in damaging the material).

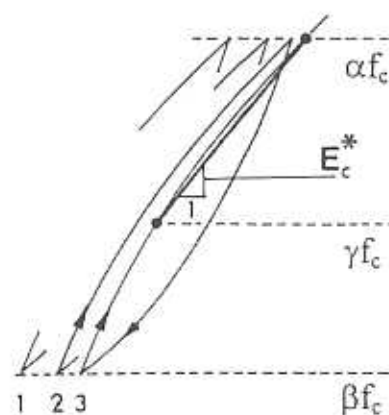
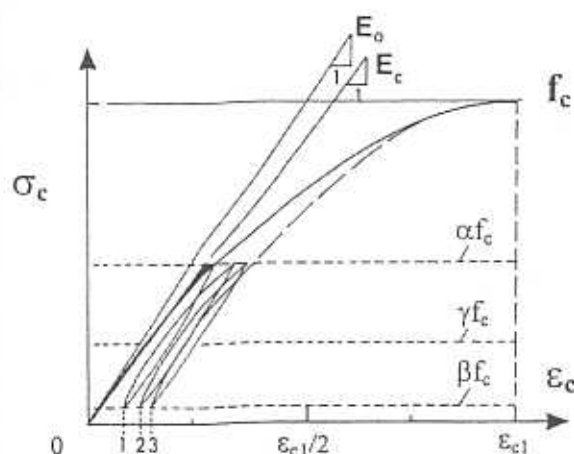


Fig.6-18: Elastic modulus of the concrete: E_o tangent modulus, E_c secant modulus, and E_c^* stabilized modulus (RILEM, 2001). For simplicity, only the symbol E_c will be used in the following, whatever the nature of the modulus.

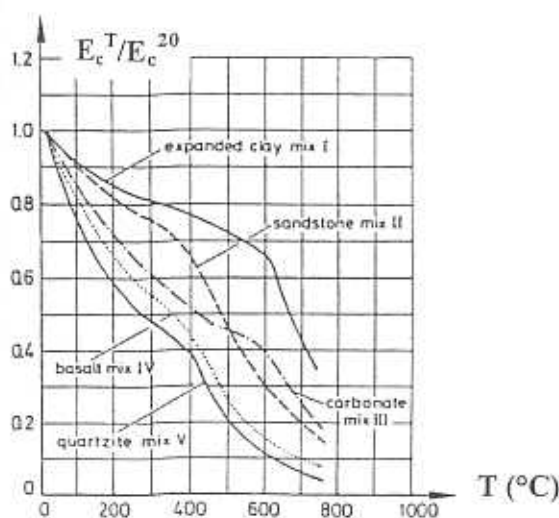


Fig. 6-19: Elastic moduli of similar mixes containing different types of aggregate: Mix I, expanded clay, $f_c = 18$ MPa; Mix II, sandstone, $f_c = 44$ MPa; Mix III, carbonate, $f_c = 28$ MPa; Mix IV, basalt, $f_c = 42$ MPa; Mix V, quartzite, $f_c = 35$ MPa; cement content $c = 350$ kg/m³; aggregate/cement ratio ≥ 5.6 (Mix I: 2.4); $w/c = 0.50-0.65$ (RILEM, 1985).

Note that since the thermally-induced damage depends mostly on the maximum temperature reached by the material, the very limited difference between the hot and the residual tests is not unexpected. This is also an indirect proof that the static method (Figs. 6-20a,b) and the dynamic method (Fig. 6-20c) are equally reliable in the evaluation of the elastic modulus.

The aggregate/cement ratio and the water/cement ratio have sizable and similar effects on the residual elastic modulus, as shown by the tests run by Thienel and Rostasy (1993) on highly-siliceous NSC mixes. In Fig. 6-21a the leaner mix loses less than the richer mix, in terms of elastic modulus after a thermal cycle at 300°C (-40% compared to -50%), because the thermal cracking due to the different thermal expansion of the aggregates and of the cement paste plays a smaller role in lean mixes. After a cycle at 600°C the opposite is true, because of the great thermal sensitivity of the siliceous aggregate, which plays a greater role in lean mixes (roughly 5% left, compared to 15% in the richer mix). The same is exhibited by the effects of the water/cement ratio (Fig. 6-21b), since the higher value helps after a cycle at 300°C (roughly -30% compared to -50%), but is detrimental after a cycle at 600°C (roughly 7% left, compared to 14%). The former effect may be explained with the greater thermal diffusivity ensuing from higher w/c ratios (more porosity, more water and steam transport, smaller thermal gradients and microstructural damages at medium temperatures), while the

higher porosity may explain why at elevated temperature a higher w/c ratio leads to an enhanced stiffness decrease.

Among the most recent studies on the mechanical behavior of high-performance concrete at high temperature, the experimental program carried out within the Project BHP 2000 (Pimienta and Hager, 2002) should be quoted. Here only the data on the elastic modulus are presented and commented, since the material's stiffness is practically the same at high temperature and after cooling, and the hot values give valuable information on the residual values (as a matter of fact, the damage in the material is mostly a function of the maximum temperature reached before or during the test). The concrete cylinders were preheated up to the nominal temperature (120, 250, 400 and 600°C) and then were wrapped with an insulating cloth, to keep them hot during (a) the mounting of the instruments, (b) the placing of each specimen between the press platens and (c) the loading. Only the ascending branches were measured, and the elastic modulus was evaluated as the slope of the stress-strain curve up to a prefixed strain value (0.5‰, 1.0‰ and 4‰, secant modulus). Four different mixes were tested in compression, to evaluate the compressive strength and the elastic modulus at high temperature: 3 mixes had calcareous aggregates (M30C, M75C and M100C, with $f_c = 30, 75$ and 100 MPa) and 1 had silico-calcareous aggregates (M75SC, with $f_c = 75$ MPa). The plots of Fig. 6-22 show that the silico-calcareous concrete (M75SC) undergoes the largest loss at any temperature (except at 120°C), the stiffness being completely lost at 600°C. On the contrary, the strongest concrete (M100C) has the best behavior, while the normal-strength concrete (M30C) and the intermediate high-performance concrete (M75C) fall between the two extremes. On the whole the HPC- M100C performs quite well, and better than the NSC- M30C (at 600°C, in the former case $E_c/E_c^{20} = 16\%$, while in the latter case $E_c/E_c^{20} < 10\%$).

Looking at "special" cementitious composites, the above-mentioned white concrete (Fig. 6-16) behaves in pretty much the same way as the silica-fume mixes tested by Phan and Carino (Fig. 6-20c), while the high-performance silica-fume light-weight concrete tested recently by Felicetti et al. (2002) seems to be as affected by the temperature as a similar ordinary concrete, but more affected than a similar normal-strength light-weight concrete (Fig. 6-23).

Finally, in Fig. 6-24 the secant elastic moduli of 3 of the 4 cementitious composites presented in Fig. 6-17 are plotted as a function of the temperature. In spite of the limited number of the tests (at high temperature, Imperial College, London-UK, and after cooling, Milan University of Technology, Milan-I, see Felicetti et al., 2000), and in spite of the good performance of both CRC and RPC in terms of strength (Fig. 6-17), the elastic moduli are tremendously affected by the temperature, more in the fiber-reinforced composites than in the high-strength concrete ($\approx 15\%$ compared to $\approx 20\%$ after a cycle at 600°C). For the HSC the elastic moduli at high temperature and after cooling tend to coincide, as already shown by other test results (Phan and Carino, 2002). For the ultra high-performance composites the conclusion seems to be the same, but further tests are necessary.

As for the Poisson's ratio, it is generally evaluated by measuring the transverse strains in a specimen loaded in uniaxial compression, but can also be inferred from the elastic and shear moduli (by loading a cantilever beam subjected to bending and torsion). The few and often very dispersed data on this parameter show that in the case of siliceous aggregates the Poisson's ratio tends to decrease with the temperature, with no significant differences between the high-temperature and the residual values above 100°C ($\nu_c = 0.23-0.10$, with 0.10 both at 400°C and after cooling, RILEM, 1985). However, other results indicate a completely contrary trend for both siliceous and calcareous mixes (up to 0.23 at 500°C), with a marginal dependency on the temperature for light-weight mixes. Furthermore, some recent results obtained in Milan (Bamonte et al., 2006) by measuring the residual dynamic moduli (E_{CD} and G_{CD}) of a high-performance siliceous micro-concrete ($f_c = 120$ MPa at 20°C) have shown that the residual Poisson's ratio ν_{CD} first increases up to 0.28-0.30 for $T = 450-500^\circ\text{C}$ (Fig. 6-25)

and then sharply decreases. A possible explanation is that at first the material weakens – and dilates – because of thermally-induced microcracking. After, the outward expansion is counterbalanced by a sort of inward expansion, that takes advantage of the sizable extra-porosity due to thermal microcracking.

6.1.3.5 Tensile strength

Very little attention has been devoted so far to concrete behavior in tension, be it direct tension or indirect tension (in bending or splitting). As a matter of fact, before the mid-eighties (RILEM, 1985) the studies on this topic could be counted on one-hand fingers and a few of them were still unpublished. The situation has improved a bit in the last 15 years, because of the need to have more or new information on the mechanical behavior of the many innovative cementitious composites, that have been proposed and are entering the market. Furthermore, another reason to investigate concrete properties in tension is the “spalling” of the material. This multi-faceted phenomenon is triggered by high temperatures and thermal gradients, but depends also on the structural context, on concrete microstructure and water content (Khoury, 2000).

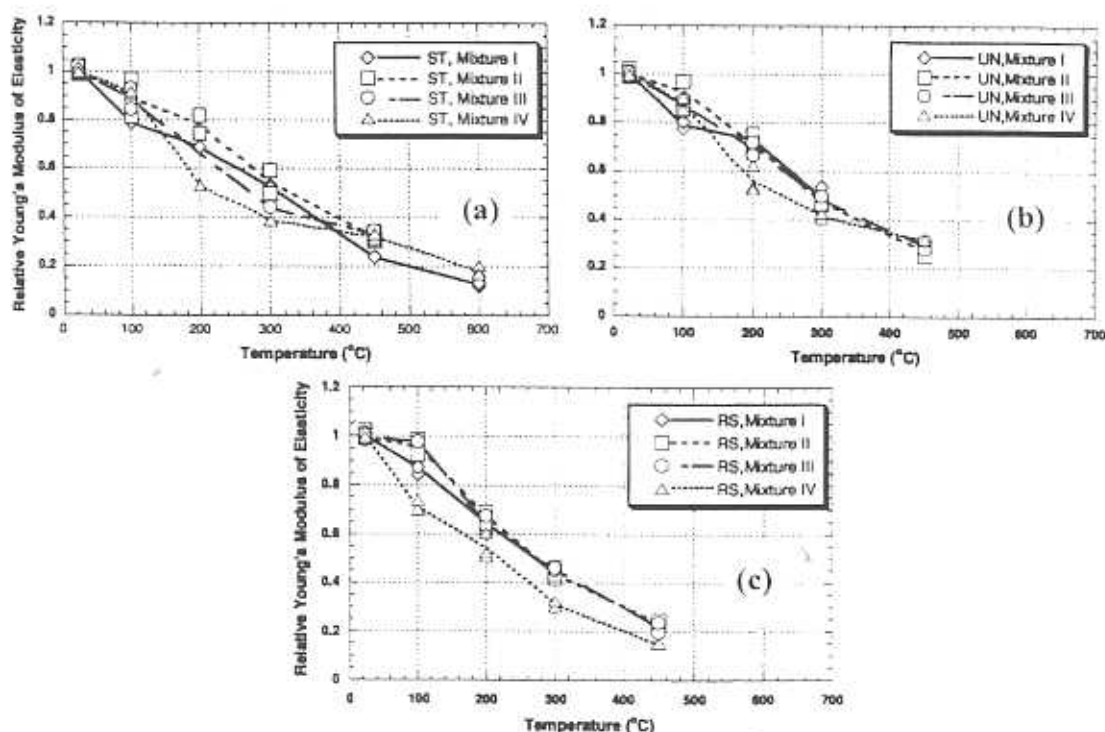


Fig. 6-20: Hot stressed (a), hot unstressed (b) and residual unstressed (c) tests by Phan and Carino (2002): elastic moduli of different HSC/HPC mixtures (Mixtures I and II, $f_c = 98$ and 81 MPa, $E_c = 47$ and 44 GPa, with silica fume; Mixtures III and IV, $f_c = 72$ and 47 MPa, $E_c = 44$ and 37 GPa, without silica fume). In (a,b) static moduli, and in (c) dynamic moduli. Strength and elastic modulus at 400 days from casting.

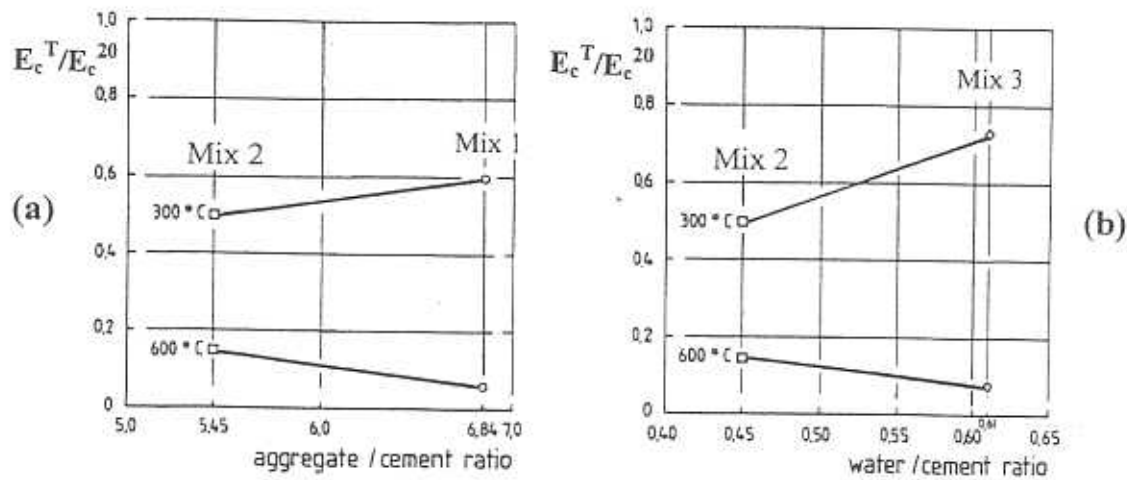


Fig. 6-21: Residual unstressed tests on normal-strength mixes with quarzitic aggregates: (a) effects of the aggregate content; and (b) effects of the water/cement ratio; $\square$ Mix 1, $f_c = 36$ MPa; $\square$ Mix 2, $f_c = 45$ MPa; $\diamond$ Mix 3, $f_c = 36$ MPa; $E_c^{20} \geq 33$ -40 GPa (Thienel and Rostasy, 1993)

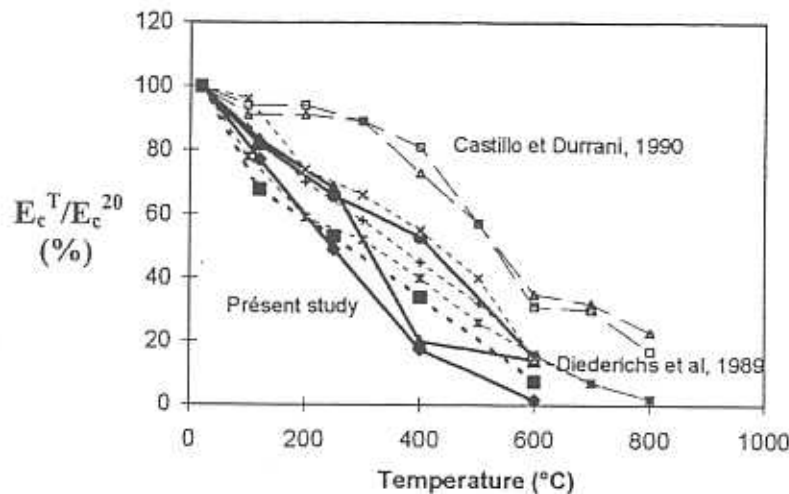


Fig. 6-22: High-temperature unstressed tests on NSC and HPC with calcareous-C or silico-calcareous-SC aggregates (Pimienta and Hager, 2002): $\blacksquare$ M30C, $\blacktriangle$ M75C, $\diamond$ M75SC, and $\bullet$ M100C, with $f_c = 37, 107, 92$ and 113 MPa and $E_c = 36, 48, 49$ and 51 GPa; $\times$ $f_c = 91$ MPa with fly ash; $\times$ $f_c = 85$ MPa with blast-furnace slag; $+$ $f_c = 106$ MPa with silica fume; $\square$ $f_c = 31$ MPa (unstressed); and $\triangle$ $f_c = 63$ MPa (unstressed)

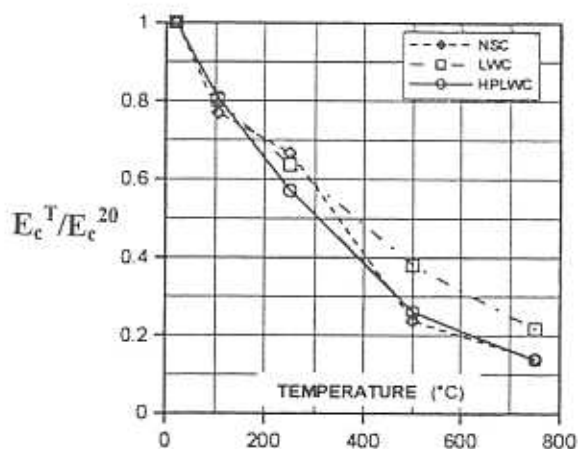


Fig. 6-23: Residual tests on normal and light-weight mixes with mostly calcareous aggregates (Felicetti et al., 2002): NSC, $f_c = 30$ MPa, $E_c = 25$ GPa, $w/c = 0.67$, $c = 300$ kg/m³; LWC, $f_c = 39$ MPa, $E_c = 16.5$ GPa, $w/c = 0.63$, $c = 350$ kg/m³; HPLWC, $f_c = 56$ MPa, $E_c = 17.5$ GPa, $w/b = 0.43$, $c = 500$ kg/m³ and $sf/c = 10\%$; All E_c values are stabilized values

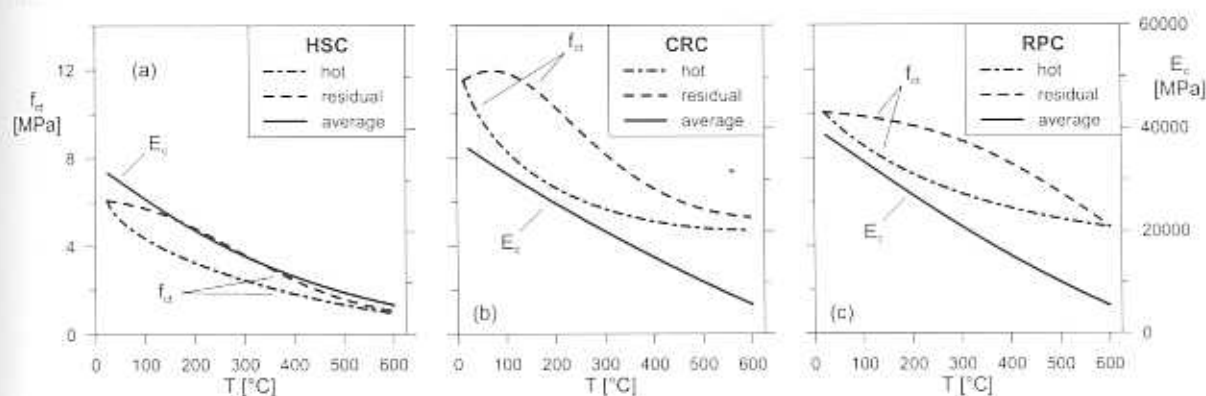


Fig. 6-24: Hot and residual tests on high-performance and ultra high-performance cementitious composites: HSC (hyposiliceous aggregates), $f_t = 92$ MPa; CRC-Compact fiber-Reinforced Concrete, $f_t = 158$ MPa (steel microfibers with $v_f = 6\%$ by volume); and RPC-Reactive-Powder Concrete, $f_t = 165$ MPa (steel microfibers + polymeric fibers, $v_f = 2 + 2\%$ by volume), see Felicetti et al., 2000

As a rule, high temperature affects concrete strength more in tension than in compression, and the residual strength in tension tends to be somewhat lower than at high temperature in ordinary concrete.

While the heating rate and the storage time-length after cooling have negligible effects on the residual strength in tension, the mix design and the type of aggregate have a moderate/significant influence. Lean mixes (with low cement content) exhibit lower strength reductions, while calcareous aggregates make concrete strength in tension much more sensitive to high temperature, than siliceous aggregates (at 500°C , in the latter case the strength in tension is twice as much as in the former case, RILEM, 1985).

By far, most of the tests have been performed after cooling, to measure the residual properties in tension, and to evaluate some fracture parameters, like specific fracture energy, characteristic length and toughness. With reference to high-temperature, direct-tension tests, those performed by Takeuchi et al. (1993) on ordinary, siliceous concrete ($f_c = 50$ MPa), and by Felicetti and Khoury (Felicetti et al., 2000) on high-performance and ultra high-performance concretes ($f_c = 92$ -165 MPa) should be quoted, since they are the only well-documented tests available so far.

Some of the results by Takeuchi et al. (1993) are shown in Fig. 6-26, where the decay of the strength in tension appears to be comprised between the decay of the strength in compression (less affected by the temperature) and that of the secant elastic modulus (more affected). Takeuchi performed the tests on standard-cured cylinders (100% R.H., $f_c = 43.5$ MPa), on air-dried specimens (65% R.H., $f_c = 40.1$ MPa), at high temperature (65 - 800°C) and after cooling (800°C). All specimens were heated 5 weeks after casting ($\Delta T/\Delta t = \pm 1^\circ\text{C}/\text{min}$). At 800°C the residual properties were always lower than the hot properties, while no major differences were found between the moist-cured and the air-cured specimens.

The test results obtained in the previously-mentioned HITECO Project (Felicetti et al., 2000) are shown in Fig. 6-24. Three high-performance and ultra high-performance cementitious composites (HSC, CRC and RPC) were tested both at high temperature and after cooling (105 , 300 and 500°C in the hot tests; and 105 , 250 , 400 and 600°C in the residual tests). As it is well known for virgin steel fiber-reinforced concrete, medium-to-high steel fiber contents ($v_f^{\text{steel}} = 2$ - 6%) guarantee high residual-strength values (Fig. 6-24b,c) compared to an unreinforced concrete (Fig. 6-24a).

As already mentioned, most of the tests in tension have been performed after cooling, since in this way the tests are by far simpler (no need of relatively large, water-cooled grips, and of complex instrumentation to measure the displacements; no specimen limitations due to

the size of the furnace chamber; less time required by each test, since heating and testing can be performed at different times).

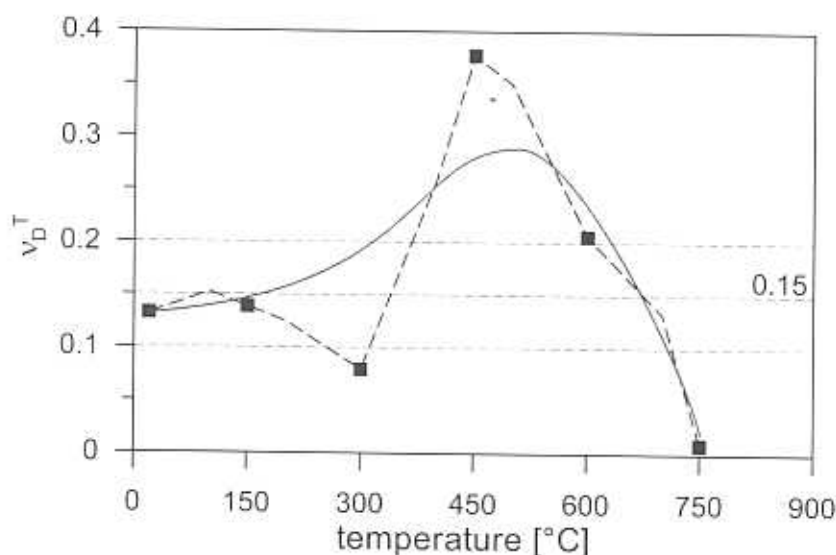


Fig. 6-25: Plot of the dynamic Poisson's ratio obtained from the dynamic moduli E_{CD} and G_{CD} of a high-performance micro-concrete ($f_c = 120$ MPa, Bamonte et al., 2006)

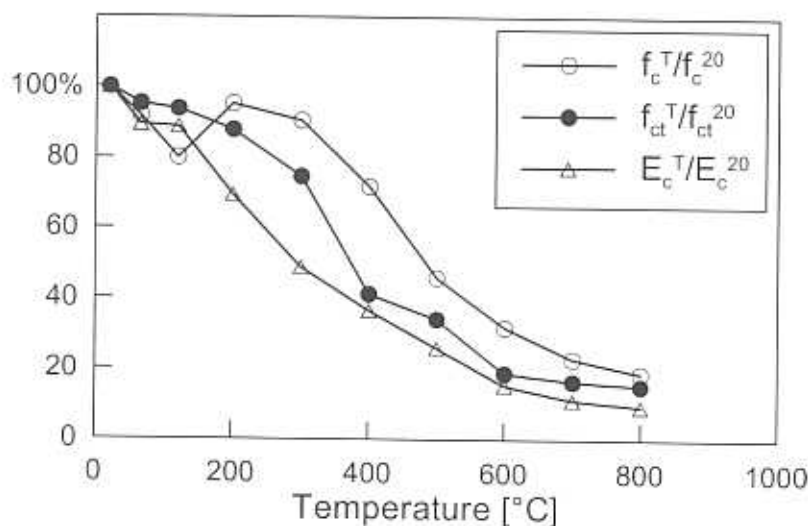


Fig. 6-26: Tests at high temperature by Takeuchi et al. (1993): $\circ$ relative compressive strength; $\bullet$ relative tensile strength; and $\triangle$ relative Young's modulus; $f_c = 40$ (43) MPa, $f_{ct} = 2.7$ (3.3) MPa, and $E_c = 34$ (40) GPa at 20°C, curing at 65% (100%) R.H.

Generally speaking, after cooling the residual strength in tension is:

- marginally higher for silica-fume concretes (direct tension, Toutanji et al., 2003, Fig. 6-27);
- roughly constant for NSC and decreasing with the temperature for silica-fume HPC at medium temperature ($\leq 400^\circ\text{C}$, bending and splitting strength, Balendran et al., 2003, Fig. 6-28a);

- always decreasing at high temperature ($\geq 600^\circ\text{C}$), more in HPC than in NSC, but the higher the temperature, the lower the dependency on concrete strength in compression (Chan et al., 1996, Fig. 6-28b splitting strength, no silica fume; Felicetti and Gambarova, 1999, direct tension and bending strength, with silica fume, Fig. 6-29a);
- decreasing with the temperature (all authors, see also Fig. 6-29a,c,d);
- little affected by the type of cooling (controlled in the furnace and slow, or in water and very quick): water-sprayed specimens may even exhibit a better splitting strength for $f_c < 40$ MPa (Barragán et al., 2001, Fig. 6-30);
- less affected by temperature if natural (river) aggregates are used (Barragán et al., 2001, Fig. 6-30);
- more affected by temperature in direct tension than in bending (Felicetti and Gambarova, 1999, Fig. 6-29b);
- more affected by the temperature in bending tests than in splitting tests, for increasing values of the residual compressive strength (Barragán et al., 2001, Fig. 6-30a,b);
- closer to the hot strength in high-performance silica-fume concrete, than in normal-strength concrete (this is generally accepted, since in the former material there is less portlandite undergoing dissociation during the heating process, and consequently less calcium-dioxide rehydration during the cooling process).

One should also note that the ratio between the strengths in compression and in direct or indirect tension increases with the temperature (Felicetti and Gambarova, 1999, Fig. 6-29b; Barragán et al., 2001, Fig. 6-30; Zhang et al., 2000b), and so does the ratio between the bending strength and the direct strength in tension (Felicetti and Gambarova, 1999, Fig. 6-29b).

6.1.3.6 Fracture parameters

The fracture parameters examined in the following are the specific fracture energy G_f (= energy required to extend a cracked surface by a unit area, $\text{FL}/\text{L}^2 = \text{F}/\text{L}$), the characteristic length (representing the size of the process zone = damaged zone beyond the crack tip and astride the crack, where most of the inelastic phenomena occur and energy is dissipated), and the toughness K_{IC} (= critical stress-intensity factor, representing the resistance of concrete to cracking, $\text{F}/\text{L}^{1.5}$).

As already observed, none of the previous parameters has been so far evaluated during high-temperature tests, because the difficulties associated with such an extremely severe environment make test control hardly feasible. As a consequence, all fracture parameters have been evaluated after cooling to room temperature (residual values).

The first parameter to be discussed is fracture energy, that can be measured in different ways, depending on whether a part of the area or the entire area enveloped by the stress-crack opening curve (direct tension) or by the load-displacement curve (three-point bending) is considered (Fig. 6-31c). Of course, the energy dissipated in the solid material during the loading phase (G_f^0 in Fig. 6-31c) should always be deducted.

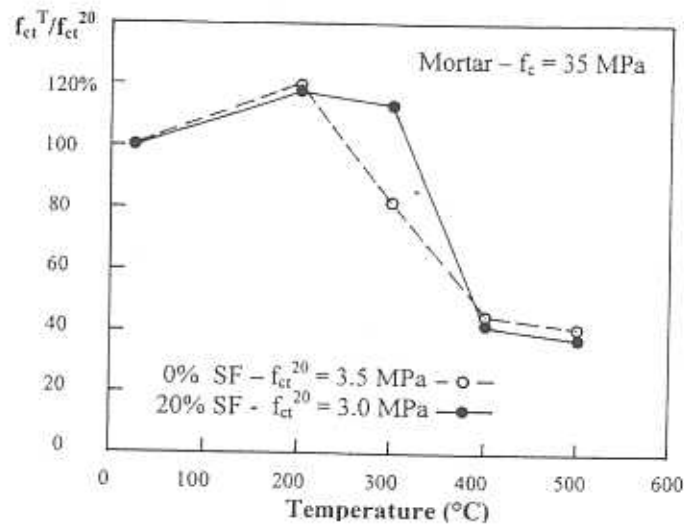


Fig.6-27: Plots of the residual tensile strength of a normal-strength mortar, with/without silica fume ($sf/c = 0.20$, $w/c = 0.33$) as a function of the maximum temperature reached during the heating process (Toutanji et al., 2003)

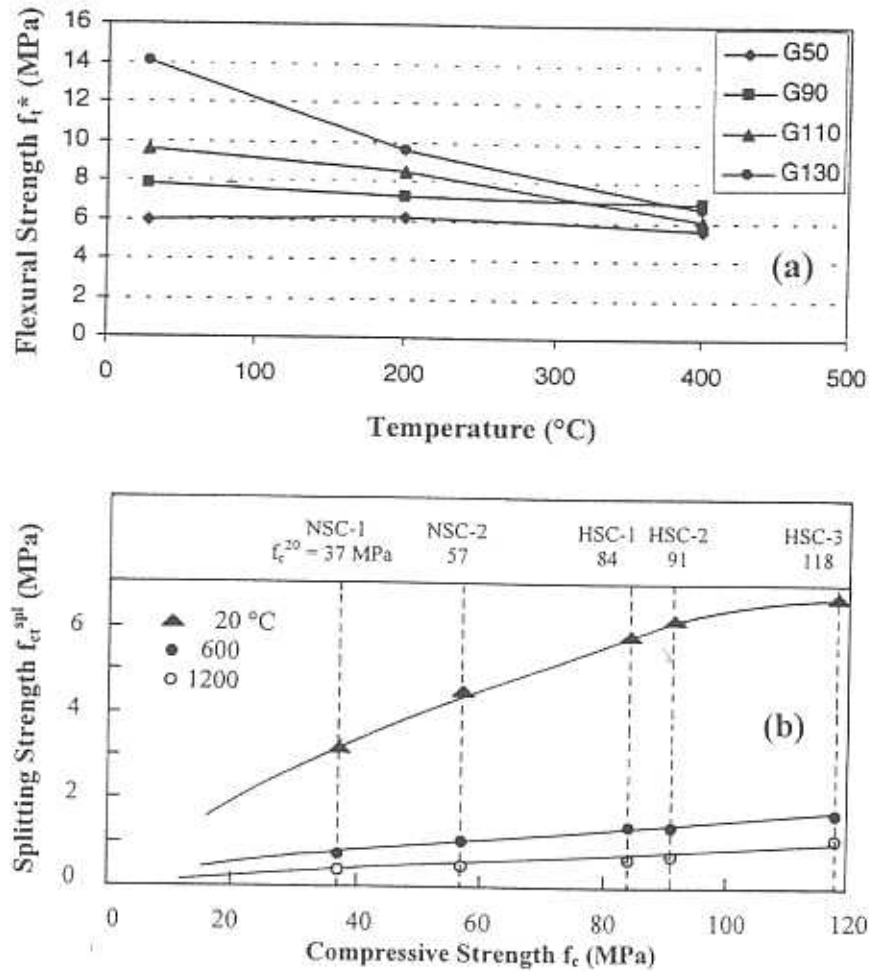


Fig. 6-28: Plots of the flexural and split-cylinder strengths: (a) as a function of the temperature (Balendran et al., 2003); and (b) as a function of the compressive strength (Chan et al., 1996). G50 = NSC with $w/c = 0.50$; G90, 110, 130 = HPC with $sf/c \approx 0.10c$, $w/c = 0.38, 0.33, 0.24$; NSC-1,2 with $w/c = 0.66, 0.57$; HSC-1,2,3 with $w/c = 0.35, 0.31, 0.28$, no silica fume.

The results by Felicetti and Gambarova (1999) are considered first, because they are well documented, even if they refer to two uncommon high-performance concretes, containing highly-siliceous aggregates (flint gravel and pebbles; $f_c = 72$ and 95 MPa; cement content $c = 290$ and 415 kg/m³; silica fume/cement = 9.4% and 6.7% ; water/binder ratio = $w/sf+c = 0.43$ and 0.30 ; with/without calcareous filler; maximum aggregate size $d_a = 25$ mm; $T = 105, 250, 400$ and 500°C ; $\Delta T/\Delta t = 0.5^\circ\text{C}/\text{min}$; rest at the maximum temperature 12 hours; Fig. 6-31).

The residual fracture energy appears to be roughly independent of the temperature, both in direct tension and in bending tests (Figs. 6-30a,b). However, the results are very scattered, and some interaction between the process zone and the top surface of the specimen cannot be ruled out for the largest value of the notch depth. The toughness is a decreasing function of the temperature, while the characteristic length (according to Hillerborg) tends to increase above 200°C (Fig. 6-31d). Summing up, the material becomes more brittle, but the damage is more diffused and the damaged volume increases. In the end the material becomes more ductile and so the structural behavior, even if further studies are needed with reference to the structural behavior. As for the role of concrete grade, there are apparently no major differences in the two concretes tested in this project.

The previous results are confirmed by Barragán et al. (2001) for $f_c < 40$ MPa, while for $f_c > 50$ MPa it appears that the fracture energy first increases and then decreases (Fig. 6-32a).

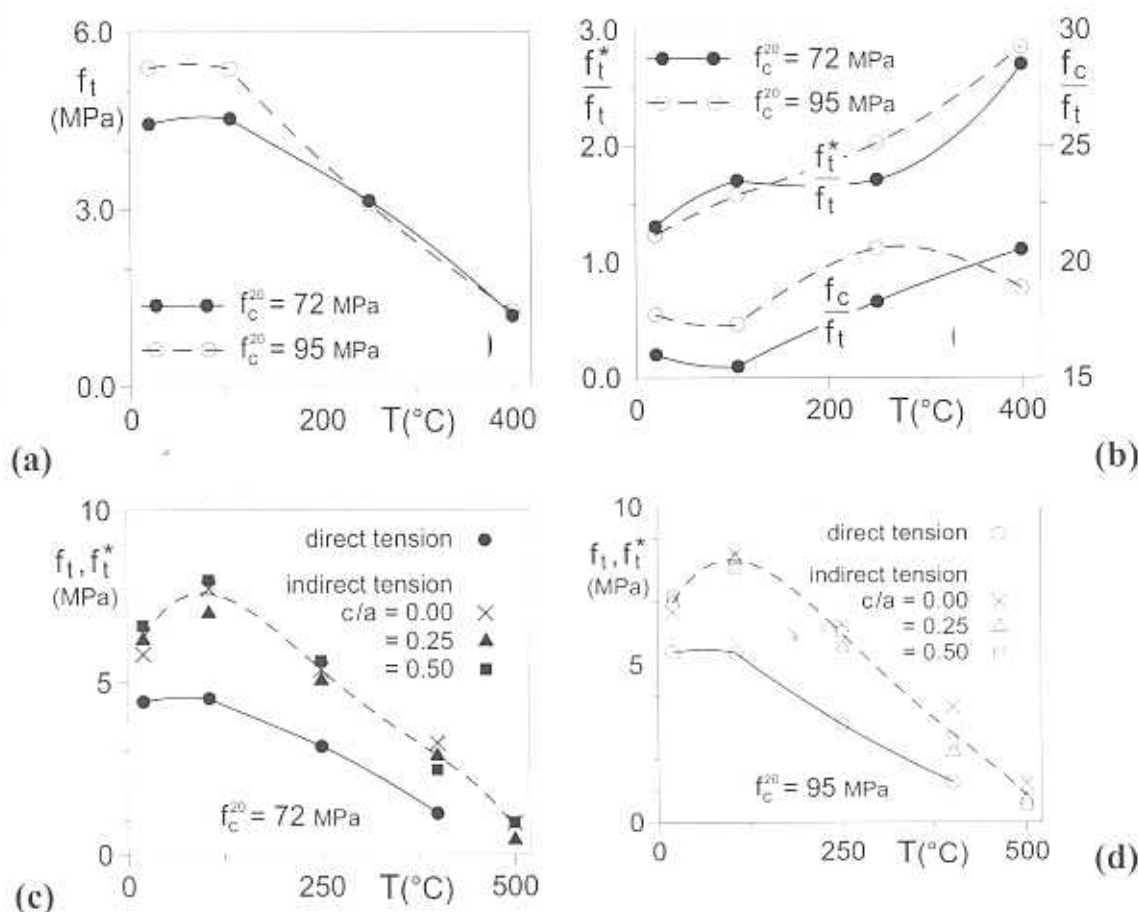


Fig. 6-29: Direct tension, notched cylinders, and indirect tension, notched and unnotched prisms: (a) plots of the residual strength; (b) plots of two strength ratios; and (c,d) plots for different notch depths (f_c = compressive strength; $f_t = f_a$ = tensile strength; f_t^* = indirect tensile strength in three-point bending); highly-siliceous concretes (Felicetti and Gambarova, 1999)

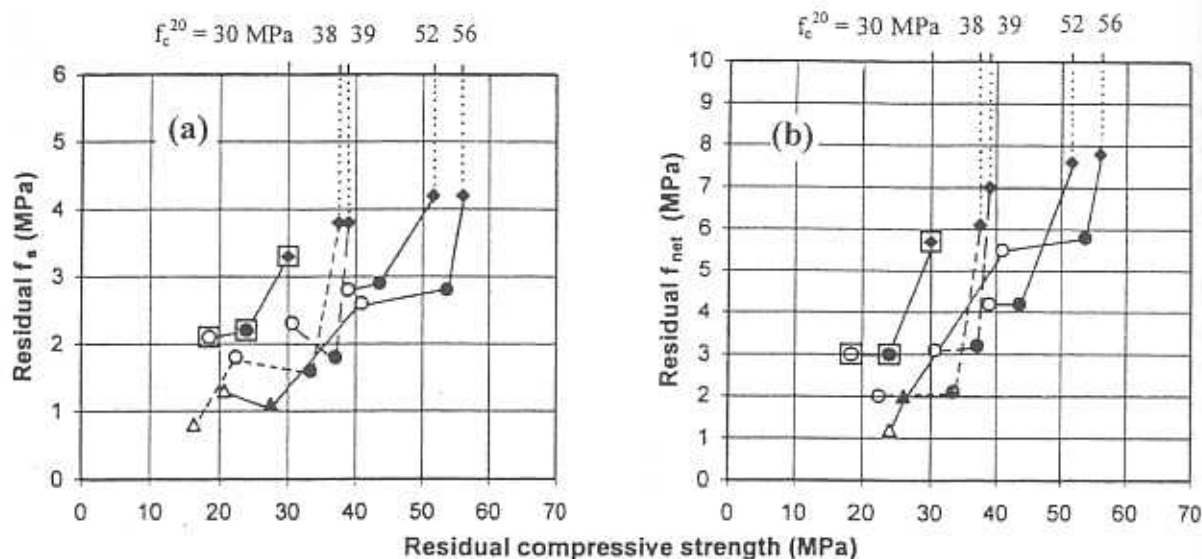


Fig. 6-30: Indirect tension: plots of the residual strength measured (a) on cubes subjected to splitting ($f_s = f_{ct}^{sp}$), and (b) on notched prisms loaded in three-point bending ($f_{net} = f_t^* =$ modulus of rupture): ♦ control concretes (virgin materials); ●/○ (△/▲) cooling from 500°C (700°C): ● ▲ controlled inside the furnace (slow cooling), and ○ △ under sprayed water (quick cooling); coarse aggregates: ● ○ ▲ △ ⇒ crushed granite; boxed values ⇒ natural (river) gravel; the connecting lines identify the tests concerning the same concrete mix-design (Barragán et al., 2001).

This latter tendency is confirmed by Zhang et al. (2000a) – Fig. 6-33a – who tested two concretes (NSC with $f_{cu} = 57.4$ MPa and HPC with $f_c = 77.6$ MPa at 28 days, rapid-hardening Portland cement, natural aggregates, concrete age 14 days). As a result of the further hydration of the cement paste and of the strengthening of the aggregate-mortar interface, fracture energy markedly increases up to 300°C in both NSC and HPC, but then starts decreasing, because of the unfavorable effects of microcracking, dehydration and chemical decomposition. The same was found by Felicetti et al. (2000) with reference to ultra high-performance concretes (CRC and RPC, Fig. 6-33b), where the sizable amount of steel microfibers increased the fracture energy by two orders of magnitude with respect to a typical high-performance concrete. However, the HPC still showed a markedly constant fracture energy at the various temperature levels (Fig. 6-33b).

As for concrete toughness, there is a general consensus on the loss of toughness accompanying the exposure to high temperature (Felicetti and Gambarova, 2000, Fig. 6-31d; Zhang and Bicanic, 2002, Fig. 6-34a; Hamoush S.A. et al., 1998).

Finally, the characteristic length increases with the temperature, in such a way that the larger the strength, the larger the characteristic length (Felicetti and Gambarova, 2000, Fig. 6-31d; Zhang et al., 2000a, Fig. 6-34b), and the quicker the cooling, the larger the characteristic length, because of the greater damage that softens the material (Barragán et al., 2001, Fig. 6-32b; Zhang et al., 2000a). According to Hillerborg's formulation, the characteristic length is a comprehensive brittleness parameter, which brings in the fracture energy, the tensile strength and the elastic modulus:

$$l_{ch} = G_f E_c / f_{ct}^2 \quad (6-1)$$

Since the characteristic length is a measure of the damaged volume, the larger the characteristic length, the more diffused the damage and the less brittle the concrete.

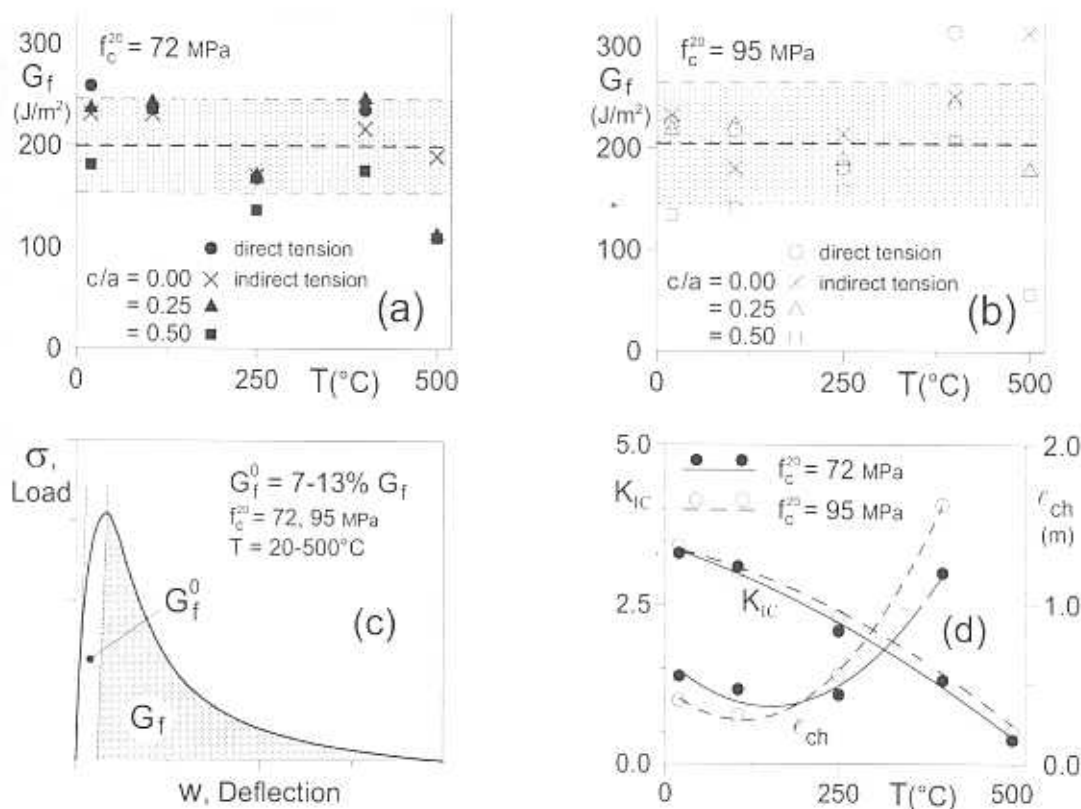


Fig. 6-31: Direct and indirect tension tests: (a,b) fracture energy G_f as a function of the temperature, with $G_f^{AV} = 200 \pm 45$ N/m (a) and $G_f^{AV} = 205 \pm 60$ N/m (b); (c) definition of the fracture energy; and (d) plots of the toughness K_{IC} and of the characteristic length l_{ch} as a function of the temperature: $K_{IC} = (G_f E_c)^{1/2}$ and $l_{ch} = G_f E_c / f_{ct}^2$; c/a = notch depth-to-section depth ratio (Felicetti and Gambarova, 1999).

Summing up, exposing concrete to high temperature makes the material more cracking-prone (the toughness decreases), but the overall behavior becomes more ductile, since the damage is more diffused (the characteristic length increases), as a result of the increasing fracture energy (at least up to medium temperatures) and of the decreasing tensile strength. The variations of the last two parameters offset the decreasing elastic modulus and increase the characteristic length, more in high-grade than in low-grade concretes, since – as it is well known – the tensile strength does not keep up with the compressive strength, at any temperature.

6.1.3.7 Stress-strain curves

As shown in the previous sections, most of the research projects concerning concrete high-temperature behavior has been focused on a number of specific properties, like the tensile and compressive strengths, the elastic modulus and – more recently – the fracture energy. However, starting from the late sixties and early seventies of the past century, an increasing number of studies has been devoted to the measurement of concrete complete stress-strain curve at high temperature see Purkiss (1996), where the results by Furumura (1966,1987), Baldwin and North (1973), Popovics (1973), Schneider (1980), and Purkiss and Bali (1988) are quoted; see also RILEM (1985), and Hager and Pimienta, 2005].

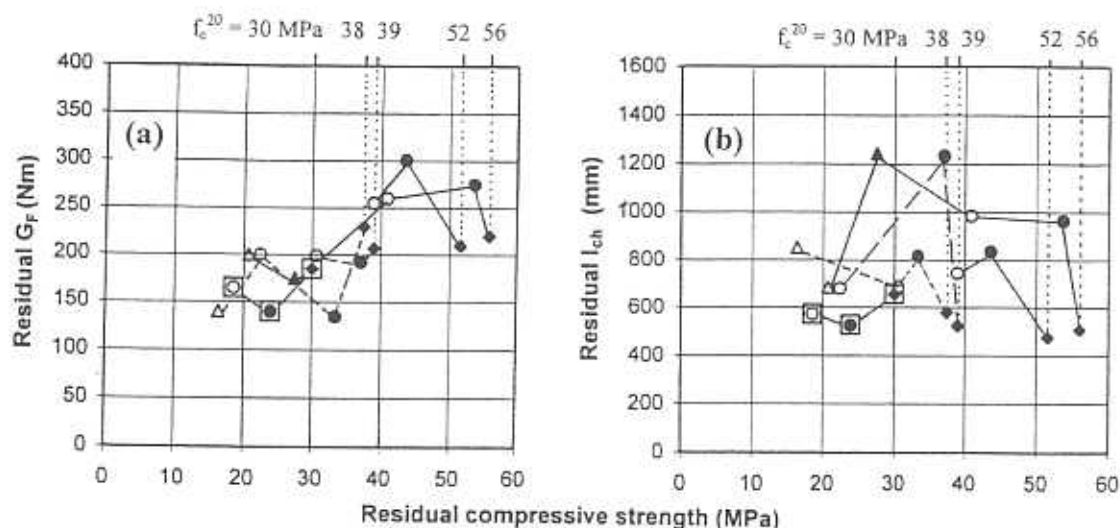


Fig. 6-32: Residual fracture energy (a) and residual characteristic length (b) for various normal-strength concretes (Barragán et al., 2001); ♦ control concretes (virgin materials); ●/○ (Δ/▲) cooling from 500°C (700°C); ●Δ controlled inside the furnace (slow cooling), and ○Δ under sprayed water (quick cooling); coarse aggregates: ●○Δ▲ ⇒ crushed granite; boxed values ⇒ natural (river) gravel; the connecting lines identify the tests concerning the same concrete mix-design.

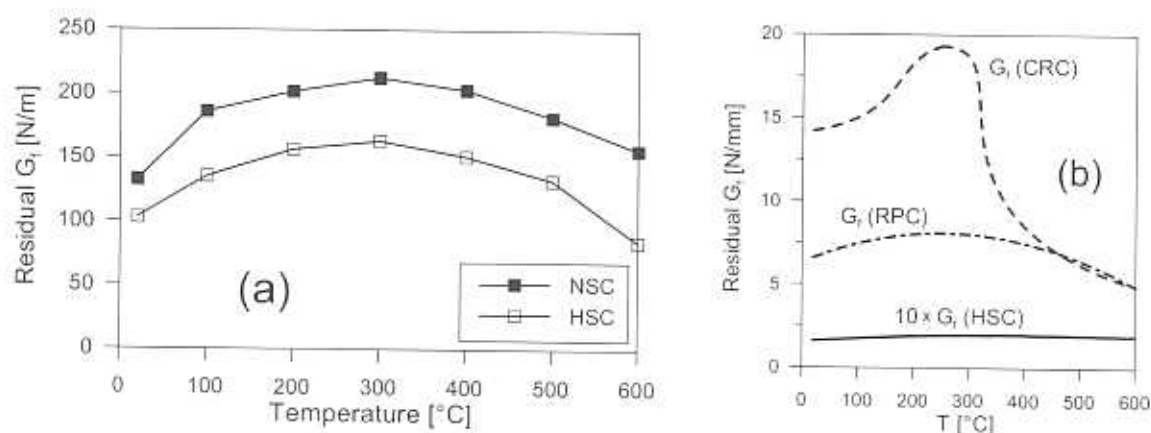


Fig. 6-33: Fracture energy for different heating temperatures: (a) NSC and HPC at 15 days ($f_{cu} = 57$ and 78 MPa at 28 days, no silica-fume, w/c = 0.54 and 0.30, see Zhang et al., 2000a; and Zhang and Bicanic, 2002); and (b) HPC and UHPC at 90-120 days ($f_c = 92$ and 158-165 MPa, see Fig. 6-26).

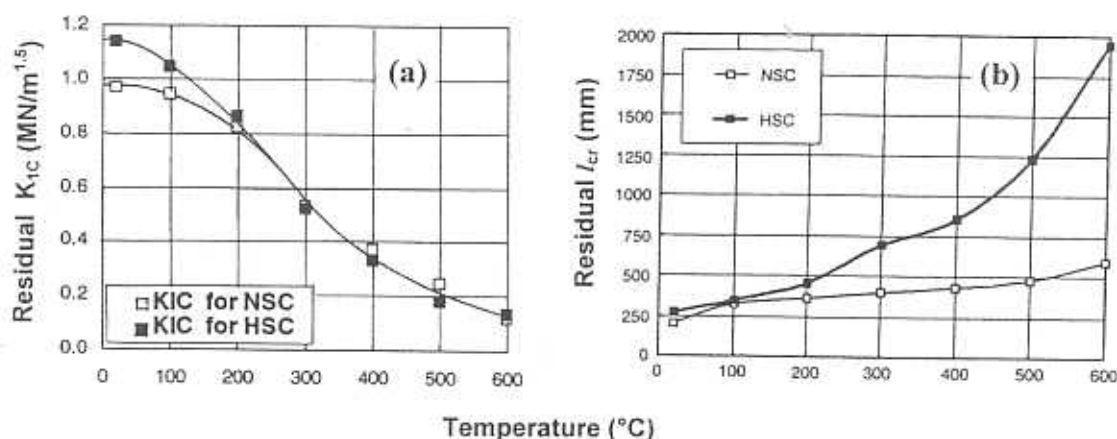


Fig. 6-34: Plots of the toughness (a) and of the characteristic length (b) for different heating temperatures (Zhang and Bicanic, 2002; Zhang et al., 2000a)

Because of the limited toughness of plain concrete (any plain concrete!), the tests had – and have – to be carried out under controlled displacements, which is very demanding at high temperature, at least at low-medium temperatures when the material is still very brittle ($T_{\max} < 400^\circ\text{C}$). Of course testing in residual conditions is much easier, since the specimen is instrumented, loaded and monitored at room temperature (a thermally-damaged concrete is like a virgin concrete with a lower strength and stiffness).

In compression, the stress-strain curve is generally obtained by using the circumferential strain as the feedback parameter to control the test (Jansen et al., 1995; Taerwe, 1992; Hsu and Hsu, 1994). As a matter of fact, this strain is an increasing function throughout the whole test, while the same is not true for the longitudinal strain, unless the material is highly damaged (for $T \geq 400^\circ\text{C}$ the material softens and becomes ductile, Felicetti and Gambarova, 1998).

In direct tension, since notched specimens are generally used, the attention is focused on the relationship between the mean stress in the notched section and the relative displacement at the notch mouth. This displacement – called CMOD (Crack Mouth Opening Displacement) – tends to coincide with the CTOD (Crack Tip Opening Displacement), once the crack starts at the tip of the notch and propagates to the whole notched section. The complete stress-CMOD or stress-CTOD curves are instrumental in evaluating the fracture energy, the stress-strain curve in direct tension and the “critical” crack opening (when there is no stress transfer across the crack), that are often requested by FE codes (see for instance Felicetti and Gambarova, 1999).

In the following, the attention is mostly focused on concrete residual behavior in compression, since for the behavior in tension there is no need of very realistic curves and rather simplified curves are generally adopted, on the basis of few parameters (like the tensile strength and the fracture energy).

According to Furumura, Baldwin and North (see Purkiss, 1996), the stress-strain curve of any concrete can be put in the following non-dimensional form:

$$\sigma_c/\sigma_{cl} = (\varepsilon_c/\varepsilon_{cl}) \exp [1 - (\varepsilon_c/\varepsilon_{cl})] \quad (6-2)$$

where σ_c and ε_c are the actual stress and strain, and σ_{cl} and ε_{cl} are the stress and strain corresponding to the peak of the curve.

A more general formulation was introduced by Popovics (see Purkiss, 1996).

$$\sigma_c/\sigma_{cl} = (\varepsilon_c/\varepsilon_{cl}) \{ n / [n - 1 + (\varepsilon_c/\varepsilon_{cl})^n] \} \text{ with } n = [1 - \sigma_{cl}/(\varepsilon_{cl} E_c)]^{-1} \quad (6-3)$$

The parameter n is – unfortunately – a highly-variable parameter (from 2 to 7), that depends on concrete compressive strength, aggregate size, aggregate-cement ratio, and – more generally – on any parameter affecting concrete non-linearity, like – for instance – the preload in the case of fire-exposed structures.

In EC2 – Fire Design the stress-strain curve at high temperature is given the formulation introduced by Schneider in the early eighties (see RILEM, 1985):

$$\sigma_c/f_c^T = 3\varepsilon_c/\{\varepsilon_{cl}^T[2 + (\varepsilon_c/\varepsilon_{cl}^T)^3]\} \text{ for } \varepsilon_c \leq \varepsilon_{cl}^T \quad (6-4)$$

where: ε_{cl}^T (strain at the peak stress) =

$$2.5 \times 10^{-3} + 4.1 \times 10^{-6} (T - 20) + 5.5 \times 10^{-9} (T - 20)^2 \leq 10^{-3}.$$

As for the softening branch, a linear or nonlinear formulation can be adopted, with the values of the ultimate strain ε_{cu}^T ranging from 2% to 4% ($T = 200 - 800^\circ\text{C}$, Franssen, 2004, Fig. 6-35).

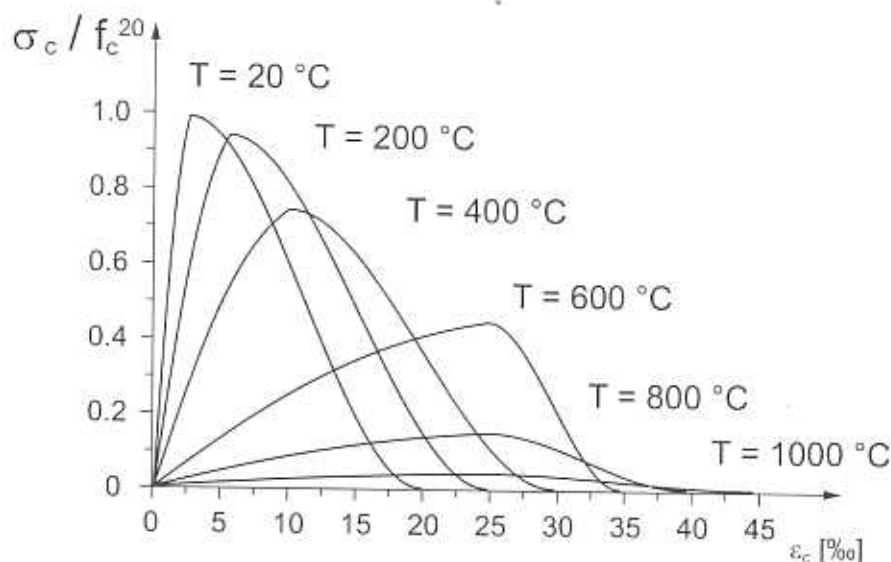


Fig. 6-35: Stress-strain curves for concrete at high temperature (EC2 – EN 1992-1-2 – Fire Design): as a first approximation, the residual curves may be obtained from the high-temperature curves, by means of a 15-20% reduction of the peak stress f_c^T .

In Figs. 6-36a,b two sets of curves are plotted for a highly-siliceous concrete, in the virgin state and after a thermal cycle at 500°C ($f_c = 72$ MPa, 3 nominally-identical specimens in each case, Felicetti and Gambarova, 1998). For the same concrete, Fig. 6-36c shows the mean stress-strain curves, while Fig. 6-36d refers to a closely-related concrete, with a higher compressive strength. Note that there are no major differences between the first concrete - lean mix, $c = 290$ kg/m³, calcareous filler = 105 kg/m³ - and the second concrete with $f_c = 95$ MPa (rich mix, $c = 415$ kg/m³, no calcareous filler).

For the above-mentioned concretes ($f_c = 72$ and 95 MPa), the stress-CMOD curves are shown in Fig. 6-36 (Felicetti and Gambarova, 1999). The reduction of the softening branch is remarkable indeed (the larger the temperature, the more elastic-plastic the behavior of the material, with no major differences between the two mixes).

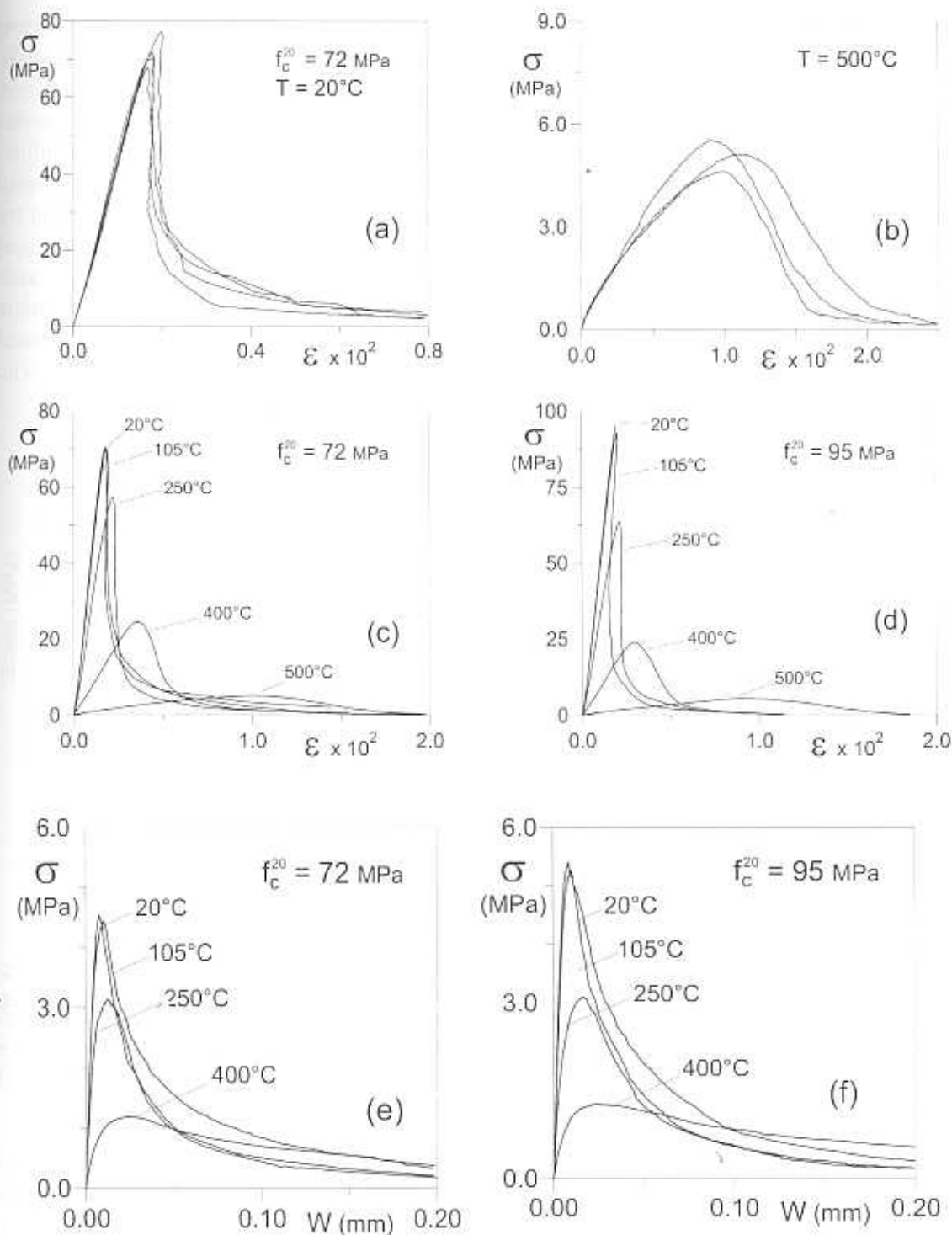


Fig. 6-36: Compression - Stress-strain curves of two high-performance, siliceous concretes: (a,b,c) lean mix, virgin state, after a cycle at 500°C and mean curves; and (d) rich mix, mean curves (see also Figs.11 and 29, Felicetti and Gambarova, 1998); (e,f) Direct tension - Stress-CMOD curves of the previous concretes, in the virgin state (20°C) and after a thermal cycle ($T = 105, 250$ and 400°C); notched specimens: $\varnothing = 100$ mm; $\varnothing_N = 84$ mm; $h = 150$ mm (Felicetti and Gambarova, 1999).

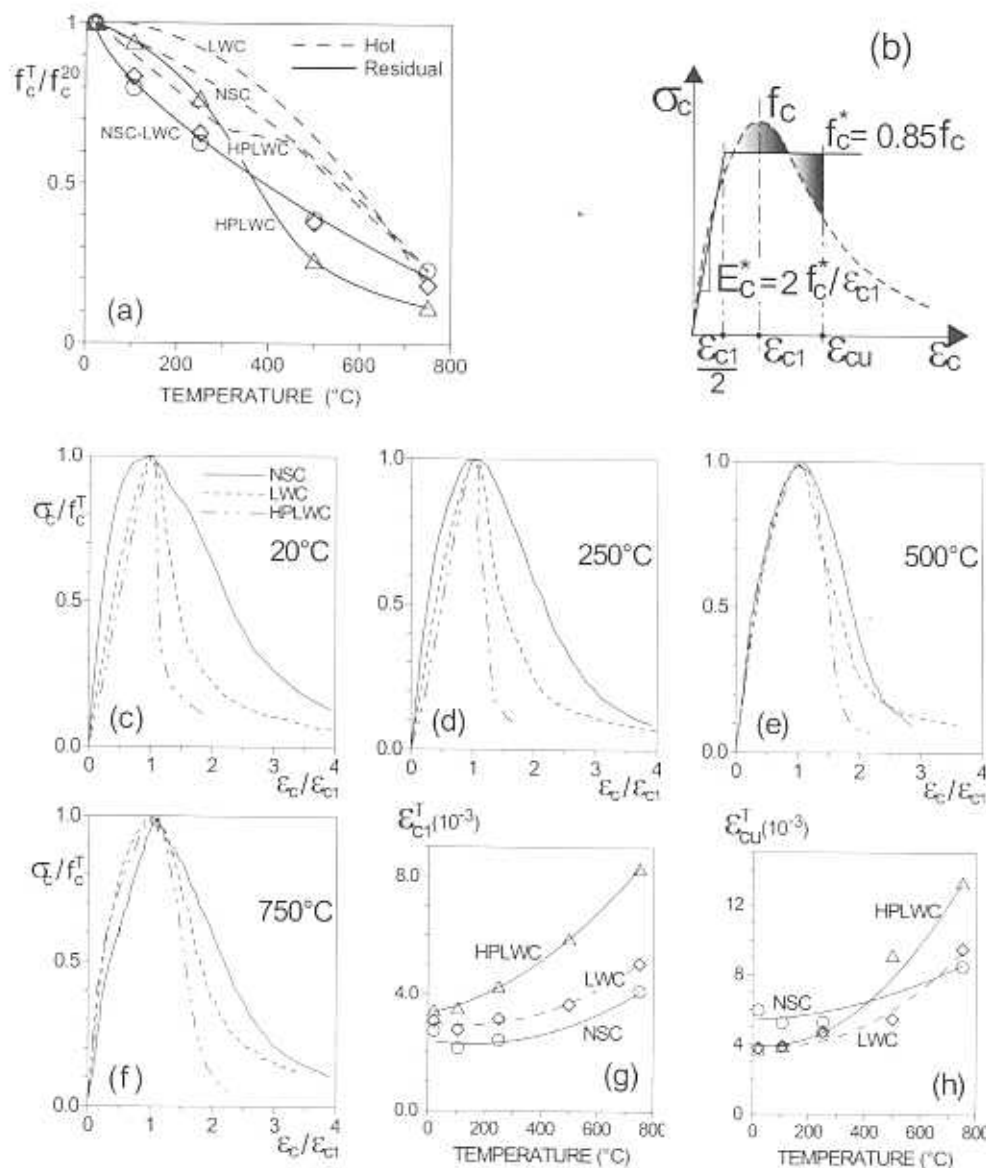


Fig. 6-37: Compression – NSC, LWC and HPLWC: (a) residual compressive strength as a function of the temperature; (b) bilateral idealization; (c-f) non-dimensional curves; and (g,h) plots of the strains (see also Fig. 6-23; $f_c = 30, 39$ and 56 MPa; $c = 300, 350$ and 500 kg/m³; $\rho_c = 2309, 1809$ and 1920 kg/m³, Felicetti et al., 2002)

Going back to compression, the stress-strain curves of two normal-strength concretes (with ordinary and light-weight aggregate respectively) and of one high-performance concrete (with light-weight aggregate) are shown in Fig. 6-37. In Fig. 6-37a the compressive strength is plotted as a function of the temperature, while in Fig. 6-37b the bilateral simplification is sketched, together with the strain at the peak stress (ϵ_{c1}) and the ultimate strain (ϵ_{cu}).

The gain in terms of ductility after a thermal cycle, the loss in terms of compressive strength and the advantages offered by the passive confinement exerted by the ties are highlighted by the test results of Bo Wu et al. (2002), as can be seen in Fig. 6-38 ($f_c = 70$ MPa, silica fume = 12% cement; cement = 500 kg/m³, water/binder ratio = 0.27). Prismatic specimens made of plain and reinforced concrete were tested in virgin conditions and after a cycle at 100-900°C ($\Delta T = 100^\circ\text{C}$; prism length and side $h, a = 315$ and 100 mm). The specimens of Group A had no reinforcement, while prisms of Groups B and C had 5 and 9 square ties respectively (tie diameter $\varnothing_t$ and sectional area $A_t = 6$ mm and 28.3 mm²; tie spacing $s_t = 75$ and 37 mm; total steel ratio $\rho_t = 4A_t/s_t a = 1.5\%$ and 3.0% respectively). The

specimens of Groups B and C had also 4 longitudinal bars at the corners, to keep up the ties ($\varnothing_L = 6$ mm). In Fig. 6-38 only the stress-strain curves of groups B and C are shown, both in their dimensional (Fig. 6-38a,b) and non-dimensional form (Figs. 6-38c,d). From the latter curves, what clearly appears is that the descending branches become flatter when the confinement is increased, but there is not much difference between the two confinement ratios. Furthermore, looking at the non-dimensional curves, the effects of the confinement are not univocal. (It should be noted that the non-dimensional curves are useful to identify the general trends of both the ascending and the descending branches, but are rather misleading since the ductility is “hidden” in the strain at the peak stress, this strain being used to put the strains in a relative form). However, in spite of the rather large scatter, the general trend of the descending branches is pretty much the same for any temperature. In Figs. 6-38c,d, the full curves represent the simple relationships proposed by the authors.

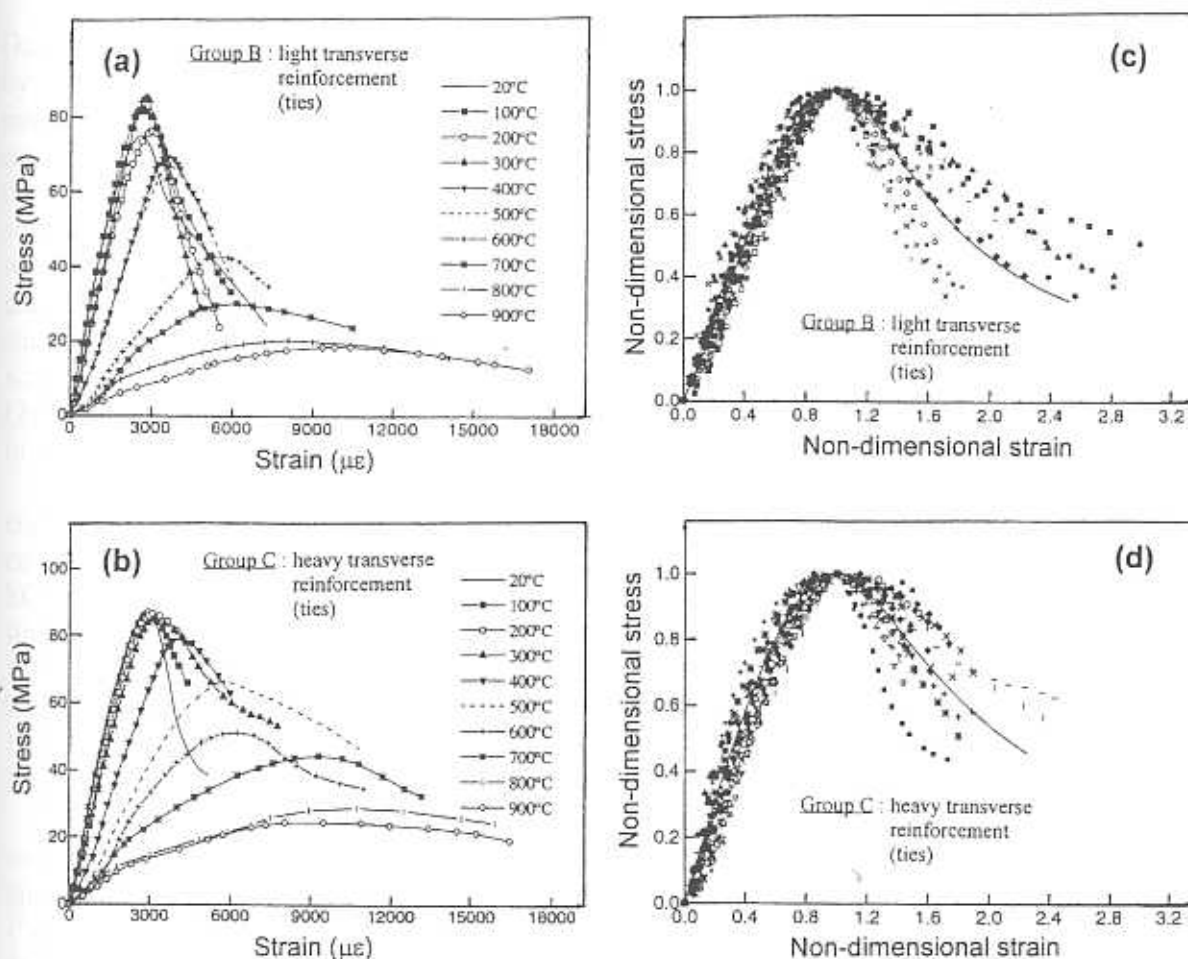


Fig. 6-38: Compression – Stress-strain curves of transversely-reinforced prisms (Bo Wu et al. (2002): (a,b) dimensional curves, and (c,d) non-dimensional curves

Finally, the critical issue to what extent the “residual” behavior in compression is different from the “hot” behavior should be addressed, since recent results are rather contradictory at least in the case of high-performance cementitious composites with small aggregate particles. The test results by Gawęska-Hager (2004) show that for a concrete with $f_c = 90$ MPa ($c = 377$ kg/m³, s.f. = 10%, w/b = 0.30, calcareous aggregate) the residual behavior is generally worse in terms of strength and elastic modulus (Figs. 6-39a,b), while other results – limited to the compressive strength (Fig. 6-39c, Bamonte et al., 2006) – show that above 250°C the two strengths are very close or even coincident. However, it is fair to say that the concrete tested

by Bamonte et al. is a very-high strength micro-concrete ($f_c = 120$ MPa, $c = 635$ kg/m³, $w/c = 0.31$, siliceous aggregate) without silica fume, while the concrete investigated by Gawęska Hager is one of the concretes studied within the Project BHP 2000. To explain the above contradictions, one should remember that there are at least 3 reasons why the residual behavior tends to be worse than the hot behavior: (a) possible reversed thermal gradients during the cooling phase, (b) lack of transient thermal creep and lack of compatibility between the thermal strains in the aggregate and in the mortar, and (c) re-hydration of calcium oxide to form calcium hydroxide (portlandite) accompanied by a volume increase.

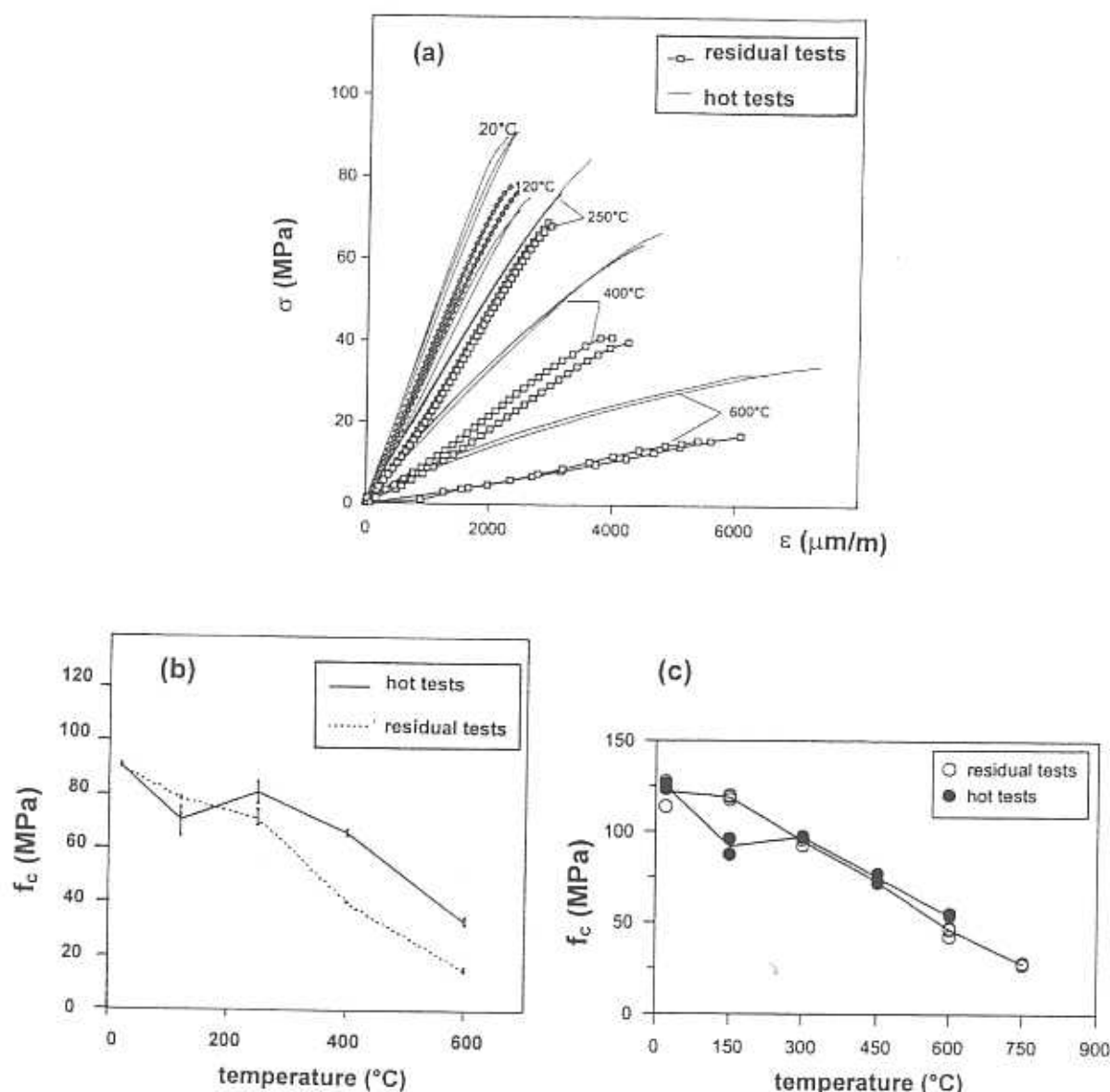


Fig. 6-39: Compression – hot and residual tests: (a,b) by Gawęska Hager (2004) on concrete M100C ($f_c = 90$ MPa, $c = 377$ kg/m³ - CEM I 52.5R, s.f. = 10% c , $d_a = 20$ mm, calcareous aggregate, $v_f = 0.1\%$ by volume, polypropylene fibers, $w/b = 0.30$); and (c) by Bamonte et al. (2006) on a micro-concrete ($f_c = 120$ MPa, $c = 635$ kg/m³ - 52.5R, no s.f., $d_a = 4.5$ mm, siliceous aggregate, $v_f = 0.6\%$ by volume – macro- and micro-polypropylene fibers, $w/b = w/c = 0.31$)

The lack of calcium content and of thermal gradients in the specimens tested by Bamonte et al. (because of the limited size of the cylinders - $\varnothing = 36$ mm - and of the very low cooling rate) give a partial explanation of the better results obtained in Milan, but two other factors come into play as well: the very homogeneous microstructure ($d_a = 4.5$ mm), that favors

mortar-to-aggregate strain compatibility, and the lack of silica fume, since there is a general consensus on the unfavorable – but still not completely clarified – effects that silica fume has on concrete high-temperature behavior (Cheyrezy, 2001).

Summing up, high-performance concretes with mixed aggregates seem to have a residual behavior which is worse than that at high temperature, as usually occurs in normal-strength concretes, while some results indicate that very small aggregate particles accompanied by a high cement content reduce or even nullify the gap between the residual and the hot behavior in compression.

6.1.4 Recent developments

6.1.4.1 Self-compacting concrete

One of the major success stories in concrete research and applications is certainly represented by Self-Compacting Concrete – SCC, since no energy is required to compact the material, to envelope the reinforcement and to fill the formwork up to most hidden nook. Because of its astonishing workability, SCC is particularly suitable for any structural member requiring high durability (homogeneity, permeability, chemical resistance), which is typical of the structures exposed to severe conditions (like – for instance – those in contact with soil and water, or in an industrial environment, like dams, tunnels, piers, tanks, bridges, slabs on grade, roadways and runways). While many properties related to SCC's durability have been studied in the past 10 years (see references in Persson, 2004), fire resistance have received scanty attention, and mostly for spalling (Jansson and Boström, 2005). The paper by Persson (2004) is still the most authoritative in terms of fire resistance, and its conclusions are recalled in the following.

Persson investigated the hot and residual behaviors of 12 SCCs and 4 VCs (vibrated concretes), with a compressive strength ranging from 40 to 88 MPa. Four SCCs and one VC contained Portland cement ($c = 411\text{--}518 \text{ kg/m}^3$; $w/c = 0.40$; $f_c = 75\text{--}88 \text{ MPa}$), while eight SCCs and three VCs contained blended cement (the clinker was partly replaced with a limestone filler = 15% by clinker mass; $c = 268\text{--}447 \text{ kg/m}^3$; $w/c = 0.40\text{--}0.70$; $f_c = 40\text{--}80 \text{ MPa}$). In two SCCs (one with portland cement and one with blended cement) the mix included also glass powder ($gp = 0.15c$ and $0.25c$, respectively), while in ten SCCs the mix included limestone powder ($lp = 0.35c$ and $0.63c$ in the concretes containing portland and blended cement respectively). The cement grade was 42.5 R in all cases.

The reference temperatures were 20, 200, 400, 600 and 800°C; as a rule, the heating and cooling rates were 4 and 1°C/minute respectively; at the reference temperature, all cylindrical specimens rested for ½ h, before being tested in compression ("hot" tests) or being cooled ("residual" tests). The residual tests were carried out one week after cooling. In normal environmental conditions, adding 1 kg/m³ of limestone or glass powder increased the compressive strength by 0.07 MPa, most probably because of the better packing of the fine particles (however, the cement content in SCC was generally lower than in VC).

In Fig. 6-40 the mean residual properties of the 10 self-compacting limestone-concretes tested by Persson are plotted in a non-dimensional form, as a function of the reference temperature. For the sake of comparison, also the strength decays of calcareous and siliceous concretes suggested by EC-2 are plotted.

On the basis of the results obtained by Persson, the following conclusions can be drawn:

- The residual-strength decay in compression is practically nil up to 200°C and then tends to become a linear function of the temperature, very similarly to vibrated concrete; furthermore, the residual compressive strength is smaller than the hot

compressive strength (not shown here), but the difference seems to be less pronounced than in vibrated concrete and tends to vanish at high temperature ($T \geq 600^\circ\text{C}$).

- The residual modulus is more affected by the temperature than the residual compressive strength, as in vibrated concrete; furthermore, the hot and residual moduli are practically coincident for $T \leq 600^\circ\text{C}$, while at higher temperatures the residual modulus tends to decrease more rapidly and is practically nil at 800°C .
- The residual strain at the peak stress tends to increase more than linearly with the temperature, but less than the hot strain (not shown here) up to 500°C ; on the contrary, the hot strain is larger below 500°C , but then peaks at $600\text{--}700^\circ\text{C}$ and later starts decreasing.

Persson concludes that on the whole SCC and vibrated concrete behave pretty much in the same way at high temperature and after cooling. However, with reference to HPC, the elastic modulus seems to be more affected by high temperature in SCC than in VC.

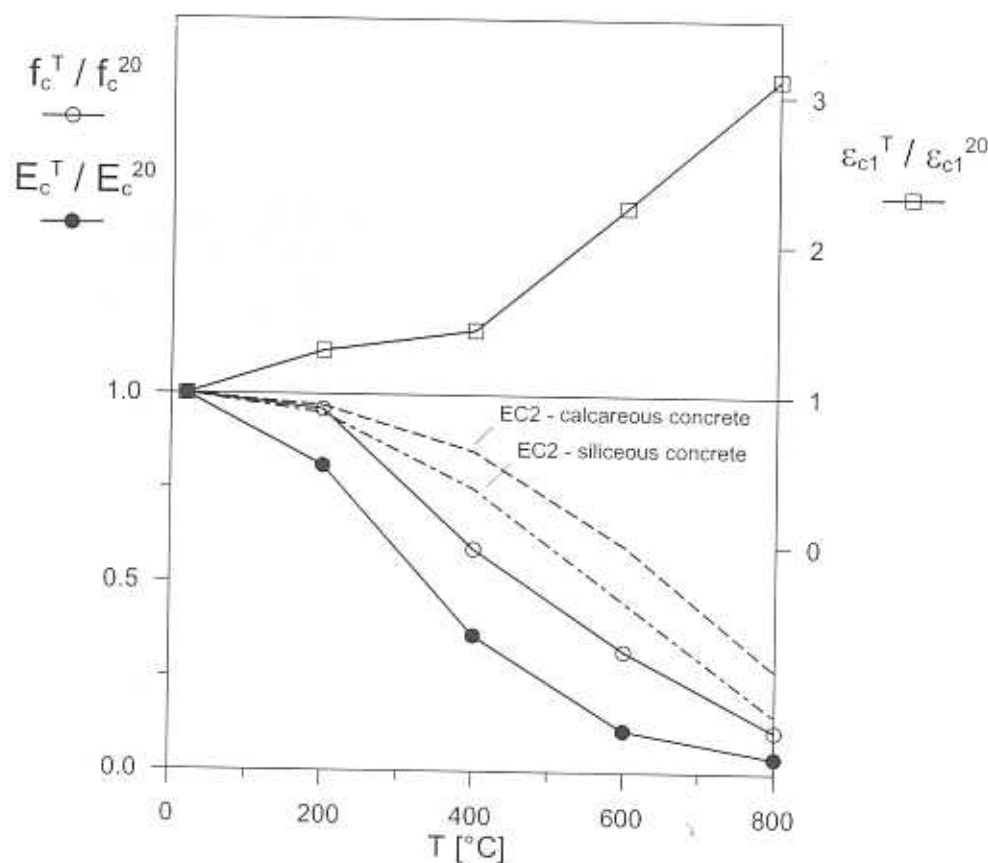


Fig. 6-40: Mean residual properties of the 10 limestone concretes tested by Persson (2004), and strength decay proposed in EC2; ϵ_{c1}^T is the strain at peak stress.

The conclusions by Persson are mostly confirmed by some very recent studies, that show a renewed interest for SCC subjected to high temperature ($f_c = 25\text{--}90\text{ MPa}$): the papers by Reinhardt and Stegmaier (2006; standard fire, hot and residual tests); by Noumowé et al., and by Sideris (2006 and 2007, respectively; steady thermal conditions, residual tests); and by Bamonte et al. (2008; steady thermal conditions, hot and residual tests) are definitely a step forward in the direction of a better understanding of SCC's thermal and mechanical behaviour during and after the exposure to high temperature

6.2 Non-destructive test techniques for concrete

6.2.1 Introduction

As already declared in the above chapters, Fire Design has to do not only with the structural behavior at high temperature, during a fire, but also with the residual behavior, when the safety of the damaged structure has to be checked, in order to define the best strategy to repair and/or strengthen the structure, as an alternative to its demolition.

Within this context, any experimental and non-destructive method fit for giving information on the maximum temperature locally reached inside the concrete and the bars, on the local damage, on the residual strength and stiffness is welcome.

In the following, starting from the well-known paper by Schneider (1990), most of the non-destructive techniques developed so far are recalled and commented.

Further experimental results are shown, as an extension of those presented by Schneider (1990), and new techniques are cited, with reference to the following list:

- new results on the rebound hammer test;
- further data on Ultrasonic Pulse Velocity;
- methods for the interpretation of the indirect UPV technique;
- pull-out test (Capo-test);
- concrete colorimetry;
- digital image analysis of microcracks;
- sonic methods (e.g. MASW); impact echo);
- resistivity;
- coring and destructive perforation;
- georadar.

A detailed treatment of the Non-Destructive Testing techniques (NDT) is in progress in the form of a report by RILEM TC "INR" (Interpretation of NDT results, and assessment of R/C structures), where all possible techniques are presented and commented, including those aimed at the special case of thermally-damaged structures.

6.2.2 General remarks on concrete testing after a fire

Concrete is known to exhibit a good behavior at high temperature, thanks to its incombustible nature and its low thermal diffusivity, which guarantee a slow propagation of thermal transients within the structural members. As a consequence, the reinforcement cover experiences very strong thermal gradients during a fire and the material thermal damage rapidly decreases from a maximum to nil within a few centimeters depth (Fig. 6-41). Only in the case of quite a long fire duration and relatively small cross sections the thermal damage could deeply affect the integrity of the structural members (Fig. 6-41b). For this reason, most of the RC structures surviving a fire still keep a part their load capacity, and their assessment is of prime interest for planning any strengthening or repair.

In the occurrence of spalling, the loss of concrete during a fire has the effect of exposing deeper layers of concrete to the maximum fire temperature, thereby increasing the rate of transmission of heat to the inner layers of the structure. Nevertheless, in some cases the build-up of vapor pressure triggers the explosive expulsion of concrete chips. This phenomenon usually take place within a relatively low temperature range ($< 400^{\circ}\text{C}$) and then the material

remaining after the fire might have not experienced a significantly high temperature (as in the case of the "Chunnel" fire).

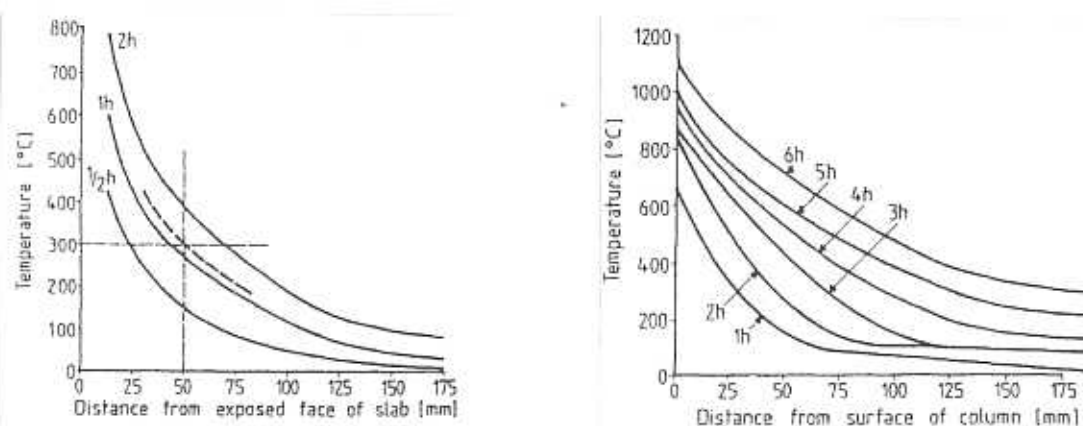


Fig. 6-41: Temperature distribution in (a) slabs and (b) columns ($l = 380\text{mm}$) exposed to a standard fire (Schneider, 1990)

Assessing the residual capacity of concrete structures exposed to fire is then a quite difficult task, because the traditional destructive or non-destructive testing techniques are generally not suitable for the inspection of such a highly heterogeneous layered-material. The possible approaches to this problem generally involve the inspection of the average response of the concrete cover, a point by point analysis of small samples taken at different depths or some special techniques aimed to interpret the overall response of the concrete member after fire (Table 6-1).

Table 6-1: Possible approaches to Non-Destructive assessment of fire-damaged concrete structures.

Average response of the concrete cover	Point by point response of small samples	Special interpretation techniques
Schmidt rebound hammer Windsor probe Capo test BRE internal fracture Ultrasonic Pulse Velocity	Small-scale mechanical tests Differential Thermal Analysis (DTA) Thermo-gravimetric Analysis (TGA) Dilatometry (TMA) Thermoluminescence Porosimetry Colorimetry Microcrack-density analysis Chemical analysis	UPV indirect method Impact echo Sonic tomography Modal Analysis of Surface Waves (MASW) Electric Resistivity

Several chemo-physical transformations take place in the concrete at increasing temperature (Fig. 6-42): the physically combined water is released above 100°C ; the silicate hydrates decompose above 300°C and the portlandite will be dehydrated above 500°C ; some aggregates begin to convert or to decompose at temperatures above 600°C (α - β - SiO_2 -conversion, decomposition of limestone).

The mechanical response of the material is weakened concurrently and the compressive strength is expected to be reduced slowly below 450-500°C and rapidly above 500°C. However, the decay can significantly depend on the mix design and on the heating and cooling conditions (Fig. 6-42b). As a consequence, no clear relationship can be found between the maximum temperature and the residual concrete strength. This latter parameter is then of greater interest in the assessment of the residual capacity of a structural member and it is generally the object of the Non-Destructive evaluations. On the other hand, the maximum temperature is of great interest in the assessment of the steel reinforcement residual capacity or to ascertain some critical chemo-physical transformations, such as the quartz transition.

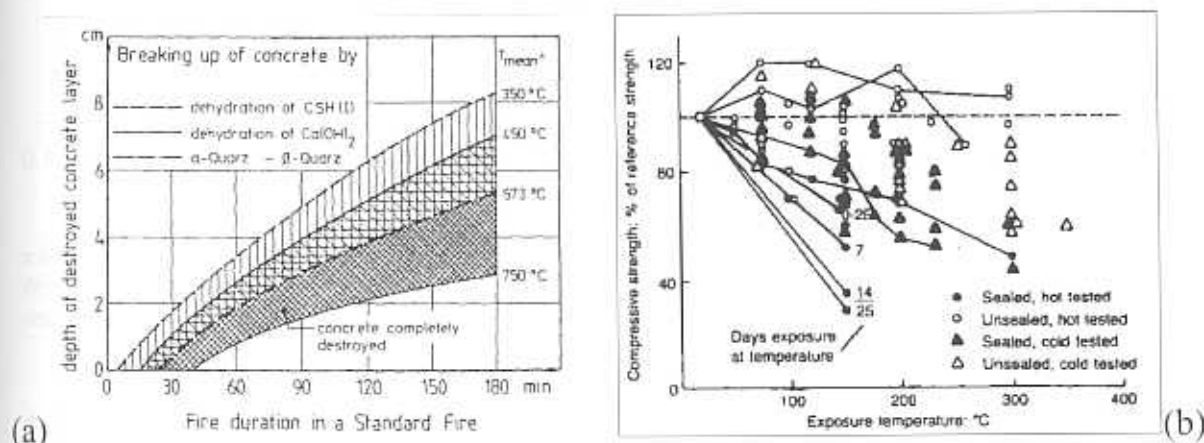


Fig. 6-42: Depth of transformation at increasing duration of the ISO fire (Schneider, 1990); and (b) compressive strength decay of concrete under different test conditions (Khoury, 1992)

Concerning the other mechanical parameters, a more marked decrease is usually observed for the Young's modulus, whereas the tensile strength shows the most temperature sensitive behavior (Fig. 6-43). Other physical properties are more or less affected by the high temperature exposure, such as density, porosity (total volume and average size of pores), concentration of microcracks, color, electric conductivity, etc.

This extensive series of transformations casts the base for the material assessment after a fire by means of NDT techniques. With regards to this point, the most common requirement of in-situ investigations is the estimation of concrete strength, through the assessment of one or more well defined concrete properties and the adoption of specific calibration curves. Due to the fact that not only the material strength is affected by the exposure to fire, different strength estimates have to be expected from a virgin and a damaged concrete exhibiting the same ND test response.

As an example, it is known that increased values of rebound hammer index are recorded in the case of a dry concrete. Then, the thermally induced strength decay is initially offset by the simultaneous water loss and only after a sizable damage the rebound index begins to diminish.

As a consequence, the usual calibration curves supplied by the ND instrument manufacturer should not be adopted for heated concrete evaluation. In the case a specific calibration curve for the thermally damaged concrete at issue is not available, sufficiently reliable conclusions can generally be attained by determining the variation of the tested parameter compared to unheated concrete. Weighing this variation against the corresponding strength decay allows to ascertain the sensitivity of a testing technique (Fig. 6-44). This kind of relationship is a valuable basis to form an opinion on the effectiveness of each NDT technique for the assessment of fire effects on concrete structures.

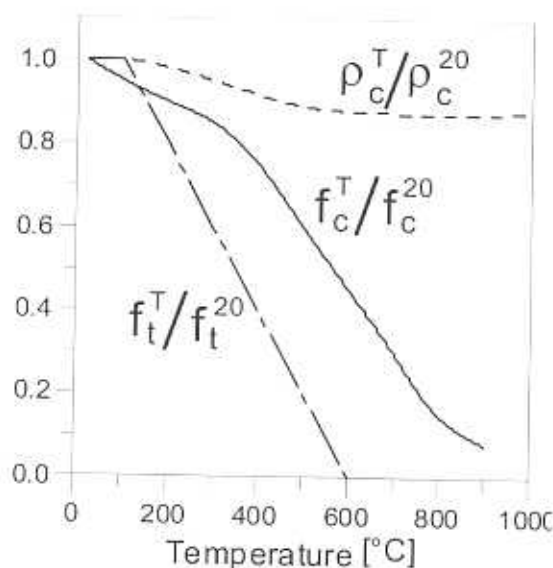


Fig. 6-43: Decay of concrete density and strength (compressive and tensile) according to EC2

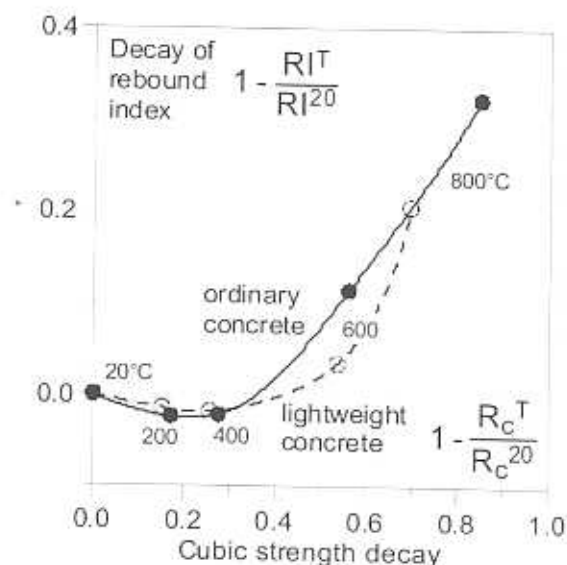


Fig. 6-44 Decay of the rebound hammer index versus temperature induced cubic strength decay (Felicetti, 2003, private communication)

6.2.3 Core test

The most direct method of estimating the strength of in situ concrete is by testing cores cut from the structure. However, little information is available on the behavior of core specimens having varying strength along the core (which are likely to result from fire-damaged concrete structure).

Nonetheless, extracting cores from a fire-damaged structure is usually the first step for a number of available test methods (colorimetry, porosimetry, chemical analysis, etc). It should also be noted that cutting a core itself provides useful information on the damage depth, provided that a continuous recording of both the coring depth and the energy consumption is carried out. This kind of inspection has been performed during the assessment of the concrete lining in the Mont Blanc Tunnel and is the object of an experimental program in progress at Milan University of Technology.

6.2.4 Schmidt hammer test

This test gives a measure of the surface hardness of the concrete, although there is no obvious relationship between this and the strength. In fact, the larger elastic and inelastic deformability, the more pronounced porosity and microcracking, and the free water loss play a not negligible role in determining the decay of the rebound index at increasing temperature. Moreover, it has to be considered that the instrument provides information about the average response of a 20-30 mm thick surface layer. Due to the necessity of a flat surface to make the test, and of a large number of tests for statistical reasons, this test is not generally suitable for heavily-damaged surfaces, as often occurs in fire-damaged structures, because of spalling.

The results available in the literature are very scattered, probably because of different concrete mixes (aggregate type and content) and initial moisture content (Fig. 6-45). However, the rebound hammer is a very popular tool and the test is very easy to be performed. Hence its application can be suggested for a fast detection of the areas where the concrete of the exposed surface has lost 30-50% of its original strength.

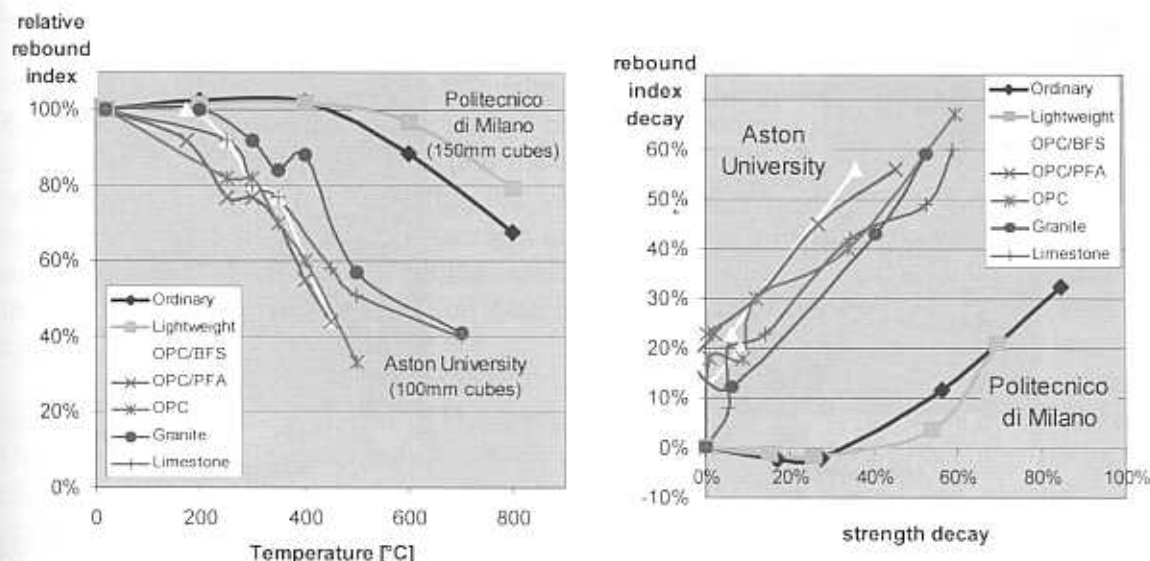


Fig. 6-45: Rebound-index decay in uniformly-heated concrete cubes and sensitivity of Schmidt's hammer test to a thermally-induced strength decay (Short et al, 2002; Felicetti, 2003, private communication)

6.2.5 Ultrasonic pulse velocity test

The use of the pulse velocity technique for the assessment of thermally damaged concrete is well established, given the fundamental relationship between pulse velocity and dynamic elastic modulus and the pronounced temperature sensitivity of this latter parameter. Moreover, the initial free water loss has the effect of emphasizing the velocity decay at relatively low temperatures (Fig. 6-46). Nonetheless the wide dispersion of experimental results precludes the outlining a general trend for the calibration curves. Again, this test requires a flat surface and is therefore appropriate only for the surfaces with no spalling. The method has been found to be particularly suitable for the use in waffle and trough floors, and for assessing the damage extent due to a localized fire.

Contrary to normal practice, the transmitting and receiving heads can be profitably placed on the same side of a structural member (indirect method). In this method, several measurements of the pulse arrival time are performed at increasing distance between the probes. The outcome is a plot on the X-T axes (probe distance - arrival time - Fig. 6-47). Assuming that the sound velocity increases at increasing depths, it is then possible to investigate deeper and deeper material layers.

Several numerical methods have been proposed for the interpretation of this type of graphs (Benedetti, 1998). In a recent study performed at Milan University of Technology a general relationship between some geometric parameters of the X-T curves and the depth and maximum severity of the pulse velocity profile has been worked out. However, the effect of possible shrinkage and delamination cracks should be taken into account in the interpretation of the results.

Other interesting methods have been developed based on the sound velocity, amplitude attenuation and vibration frequency and modes. Among them the Impact Echo can be profitably used for detecting possible delamination cracks or for determining the thickness of a tunnel lining. The Modal Analysis of Surface Waves has been profitably used in the assessment of the sound velocity maps on the Mont Blanc Tunnel lining (CETU and University of Sherbrooke).

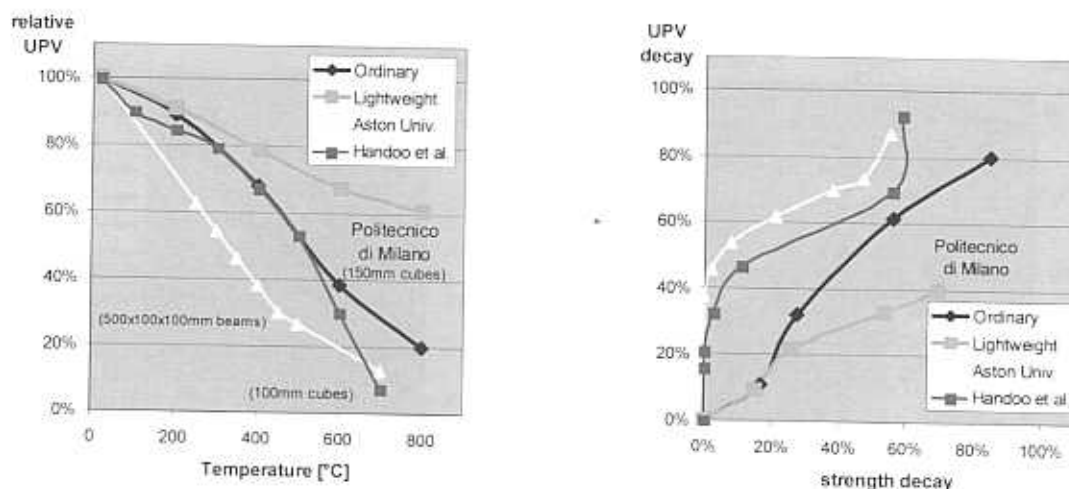


Fig. 6-46: Ultrasonic Pulse-Velocity decay in uniformly-heated concrete cubes and prisms, and sensitivity to a thermally-induced strength decay (Short et al, 2002; Felicetti, 2003, private communication)

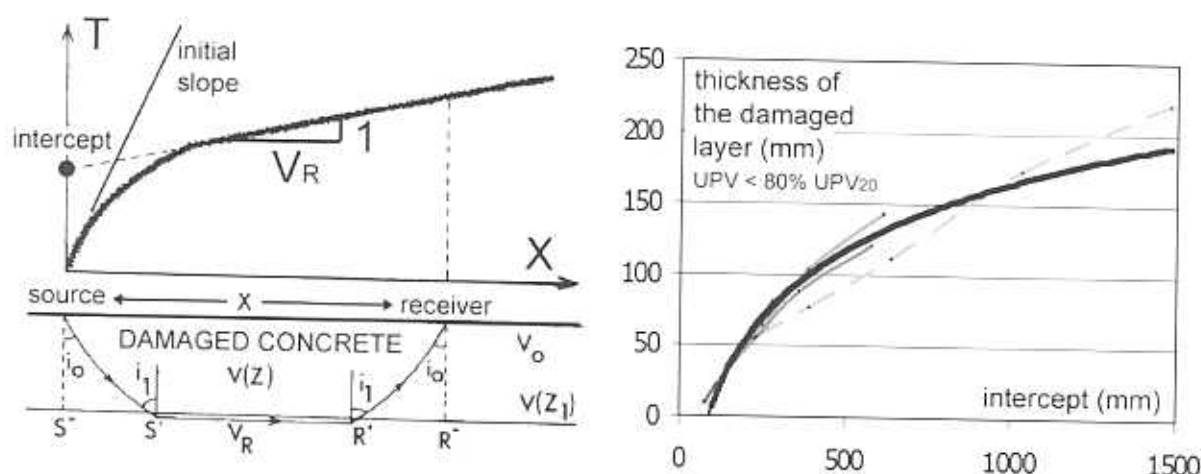


Fig. 6-47: Relevant geometric parameters of the indirect UPV method X-T curves and correlation between the intercept and the thickness of the damaged concrete layer

6.2.6 Windsor probe

This test was developed in the USA about 40 years ago but has not received much attention in most countries. The test involves shooting a steel probe into the surface of the concrete. The length of probe left exposed is measured and can be correlated with compressive strength.

This test has been found to be very quick and simple and gives a low within-test variation and can be used on surfaces subjected to spalling, provided that these surfaces are reasonably flat. Slight preparation is necessary on very rough surfaces. This test may, therefore, be used on a flat and indented surface, and hence is ideally suitable for determining the strength profile used on surfaces cut to different depths. Direct correlation with strength is slightly better with this test than the other test methods but more reliable results will be obtained by comparison with unaffected concrete. However, no specific calibration curves are available in the literature concerning the residual strength of fire-damaged concrete.

6.2.7 BRE internal fracture test and CAPO test

The BRE (Building Research Establishment) internal fracture test involves drilling a hole (6mm diameter) in which a wedge anchor bolt with expanding sleeve is placed 20mm below the surface. The torque required to pull the anchor out gives an indication of strength from a calibration chart. However, the within-test variation has been found to be fairly wide, with the result that direct determination of strength is less reliable. A comparison with sound concrete usually improves the reliability but the results are perhaps not as good as those obtained from the Windsor probe unless calibration data is available for the same type of concrete being tested.

The "cut and pull-out" test (CAPO test) is a similar technique that is based on an undercut anchor and has been developed in Denmark about 25 years ago (Fig. 6-48a,b). In this case a 45mm deep, 18mm diameter hole is drilled, after which a 25mm groove is cut at 25mm depth using a portable milling machine. An expanding ring is then placed and expanded in the groove and a pulled with a special equipment fit with a 55mm diameter restraint ring. Compared to BRE internal fracture test, this latter techniques has the advantage of more controlled fracture cone boundary conditions, which should allow for a better repeatability of the results. Some calibration tests have been performed at Milan University of Technology, showing a good sensitivity of the method (Fig. 6-48c,d), which seems to assess the average residual strength of a 10-15mm thick surface layer.

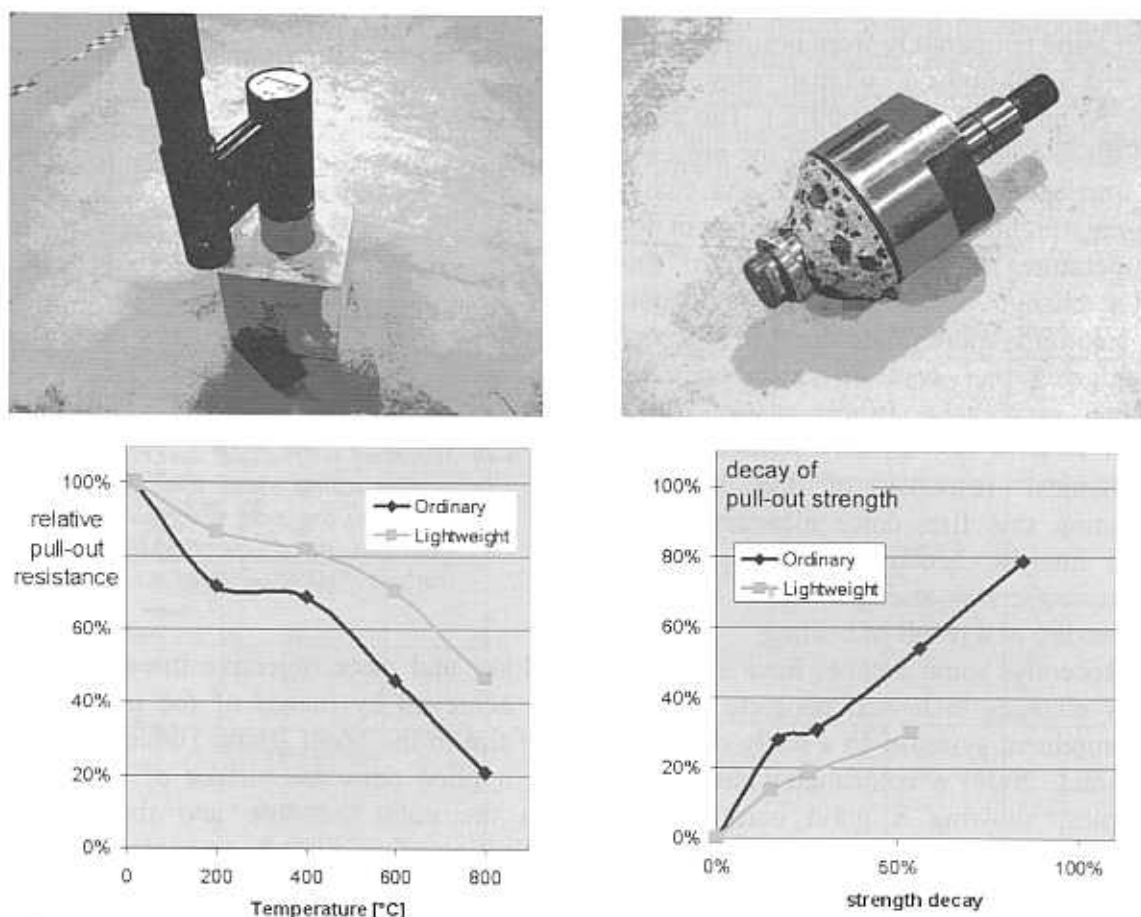


Fig. 6-48: (a) Capo-test equipment for undercut-anchor extraction and (b) fractured-concrete cone; (c) decay of the pull-out resistance on uniformly-heated concrete cubes; and (d) sensitivity to a thermally-induced strength decay (Felicetti, 2003, private communication)

A reasonably flat surface approximately 100 mm in diameter is required to support the ring of the test apparatus. A chiselled surface would be sufficient, though its greater size requires more preparatory work than in the case of the Windsor probe. This test may be carried out on vertical and horizontal surfaces, but the utmost care should be taken particularly on overhead work, since the apparatus is made of steel and, on occasions, the anchor may break before the failure of the concrete core, with a sudden rather than a gradual failure.

6.2.8 Concrete colorimetry

A traditional method for the assessment of concrete damage after a fire is based on the visual inspection of the material color. The color of concrete generally changes at increasing temperature from normal to pink or red (300-600°C), whitish gray (600-900°C) and buff (900-1000°C). The pink-red discoloration ensues from the presence of iron compounds in the fine and coarse aggregates, that dehydrate or oxidize in this temperature range. The strength of this color change depends on the aggregate type and it is more pronounced in siliceous aggregates and less in calcareous and igneous aggregates (Short et al., 2001). Even if it is not directly related to the mechanical response of the material, detecting this first color alteration is of great interest, because it usually occurs when concrete strength starts sharply decreasing as a result of heating.

Recently, some authors have shown that a closer and more objective inspection of the color changes in heated concrete can be viably achieved by means of the modern color measurement systems. In a study on the effects of fire in the Mont Blanc Tunnel (Faure and Hemond, 2004) a colorimeter has been directly applied onto the surface of the concrete samples, showing a good correlation between the color measure and the maximum experienced temperature. Another interesting application of colorimetry to heated concrete is based on a optical microscope combined with a digital image analysis workstation (Short et al., 2001). Optical magnification and image resolution make it possible to carry out a point-by-point examination of concrete constituents and to obtain of the color profiles.

Following the above encouraging results, a new technique has been recently proposed at Milan University of Technology, based on a proper processing of the digital images taken via a common low-cost digital camera. Besides the advantage of focusing the measurement on

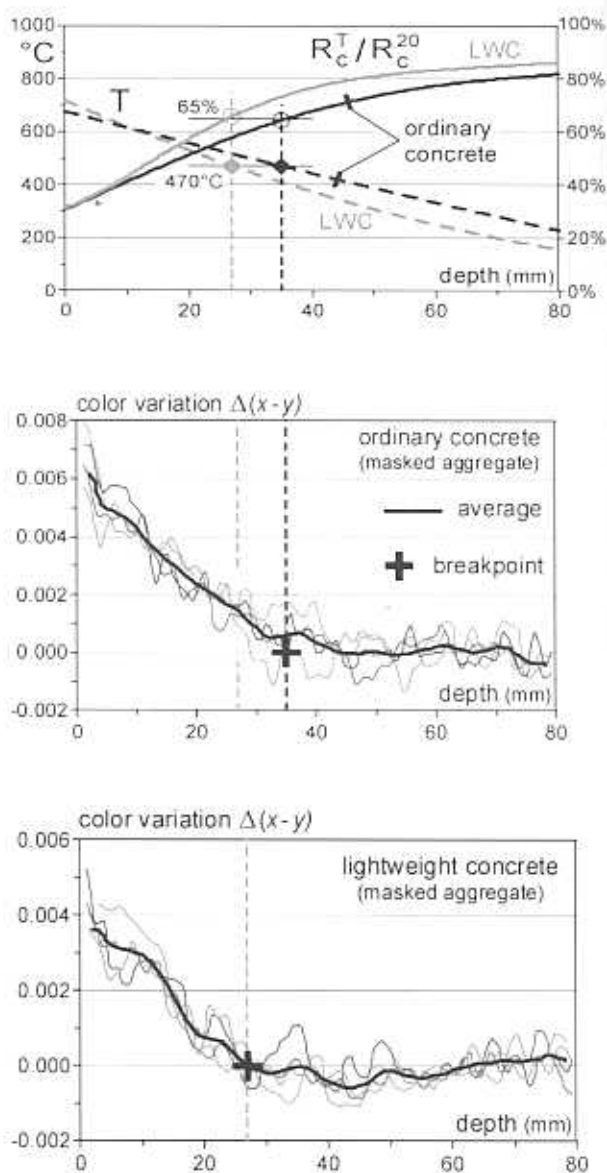


Fig. 6-49: Maximum temperature and residual strength profiles within 80mm thick concrete panels heated on one side; color-variation profiles measured on 4 cores via digital image analysis (Felicetti, 2005a)

this commonly available device, a single side picture of a concrete core is sufficient to assess the material color profile at increasing depths and a separate analysis of the cement mortar and the aggregate can be easily performed (Fig. 6-49, Felicetti, 2005a).

6.2.9 Thermoluminescence tests

Thermoluminescence is the visible light emission, which occurs while heating certain minerals, including quartz and feldspar; the curve of the light output as a function of the temperature for a given sample depends on its thermal and radiation history (Bungey, 1982).

In the case of fire-damaged concrete, this technique relies on the measurement of the residual thermoluminescence in small amounts of sand drilled from the concrete. A major loss of thermoluminescence occurs in the range 300-500°C, i.e. when concrete strength begins to be markedly affected by the temperature. An advantage of this technique is that only small holes are to be drilled for sampling; then, by quantifying the thermoluminescence the temperature profiles can be determined, from the heated surface. However, performing this kind of test requires a special equipment and needs a specific experience.

6.2.10 Carbonation test

Carbonation depth may be determined by spraying the concrete with a phenolphthalein solution and measuring the depth of the discolored zone. Useful information on the residual durability of a fire-damaged structure can be obtained by comparing its carbonation depth (a few years after the fire) and the carbonation depth of an undamaged building having the same age (Fig. 50). Of course, the carbonation depth depends on fire severity.

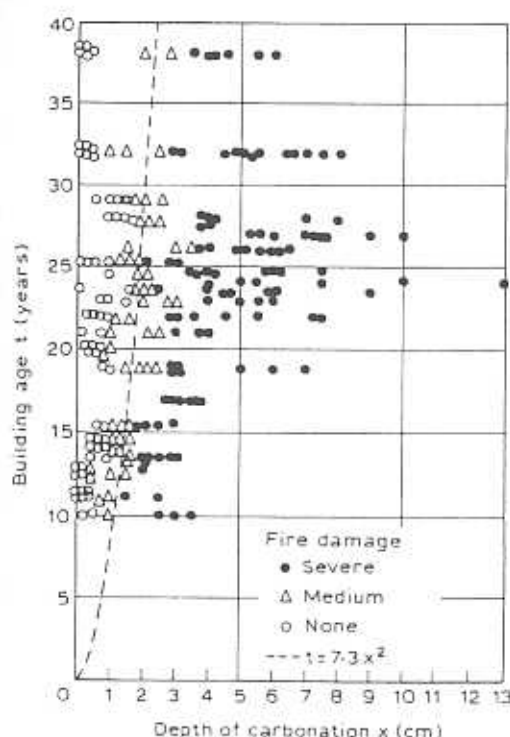


Fig. 6-50: Depth of carbonation 3-4 years after a fire, as a function of the building age (Schneider, 1990)

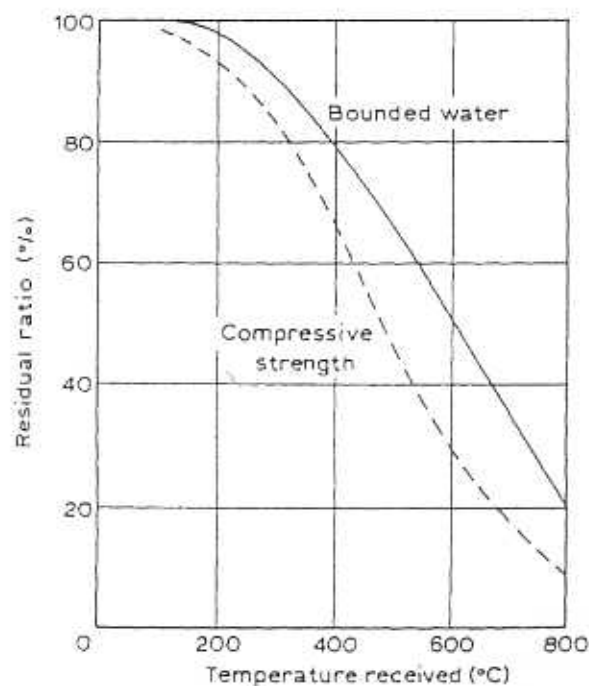


Fig. 6-51: Concrete residual properties: bounded-water content and compressive strength as a function of the temperature (Schneider, 1990)

6.2.11 Chemical analysis

Chemical analysis can be performed to find the residual combined water in hardened cement or the residual chloride in the concrete. The analysis to determine the combined water was developed in Japan about 50 years ago and is still used today. This method requires the concrete to be carefully chiselled along the thermally-damaged surface, in order to collect some concrete powder belonging to each layer. After the elimination of the sand, the sample of cement powder is heated in an electric furnace to measure the residual combined water content as shown in Fig. 51. From the relation between the residual combined water content and the maximum measured – or expected – temperature, the temperature profiles can be drawn and the strength reduction can be evaluated.

Chloride ions may attack the concrete during and after the fire due to the decomposition of plastics containing polychlorides, e.g. PVC. Originally the chloride exists in the surface layers at a depth of 5-10 mm. Due to diffusion, chloride ions may later move into deeper concrete layers and generate localized corrosion in the reinforcement. Therefore the determination of chloride content is one of the main objectives after a fire involving – for instance – the plastic ducts of a tendon. The commercially-available test methods are potentiometric titration, x-ray fluorescence and test stripes for chloride analyses.

6.2.12 X-Ray diffraction analysis (XRD)

Phase analysis of hardened cement mortar by XRD shows the presence of usual hydrated phases such as Portlandite $\text{Ca}(\text{OH})_2$, calcium silicate hydrate (C-S-H), ettringite, along with quartz due to siliceous aggregate used in concrete. Multiple XRD patterns of mortar samples exposed to increasing temperature in the range 100-1000°C show the gradual reduction in $\text{Ca}(\text{OH})_2$ content, indicating gradual deterioration in concrete quality (Fig.52a, Handoo et al., 2002).

6.2.13 Chemo-physical and mechanical tests

A number of testing techniques is based on the repeated testing of small samples of concrete taken at different depths, inside the fire-damaged members. The following techniques – to be used in specialized laboratories – are based on the chemo-physical transformations in the material:

- Thermo-dilatoMetric Analysis (TMA)
- ThermoGravimetric Analysis (TGA)
- Differential Thermal Analysis (DTA)

Most of the thermally induced transformations of concrete are not reversible. Hence, during the second heating of the damaged concrete, no significant transformations occur until the maximum temperature experienced during the previous fire is exceeded. The evidence of an ongoing transformation during the second material heating in a laboratory furnace can be provided by length (TMA), weight (TGA) and temperature (DTA) measurements.

The use of Differential Thermogravimetric Analysis after a fire, to evaluate the temperature attained by a structural member during a fire was introduced by Harmathy in 1968.

The Differential Thermal Analysis involves heating a small sample of powdered concrete in a furnace, together with a similar sample of inert material. Both samples are monitored to

provide a trace of their temperature difference, which ensues from the loss of water of crystallization in the various components and makes it possible to identify the presence of a number of minerals (Fig.52b, Handoo et al., 2002).

Further approaches based on a point-by-point study of small samples are aimed to assess the amount of defects, which is directly related to material decay (Fig. 53, Short et al., 2000). Mercury-intrusion porosimetry is commonly used to evaluate the cumulative volume of pores within a wide range of sizes. Both the quantity and the average size of pores increase owing to high-temperature exposure. Modern digital-image analysis allows also to measure micro-crack density (namely the total length of cracks per unit surface). This latter parameter seems to be markedly affected by the temperature.

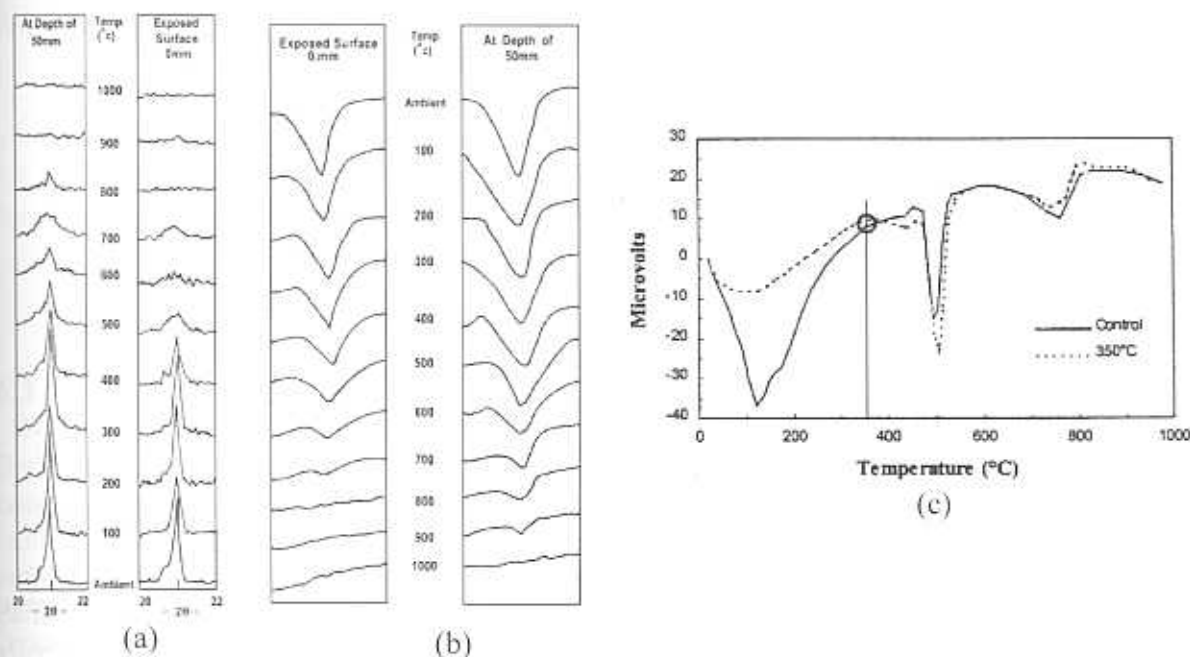


Fig. 6-52: (a) X-ray diffractograms of mortar samples to monitor Ca(OH)_2 (Handoo et al., 2002); (b) DTA patterns of mortar samples heated at different temperatures (Handoo et al., 2002); and (c) typical DTA traces and related hydrates (Short et al., 2000)

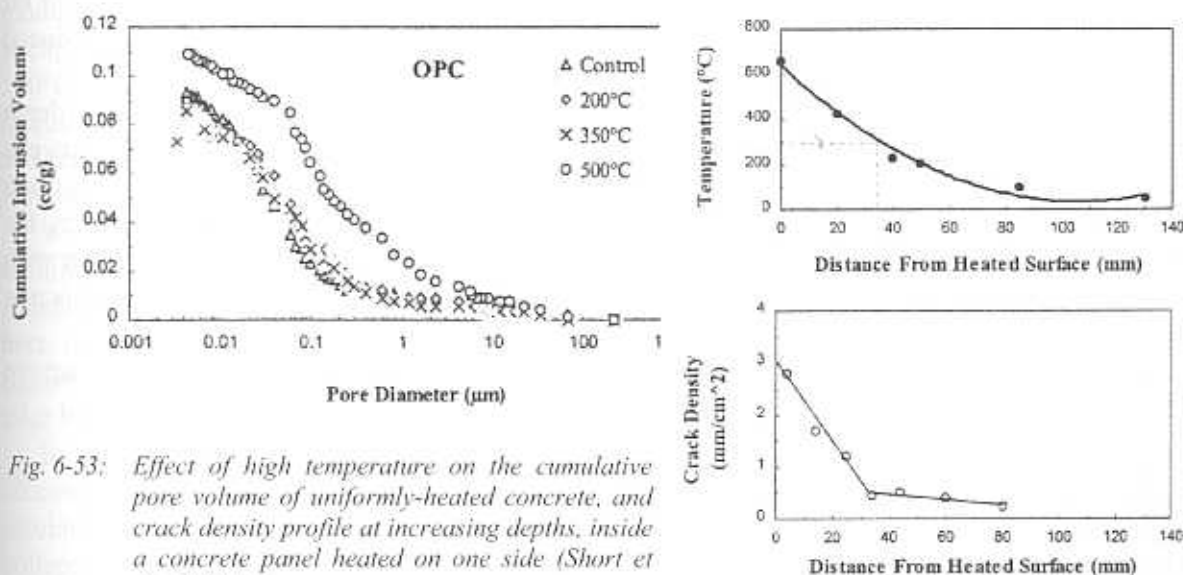


Fig. 6-53: Effect of high temperature on the cumulative pore volume of uniformly-heated concrete, and crack density profile at increasing depths, inside a concrete panel heated on one side (Short et al., 2000)

Other techniques have been proposed, such as the ultrasonic longitudinal scanning of concrete cores, with the emitter and the receiver applied in diametrically-opposed locations.

A recently-proposed technique suitable for evaluating concrete compressive strength is the so-called "Disk Punching-Test". A thermally-damaged concrete core is cut into thin disks, at different depths from the heated extremity, and then each disk is tested in punching (Fig. 54, Benedetti and Mangoni, 2005).

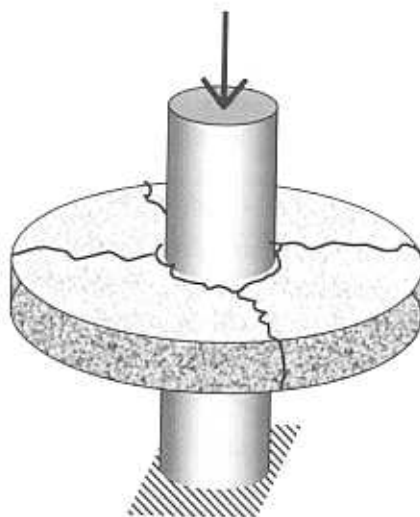


Fig. 6-54: Disk Punching-Test for the evaluation of the compressive strength (Benedetti and Mangoni, 2005)

6.2.14 Drilling resistance

As previously indicated, a common approach to the assessment of concrete damage profiles is to repeatedly analyze a series of small samples taken at different depths. However, this methodology seems to be too demanding for the assessment of R/C structures damaged by a severe fire, since a lot of testing points have to be investigated.

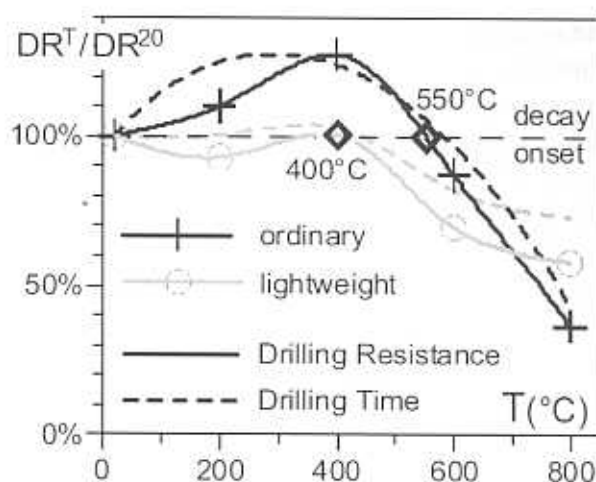
A promising and much faster technique is based on the measurement of the drilling resistance, which allows to continuously "scan" the material response in a single operation (Felicetti, 2005b). A hammer drill is usually recommended (Fig.55a) in order to prevent bit wearing and overheating. In this case, the sensitivity to the exerted thrust is markedly reduced and no special control of either the drilling force or the penetration rate is needed.

For the application to damaged concrete, the work dissipated per unit drilling depth (J/mm) appears to be the most sensitive indicator of material integrity. A correlation between this parameter and the compressive strength cannot be easily worked out, owing to the strong influence of other properties like fracture energy and aggregate hardness. However, the drilling resistance keeps its significance in relative terms (Fig.55b) and the comparison with the inner virgin material provides meaningful information on the thickness of the concrete layer damaged by the fire.

This method usually provides reliable information, especially in the case of a severe thermal damage ($R_c^T < 0.7 R_c^{20^\circ\text{C}}$).



(a)



(b)

Fig. 6-55: (a) Modified hammer drill for the measurement of the drilling resistance of thermally-damaged concrete; and (b) effect of the maximum temperature on the material drilling resistance (Felicetti, 2005b).

6.3 Concluding remarks

Both the concrete and the reinforcement suffer from the exposure to high temperature, that causes a loss in terms of strength and stiffness, starting from 400-450°C. However, the steel reinforcement tends to recover most – and in some cases the totality – of its initial mechanical properties, after cooling to room temperature, while the concrete undergoes a further loss during – and immediately after – the cooling process. (This loss is recovered in 6-18 months after cooling).

As discussed in the first part of this Chapter, the mechanical loss depends on the type of the material. High-performance silica-fume concretes tends to be more heat-sensitive, but the loss during – and immediately after – the cooling is more limited than in ordinary concretes (however, the same applies to the long-term recovery after cooling). The residual elastic modulus, whatever may be measured, is more heat-sensitive than the compressive and tensile strengths. The loss in terms of tensile strength and elastic modulus tends to build up above 100°C, while the compressive strength is rather constant up to 300-400°C. The residual fracture energy increases with the temperature (up to 200-400°C), but then starts decreasing and tends to be back to the value of the virgin material at 600°C, which means that up to this temperature the concrete is tougher than in the virgin conditions.

With reference to the residual properties of the reinforcement, hot-rolled bars are definitely less heat-sensitive than cold-worked bars (be they ordinary bars – even made of stainless steel – or high-strength bars for P/C structures). Among hot-rolled bars, quenched bars (extensively used nowadays) are slightly more heat-sensitive than carbon-steel bars (above 550°C), but both are definitely more sensitive than stainless-steel bars, that – even after being exposed to 850-900°C – entirely recover their initial strength after cooling.

In spite of materials heat-sensitivity, coupling concrete and steel is generally highly successful in fire conditions, because of concrete low diffusivity, that guarantees the thermal insulation of the reinforcement. This is the reason why seldom a R/C or P/C structure collapses during or after a fire. As a consequence, different non-destructive techniques have been devised to assess the residual safety level of fire-damaged structures, as shown in the second part of this Chapter. The first step is to evaluate the maximum temperature reached by

the outermost layers of the concrete. It can be done in different ways. The methods based on concrete colorimetry and drilling resistance are quite promising, since they allow to examine the concrete layer-by-layer, starting from the heated surface. In the former case, from the color changes of the lateral surface of a core or of a small hole drilled inside the concrete, the depth of the thermal damage can be recognized, while in the latter case the work per unit depth required by drilling a hole with an ordinary hammer drill is a reliable indication of the local residual strength of the concrete (from hence, the temperature reached during the fire can be inferred). Once the thermal field is known, the maximum temperature reached by the reinforcement can be determined as well, and so the residual capacity of the structural member in question.

Summing up, since the temperature gradients during a fire are quite high, the thermal damage rapidly decreases starting from the heated surface, making it difficult to have point-by-point information on the actual thermal field. For this reason, only by using different, more or less sophisticated techniques it is possible to have a reliable picture of the severity of the fire, that is the starting point of any structural analysis aimed to identify the best strategy to be adopted in dealing with a fire-damaged structure (demolition, rehabilitation or rehabilitation and strengthening).

References

- Balendran R.V., Nadeem A., Maqsood T. and Leung H.Y. (2003): "Flexural and Split Cylinder Strengths of HSC at Elevated Temperatures", *Fire Technology*, No. 39, pp. 47-61.
- Bamonte P., Cangiano S. and Gambarova P.G. (2006): "Thermal and Mechanical Characterization of a High-Performance Micro-Concrete", *Proc. 2nd fib Conference*, V.2, Naples (Italy), June 2006, 238-249.
- Bamonte P., Felicetti R. and Gambarova P.G. (2008): "Mechanical Properties of Self-Compacting Concrete at High Temperature and after Cooling", *Proc. of SCC 2008 - 3rd North American Conf. on the Design and Use of SCC : Challenges and Barriers to Application*, Chicago (USA), November 10-12 2008, 6 pp. (in press).
- Barragán B.E., Giaccio G.M. and Zerbino R.L. (2001): "Fracture and Failure of Thermally-Damaged Concrete under Tensile Loading", *Materials and Structures*, V. 34, June 2001, pp. 312-319.
- Bazant Z.P. and Kaplan M.F. (1996): "Concrete at High Temperatures – Materials Properties and Mathematical Models", *Concrete Design and Construction Series*, ed. by F.K. Kong and R.H. Evans, Longman Group Limited, 412 pp.
- Benedetti A. (1998): "On the Ultrasonic Pulse Propagation into Fire-Damaged Concrete", *ACI-Structural Journal*, May-June, V.95, No.3, pp. 257-271.
- Benedetti A. and Mangoni E. (2005): "Damage Assessment in Actual Fire Situations by means of Non-Destructive Techniques and Concrete Tests", *Proc. Int. Workshop "Fire Design of Concrete Structures: What now? What next?" - fib Task Group 4.3*, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp. 231-239.
- Bo Wu, Xiao-Ping Su, Hui Li and Jie Yuan (2002): "Effect of High Temperature on Residual Mechanical Properties of Confined and Unconfined High-Strength Concrete", *ACI-Materials Journal*, V.99, No.4, pp.399-407.
- Buchanan A.H. (2001): "Structural Design for Fire Safety", John Wiley & Sons Ltd, Chichester (West Sussex, England, U.K.), 421 pp.

- Bungey J.H (1982): "The Testing of Concrete in Structures", Surrey University Press, New York, 207 pp.
- Castillo C. and Durrani A.J. (1990): "Effect of Transient High Temperature on High-Strength Concrete", *ACI-Materials Journal*, V.87, No.1, pp. 47-53.
- Chan S. Y. N., Peng G. and Chan J. K. W. (1996): "Comparison between High-Strength Concrete and Normal-Strength Concrete Subjected to High Temperature", *Materials and Structures*, V.29, No.12, pp. 616-619.
- Chan Y.N., Luo X. and Sun W. (2000): "Compressive Strength and Pore Structure of High-Performance Concrete after Exposure to High Temperature up to 800°C", *Cement and Concrete Research*, V.30, pp. 247-251.
- Cheyrezy M. (2001): "Compressive Strength of HPCs at High Temperature", *Proc. Int. Workshop on Fire Design of Concrete Structures – fib Task Group 4.3 "Fire Design of Concrete Structures"*, ed. by N.P. Høj, Malta, March 2001, pp. 2-7.
- Demorieux J.M. and Levy M. (1998): "On the Fire in the Tunnel under the British Channel: General Presentation" (in French), *Annales de l'I.T.B.T.P.*, September 1998.
- Diederichs U., Jumppanen U.M. and Penttala V. (1989): "Behaviour of High-Strength Concrete at High Temperatures", Report 92, Dept. of Structural Engineering, Helsinki University of Technology, Helsinki (Finland), 76 pp.
- Faure R.M. and Hemond G. (2004): "Application of the Methods for the Analysis of a Fire-Damaged Concrete" (in French).
- Felicetti R. and Gambarova P.G. (1998): "Effects of High Temperature on the Residual Compressive Strength of High-Strength Siliceous Concretes", *ACI-Materials Journal*, V.95, No.4, pp.395-406.
- Felicetti R. and Gambarova P.G. (1999): "On the Residual Tensile Properties of High-Performance Siliceous Concrete Exposed to High Temperature", *Proc. Int. Workshop on "Mechanics of Quasi-Brittle Materials and Structures"*, ed. by G. Pijaudier-Cabot, Z. Bittnar and B. Gérard, Prague (Czech Rep.), March 1998, pp. 167-186.
- Felicetti R., Gambarova P.G., Khoury G.A. and Natali-Sora M.P. (2000): "Mechanical Behaviour of HPC and UHPC in Direct Tension at High Temperature and after Cooling", 5th RILEM Symp. BEFIB'2000, ed. by P. Rossi and G. Chanvillard, RILEM Publ. S.A.R.L., Lyon (France), September 2000, pp. 749-758.
- Felicetti R., Gambarova P.G., Silva M. and Vimercati M. (2002): "Thermal Diffusivity and Residual Strength of High-Performance Light-Weight Concrete Exposed to High Temperature", *Proc. 6th Int. Symposium on the Utilization of High-Strength/High-Performance Concrete*, V.2, ed. by G. König, F. Dehn and T. Faust, Leipzig (Germany), June 2002, pp. 935-948.
- Felicetti R. (2005a): "Digital-Camera Colorimetry for the Assessment of Fire-Damaged Concrete", *Proc. Int. Workshop "Fire Design of Concrete Structures: What now? What next?" - fib Task Group 4.3*, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp. 211-220.
- Felicetti R. (2005b): "The Drilling-Resistance Test for the Assessment of the Thermal Damage in Concrete", *Proc. Int. Workshop "Fire Design of Concrete Structures: What now? What next?" - fib Task Group 4.3*, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp. 241-248.

- Felicetti R. and Meda A. (2005): "Residual Behaviour of Reinforcing Steel Bars after a Fire", Proc. *fib* Symposium "Keep Concrete Attractive", ed. by G.L. Balázs and A. Borosnyói, publ. by Budapest University of Technology and Economics, Budapest (Hungary), May 2005, pp. 1148-1155.
- FEMA – Federal Emergency Management Agency (2002): "World Trade Center Building Performance Study: Data Collection, Preliminary Observations and Recommendations", 400 pp.
- Franssen J.M. (2005): "Plastic Analysis of Concrete Structures Subjected to Fire", Proc. Int. Workshop on "Fire Design of Concrete Structures: What now? What next?" - *fib* Task Group 4.3, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp.133-145.
- Gawęska-Hager I. (2004): "Behavior at High Temperature of High-Performance Concretes : Evolution of the Main Mechanical Properties" (in French), Dissertation, Ecole Nationale des Ponts et Chaussées and Krakow University of Technology, November 2004, Paris (France), 183 pp.
- Guerrini G.L. and Rosati G.P. (2003): "Residual Strength of White-Cement Concrete Exposed to High Temperatures", 6th CANMET/ACI Int. Conf. on Durability of Concrete, ed. by V.M. Malhotra, Thessaloniki (Greece), June 2003, pp. 587-599.
- Hager I. and Pimienta P. (2005): "Mechanical Properties of HPC at High Temperature", Proc. Int. Workshop on "Fire Design of Concrete Structures: What now? What next?" - *fib* Task Group 4.3, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp. 95-100.
- Hamoush S.A., Abdel-Fattah H. and McGinley M.W. (1998): "Residual Fracture Toughness of Concrete Exposed to Elevated Temperature", ACI-Structural Journal, V.95, No.6, pp. 689-694.
- Handoo S.K., Agarwal S. and Agarwal S.k. (2002): "Physicochemical, Mineralogical and Morphological Characteristics of Concrete Exposed to Elevated Temperatures", Cement and Concrete Research, No.32, pp. 1009-1018.
- Harada T. et al. (1972): "Strength, Elasticity and Thermal Properties of Concrete Subjected to Elevated Temperatures", ACI SP 34, Detroit (USA).
- Harmathy T.Z. (1968): "Determining the Temperature History of Concrete Constructions Following Fire Exposure", ACI-Journal, V.65, pp. 959-964.
- Harmathy T.Z. (1993): "Fire Safety Design and Concrete", Concrete Design and Construction Series, Longman Scientific and Technical, U.K.
- Hironaka M.C. and Malvar L.J. (1998): "Jet Exhaust-Damaged Concrete", Concrete International, No.10, pp. 32-35.
- Hoff G.C., Bilodeau A. and Malhotra V.M. (2000): "Elevated Temperature Effects on HSC Residual Strength", ACI-Concrete Int., No.4, pp. 41-47.
- Hsu L.S. and Hsu C.T.T. (1994): "Complete Stress-Strain Behavior of High-Strength Concrete under Compression", Magazine of Concrete Research, V.46, No.169, pp.301-312.
- Jahren P. A. (1989): "Fire Resistance of High Strength/Dense Concrete with Particular Reference to the Use of Condensed Silica Fume – A Review", 3rd Int. Conf. on HSC, V.2, Trondheim (Norway), pp. 1014-1049.
- Jansen D.C., Shah S.P. and Rossow E.C. (1995): "Stress-Strain Results of Concrete from Circumferential Strain Deedback Control Testing", ACI-Materials Journal, V.92, No.4, pp. 419-428.

- Jansson R. and Boström L. (2005): "Experimental Investigation on Concrete Spalling in Fire", Proc. Int. Workshop "Fire Design of Concrete Structures: What now? What next?" - *fib* Task Group 4.3, ed. by P.G. Gambarova, R. Felicetti, A. Meda and P. Riva, publ. by Starrylink (Brescia, Italy), Milan (Italy), December 2004, pp. 109-113.
- Khoury G.A. (1992): "Compressive Strength of Concrete at High Temperatures: a Reassessment", Magazine of Concrete Research, V. 44, No. 161, pp. 291-309.
- Khoury G. A. (2000): "Effect of Fire on Concrete and Concrete Structures", Progress in Structural Engineering Materials, V.2, pp. 429-447.
- Khoury G.A., Grainger B.N. and Sullivan P.J. (1985): "Transient Thermal Strain of Concrete: Literature Review, Condition within Specimen and Behaviour of Individual Constituents", Magazine of Concrete Research, V.37, No.132, pp. 131-144.
- Lennon T. and Clayton N. (1999): "Fire Tests on High-Grade Concrete with Polypropylene Fibres", Proc. 5th Int. Symposium on the Utilization of High-Strength/High-Performance Concrete, V.2, Sandefjord (Norway), June 1999, pp. 1200-1208.
- Malhotra H.L. (1982): "Design of Fire-Resisting Structures", Survey University Press.
- NIST Special Publication 919 (1997): Proc. Int. Workshop on Fire Performance of High-Strength Concrete, Gaithersburg (MD, USA), Editors: L.T. Phan, J. Carino, D. Duthinh and E. Garboczi, March 1997, 164 pp.
- Noumowe N.A. and Debicki G. (2002): "Effect of Elevated Temperature from 200 to 600°C on the Permeability of High-Performance Concrete", Proc. 6th Int. Symposium on "Utilization of High-Strength/High-Performance Concrete, V.1, ed. by G. König, F. Dehn and T. Faust, Leipzig (Germany), June 2002, pp.431-444.
- Noumowé A., Carré H., Daoud A. and Toutanji H. (2006): "High-Strength Self-Compacting Concrete Exposed to Fire Test", ASCE Journal of Materials in Civil Engineering, Vol. 18, No. 6, pp. 754-758.
- Papayanni J. and Valiasis T. (1991): "Residual Mechanical Properties of Heated Concrete Incorporating Different Pozzolanic Materials", Materials and Structures, V.24, pp.115-121.
- Persson N. (2004): "Fire Resistance of Self-Compacting Concrete, SCC", Materials and Structures, V.37, November 2004, pp.575-584.
- Phan L.T. and Carino N.J. (1998): "Review of Mechanical Properties of HSC at Elevated Temperature", ASCE-Journal of Materials in Civil Engrg., V. 10, No.1, pp.58-64.
- Phan L.T. and Carino N.J. (2002): "Effects of Test Conditions and Mixture Proportions on Behavior of High-Strength Concrete Exposed to High Temperatures", ACI-Materials Journal, V. 99, No.1, pp. 54-66.
- Pimienta P. and Hager I. (2002): "Mechanical Behaviour of HPC at High Temperature", Proc. 6th Int. Symposium on the Utilization of High-Strength/High-Performance Concrete, V.2, ed. by G. König, F. Dehn and T. Faust, Leipzig (Germany), June 2002, pp. 1291-1298.
- Purkiss J.A. (1996): "Fire Safety Engineering Design of Structures", Publ. by Butterworth-Heinemann, Oxford (UK), 342 pp.
- Reinhardt H.W. and Stegmaier M. (2006): "Self-Consolidating Concrete in Fire", ACI Materials Journal, V. 103, No. 2, March-April 2006, pp. 130-135.
- RILEM (1985): "Properties of Materials at High Temperatures : Concrete", Ed. by U. Schneider, Publ. by the Dept. of Civil Engrg. of the University of Kassel (Kassel, Germany), 131 pp.

- RILEM TC 200-HTC (2004): "Mechanical Concrete Properties at High Temperature - Modelling and Applications - Modulus of Elasticity for Service and Accident Conditions", *Materials and Structures*, V.37 (266), pp 139-144.
- Rosati G.P. and Sora M.P. (2001): "Direct Tensile Tests in Concrete-like Materials: Structural and Constitutive Behaviors", *ASCE-Journal of Engrg. Mechanics*, V. 127, No.4, pp. 364-371.
- Schneider U. (1990, Guest Editor): "Repairability of Fire-Damaged Structures", *Fire Safety J.*, V.16, pp. 251-336.
- Short N.R., Purkiss J.A. and Guise S.E. (2000): "Assessment of Fire-Damaged Concrete", *Proc. of the Concrete Communication Conference 2000*, BCA, Crowthorn (UK), pp. 245-254.
- Short N.R., Purkiss J.A. and Guise S.E. (2001): "Assessment of Fire-Damaged Concrete Using Colour Image Analysis", *Construction and Building Materials*, No. 15, pp. 9-15.
- Short N.R., Purkiss J.A. and Guise S.E. (2002): "Assessment of Fire-Damaged Concrete Using Crack Density Measurements", *Structural Concrete*, V. 3, pp. 137-143.
- Sideris K.K (2007): "Mechanical Characteristics of Self-Consolidating Concretes Exposed to Elevated Temperatures", *ASCE Journal of Materials in Civil Engineering*, Vol. 19, No. 8, pp. 648-654.
- Taerwe L.R. (1992): "Influence of Steel Fibers on Strain-Softening of High-Strength Concrete", *ACI-Materials Journal*, V.89, No.1, pp.54-60.
- Takeuchi M., Hiramoto M., Kumagai N., Yamazaki N., Kodaira A. and Sugiyama K. (1993): "Material Properties of Concrete and Steel Bars at Elevated Temperatures", *Proc. SMiRT-12*, Paper H04/4, Ed. by K. Kussmaul, Elsevier Science Publ., pp. 133-138.
- Thienel K.C. and Rostasy F.S. (1993): "Influences of Concrete Composition on Strength and Deformation under Uniaxial and Biaxial Loading at Elevated Temperatures", *Proc. SMiRT-12*, Paper H04/6, Ed. by K. Kussmaul, Elsevier Science publ., pp. 145-150.
- Toutanji H., Matthewson P.R., Effinger M. and Noumowe A. (2003): "Zero-Eccentricity Direct Tension Testing of Thermally -Damaged Cement-Based Materials", *Cement and Concrete Research*, No. 33, pp. 1507-1513.
- Zhang B. and Bicanic N. (2002): "Residual Fracture Toughness of Normal- and High-Strength Gravel Concrete after Heating to 600°C", *ACI-Materials Journal*, V.99, No.3, pp. 217-226.
- Zhang B., Bicanic N., Pearce C.J. and Balabanic G. (2000a): "Residual Fracture Properties of Normal- and High-Strength Concrete Subject to Elevated Temperatures", *Magazine of Concrete Researches*, V. 52, No. 2, pp. 123-136.
- Zhang B., Bicanic N., Pearce C.J. and Balabanic G. (2000b): "Assessment of Toughness of Concrete Subject to Elevated Temperatures from Complete Load-Displacement Curve - Part I: General Introduction" / Part II: Experimental Investigations", *ACI-Materials Journal*, V. 97, No.5, pp. 550-555 and 556-566.
- Zhou X. and Zhang J. (2001): "Preliminary Fatigue Tests on Concrete after the Exposure to Temperature of up to 300°C", *Advances in Structural Engineering*, V.4, No.4, pp. 197-201.

7 Post-fire investigation and repair of fire-damaged concrete structures*

7.1 Introduction

Each single compartment exhibits its specific damage pattern after a fire and requires, as soon as possible, a thorough inspection in order to obtain an answer to the following basic questions:

- is it possible to repair and refurbish the damaged building or structure?
- is it necessary to demolish the building or structure, and to build it anew?

In both cases, the answer should come after the assessment of the damage, on the basis of a cost-benefit analysis, in order to allow the owner to take the best decision.

In the following "repair" covers all the measures, that are necessary to bring the compartment or the building back to its original state and to allow it to work safely in accordance with its destination. Therefore the repair measures of fire-damaged concrete structures comprise the following steps:

- collection of data concerning the fire event and its consequences;
- examination of the damage (due to fire and to fire-extinguishing activities);
- classification of the damage;
- identification and selection of the most appropriate repair methods.

7.2 Data collection

After the end of the fire and the cooling down of the compartment, or - in other words - when the compartment becomes accessible, any possible piece of evidence concerning the evolution of the fire must be carefully collected before the clearing works are started.

Especially the location and the state of the many substances subjected to the fire, inside the compartment* (such as steel, nonferrous metals, plastics, fibres and timber), give a lot of information about the maximum temperature reached in each point and fire duration.

In Table 7-1 some information is given about the behaviour and the state at high temperature of different materials usually found in a building.

7.3 Damage analysis

7.3.1 Concrete

One method to determine the depth of the thermal deterioration is to locally break off in small pieces the concrete of some selected members, layer-by-layer, by means of a hammer and a chisel, looking for the colour of the concrete from pink (close to the surface) to gray (far from the surface). Another method is to drill small cores and to analyse them.

The depth of the discoloured zone can be regarded as the 300 °C - isotherm. Besides, drill powder may be investigated with thermal analysis and small cores by mercury porosimetry. Other properties may be investigated in situ using the rebound hammer, the drilling-resistance test machine, ultra-sound velocity methods and/or mechanical methods with bigger cores (for

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evaluating such parameters as compressive strength, tensile strength and modulus of elasticity), to have a more complete picture of concrete residual properties (see also Chapter 5). Then the temperature distribution should be studied numerically, in order to make comparisons between the calculated isothermal line 300°C and the observed depth of the damaged zone.

One should also look after the chloride content and the contamination of reinforced-concrete surfaces, to determine the chloride ingress profile.

Table 7-1: Effect of high temperatures on materials commonly found in buildings

Substance	Typical examples	Conditions	Approximate temperature °C
Polystyrene	Food-container foam, light shades, handlers, curtain hooks, radio casings	Collapse Softening Melting and flowing	120 120 – 140 150 – 180
Polyethylene	Bags, films, bottles, buckets, pipes	Shrinking Softening and melting	120 120 – 140
Polymethyl-methacrylate	Handles, covers, skylights, glazing	Softening Bubble formation	130 – 200 250
PVC	Cables, pipes, ducts, linings, profiles, handles, knobs, houseware, toys, bottles	Degradation Smoke Brown colour Charring	100 150 200 400 – 500
Cellulose	Wood, paper, cotton	Dark colour	200 – 300
Solder	Plumbing joints	Melting	250
Lead	Plumbing Sanitary devices, toys	Melting Rounding of sharp edges Drop formation	300 – 350
Aluminium and light alloys	Fixtures, brackets mechanical parts and items	Softening Melting Drop formation	400 650
Glass	Glazing, bottles	Softening, rounding of sharp edges, flowing	500 – 600 800
Silver	Jewellery, spoons cutlery etc.	Melting Drop formation	950
Brass	Locks, taps, door handles, clasps	Melting (particularly at edges) Drop formation	900 – 1000
Copper	Wiring, cables, ornaments	Melting	1000 – 1100
Cast iron	Radiators Pipes	Melting Drop formation	1100 – 1200
Zinc	Sanitary devices, gutters, down pipes	Drop formation Melting	400 420
Bronze	Windows, fittings, door bells, Ornaments	Rounding of the edges Drop formation	900 900 – 1000
Paints		Deterioration Destruction	100 250
Wood		Burning, ash formation	240

7.3.2 Reinforcing and prestressing steel

The residual deformations of the reinforcing bars and of the prestressing tendons should be checked and mapped, and the smoke and soot particles (that may be corrosive) should be analysed. It is advisable to investigate the residual properties of the steel, by clearing a few bars from the surrounding concrete and by cutting out a few samples to be tested in tension. In this way the full σ - ϵ -relationships can be determined.

In most cases the in-situ measurement of the hardness may be sufficient to have the required information on steel residual properties. In other cases a more complete screening is required, and coupling cut-out samples and hardness tests is necessary. Metallographic analyses on ground slices are required only in specific cases.

7.4 Diagnosis

Generally concrete structures (either made of plain concrete, or reinforced/prestressed) can be repaired even after a severe fire (see Section 7.9). However, for each element one should estimate and compare the repair costs with the costs for a complete or partial reconstruction. The larger the structure and the more severe the damage, the more likely the reconstruction of the whole structure or of some members.

Only a specific investigation allows to understand whether the residual deformations can be tolerated or more comprehensive measures are necessary for the rehabilitation (such as treating the cracks for static and durability purposes).

For concrete it is possible to define "damage factors" (see Figs. 7.2 and 7.3). For instance, in all the regions, where the temperature did not exceed 100 °C, the damage factor is 1, while it is 0.85 in all the regions, where the maximum temperature was between 100 °C and 300 °C, 0.4 everywhere the maximum temperature reached 300 °C to 500°C, and 0.0 in all the regions where the temperature exceeded 500 °C.

The same applies to the reinforcement, that is practically unaffected by the temperature up to 400°C. For temperatures above 400°C, the residual properties of hot-worked carbon steel are better than those of tempcore steel, of cold-worked carbon steel and of cold-drawn prestressing steel, but are worse than those of hot-rolled stainless steel. However, to take care of the mechanical decay of the reinforcing bars, additional reinforcement may be added.

As for prestressed concrete, on the whole it is more fire-sensitive than ordinary concrete, but it depends on the prestressing system (pre-tensioned members are more heat-sensitive than post-tensioned members) and on the type of the section. However, a comprehensive investigation on the residual load-bearing capacity and on the deformations is mandatory, because of the reduction of concrete Young's modulus and of the great sensitivity of cold-drawn tendons and strands to high temperature.

In general, had the original design barely respected the safety margins, the occurrence of a fire could damage the structure beyond the level that is considered acceptable for its rehabilitation.

Sometimes, repairing a fire-damaged prestressed structure can be achieved by changing the load-bearing system from prestressed concrete to reinforced concrete. In other cases, structural repairing can be achieved by adding prestressing tendons to heat-damaged RC structures.

7.5 Damage classification

The effects of high temperature and fire on buildings and structures can be characterized by introducing a few "classes" (Table 7-2). This classification makes it possible to define different strategies for the future use of the damaged building or structure:

- complete repair
- combination of partial repair and partial reconstruction
- change of the destination or use
- demolition and rebuilding

7.6 Repair criteria

The main objective of repairing fire-damaged concrete structures is to bring back the structure to its original state and destination, through the following steps:

- the reinforcement should be refurbished and protected, and the concrete sections should be brought back to their original size;

- the repaired structure should have the same residual life as before the fire;
- the repaired structure should have the same load-bearing capacity as before the fire;
- the repaired structure should meet the same fire-safety requirements as before the fire.

Table 7-2: *Classes of damage*

Class	Characterization	Description
1	Cosmetic damage, surface	Characterized by soot deposits and discoloration. In most cases soot and colour can be washed off. Uneven distribution of soot deposits may occur. Permanent discoloration of high-quality surfaces may cause their replacement. Odors are included in the class (they can hardly be removed, but chemicals are available for their elimination).
2	Technical damage, surface	Characterized by damage on surface treatments and coatings. Limited extent of concrete spalling or corrosion of unprotected metals. Painted surfaces can be repaired. Plastic-coated surfaces need replacement or protection. Minor damages due to spalling may be left in place or may be replastered.
3	Structural damage, surface	Characterized by some concrete cracking and spalling, lightly-charred timber surfaces, some deformation of metal surfaces or moderate corrosion. This type of damage includes also class 2 damages, and can be repaired in similar ways.
4	Structural damage, cross-section	Characterized by major concrete cracking and spalling in the web of I-beams, deformed flanges and partly charred cross-sections in timber members, degraded plastics. Damages can be often repaired in the existing structure. Within the class are also (a) the large structural deformations that reduce the load-bearing capacity, and (b) the large dimensional alterations, that prevent the proper fitting of the different substructures and systems into the building. This applies in particular to metallic constructions.
5	Structural damage to members and components	Characterized by severe damages to structural members and components, with local failures in the materials and large deformations. Concrete constructions are characterized by extensive spalling, exposed reinforcement and damaged compression zones. In steel structures extensive permanent deformations due to diminished load-bearing capacity caused by high temperature. Timber structures may have almost fully charred cross-sections. Mechanical decay in materials may occur as a consequence of the fire. Class 5 damages usually will cause the dismissal of the structure.

7.7 Repair methods

Depending on the damage class, repairing should be performed with one - or more - of the following techniques:

- Cleaning and aesthetical renovation.
- Repair of concrete surfaces by using approved materials (polymer-modified mortars and coatings).
- Repair of concrete members and recreation of the original shape (for instance in damaged sections) by using shotcrete (according to DIN 18551 or to equivalent EN or ISO standards).
- Replacement of single elements (in the case of steel or prefabricated-concrete members).
- Addition of extra reinforcement, by using glued carbon- or glass-fibre laminates (FRP).
- Addition of extra fire-safety equipments.
- Repair of concrete cracks by injecting resins or cement slurries.
- Pull-down of the structure and build it anew.

In the case of concrete members, first all the external temperatures in excess of 300°C should be removed according to the following techniques:

- Chiselling and sand blasting.
- Cleaning with high-pressure water jets.

Then the tensile strength of the new concrete layers should be assessed, for instance by means of pull-off tests. The correlation between the pull-off load and concrete tensile strength makes it possible to have sufficient information on concrete strength for the application of mortar and protective coatings, as well as for the instalment (if required) of anchors of different types. The following pull-off strengths are generally considered adequate (Table 7-3).

Table 7-3: Minimum values for the pull-off strength of the concrete substrate

System	Mean value [N/mm ²]	Min. single value [N/mm ²]
Concrete replacement	1.5	1.0
Coating (paint without fine mortar)	1.0	0.6
Coating systems with fine mortar	1.3	0.8
Coating systems under motorcar lanes	1.5	1.3

Should the sections be brought back to the initial size and shape by using shotcrete, the following steps would be appropriate:

- Instalment of auxiliary moulds (if necessary).
- Placement of additional reinforcement as required by the load-bearing capacity.
- Washing and moistening of the substrate.
- Shotcreting in subsequent layers not thicker than 30 mm each.
- Special treatments (if necessary) of the surfaces after shotcreting.
- Application of a mortar layer (if requested for architectural reasons).

The application of special adhesives between the subgrade (= original undamaged concrete) and the shotcrete is generally superfluous, since the rebound of the coarse aggregates contained in the shotcrete leaves a thin transition layer between the subgrade and the shotcrete. Since this transition layer is very rich in fine aggregates, its "bridging" properties (between the original concrete and the new concrete) are very good.

When using shotcrete, there is no need to apply specific anti-corrosion products to the reinforcement, because shotcrete is very similar to ordinary cast-in-situ concrete. It is true that shotcrete is an extremely-dense material, that may spall under fire, but several additives have been developed - and are available on the market - aimed at preventing shotcrete spalling at high temperature.

The repair with the aid of repair mortars requires the following steps:

- Painting of the cleaned bars (a) with an anti-corrosion epoxy-based coating (containing corrosion inhibitors and pigments), or (b) with a slurry (containing Portland cement, sand and styrol-butadien acrylate), that later solidifies and becomes elastic.
- Application of an adhesive layer on the original cleaned concrete, to favour the cohesion between the subgrade and the mortar. (This layer may be based either on epoxy resin or on a polymer-modified cementitious mortar).

- Recreation of the original size and shape of the member in question, by using polymer-modified repair mortars, that should fulfill the following requirements:
 - Pull-off strength larger than that of the subgrade.
 - High capability of remaining stuck to the subgrade (high sticking capability).
 - Good capability of retaining the water added to the mix (as in ordinary cementitious materials).
 - Same strength as in the subgrade (= original undamaged and cleaned concrete).
 - Same thermal expansion as in the subgrade.
 - Modulus of elasticity from 1/3 to 1/4 of that of the subgrade.
 - Frost resistance.
 - Good handling in ordinary building-site conditions.
- For applications on small thin spots also polymer-bound concretes are now available.

The previously-mentioned mortars have been available for the last twenty years: they are obtained by mixing together water, fine/medium aggregates and a soluble polymeric powder. These mortars are generally prepared by using different max.-size aggregates (d_a between 1-2 mm and 8 mm). The mortar should be applied in layers when the damaged areas are relatively deep (max. thickness of each layer close to 30 mm; d_a up to 8 mm). When the damaged areas are shallow, smaller aggregates should be used ($d_a = 1-2$ mm).

Whenever necessary, a fine mortar should be applied as a finish, to close the surface pores and to adjust each member to the contiguous members.

Last but not least, a protective layer can or must be applied, depending on specific requirements and specific types of exposure.

The many coatings available today can be classified as shown in Table 7-4.

In some cases, crack filling with special resins or mortars to re-establish structural continuity is required by statics.

Table 7-4: Description of the various protective surface coatings

Description	Main type of binder
Hydrophobic Impregnation	Silan, Siloxane, Silicon resins
Coatings for non accessible surfaces with very low crack-bridging capability	(a) polymer (b) polymer/cement mix
Coatings for non accessible surfaces with low crack-bridging capability	(a) polymer/cement mix (b) polymer dispersion
Coating with enhanced crack-bridging capability for non accessible surfaces	(a) polyurethane (b) two-component polymethylmethacrylate-modified epoxy resins
Coatings below bituminous or other protective layers on accessible surfaces	Polyurethane
Coatings with low or high bridging capability for accessible surfaces	(a) multilayer epoxy resins (b) epoxy resins and polyurethane resins

7.8 Real fires

7.8.1 Warehouse in Ghent

In 1974 a fire occurred in a warehouse in the port of Ghent, which was half filled with cotton bales. The 3-storey cast-in-situ R/C building, measuring 50x50m in plan, largely satisfied all criteria regarding minimum cross sections and concrete covers. Nevertheless, after about 80' of fire exposure, part of the building started to collapse. The fairly deep beams in the fire zone were heated along three faces and showed a sizable longitudinal expansion. This expansion, being restrained by the surrounding unheated structure, mainly occurred in one direction. Consequently, shear failure of several columns (Fig. 7-1) occurred, resulting in

the collapse of a substantial part of the building (Fig. 7-2). Computer simulations showed that the collapse occurred after the temperature in the beams increased - on average - by 150-200°C.

The fact that axial restraint forces can reach very high values is illustrated by the test results shown in Fig. 7-3. Fire tests were performed on precast TT sections for various values of the longitudinal expansion (Issen et al., 1970). It can be noticed that axial restraint forces up to 2.5 MN were measured. In this case, the axial forces were favourable because their line of thrust was located close to the intrados of the specimens. The same favorable situation occurs in the support conditions shown in the left part of Fig. 7-4, but not in the case of the support geometry shown in the right part of Fig. 7-4 where the restraint force causes additional deflections in the beam or slab. The values of the thrust shown in Fig. 7-3 were measured before any significant relaxation of the concrete occurred. In the case of protracted fires, relaxation effects prevail and reduce the effects of end restraints.

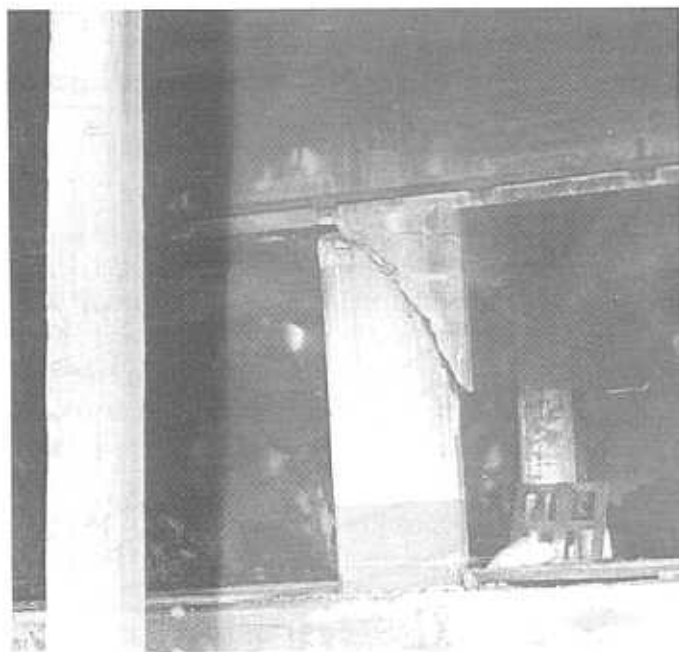


Fig. 7-1: Shear failure in a column due to axial restraint



Fig. 7-2: Fire-induced collapse of a warehouse in the port of Ghent

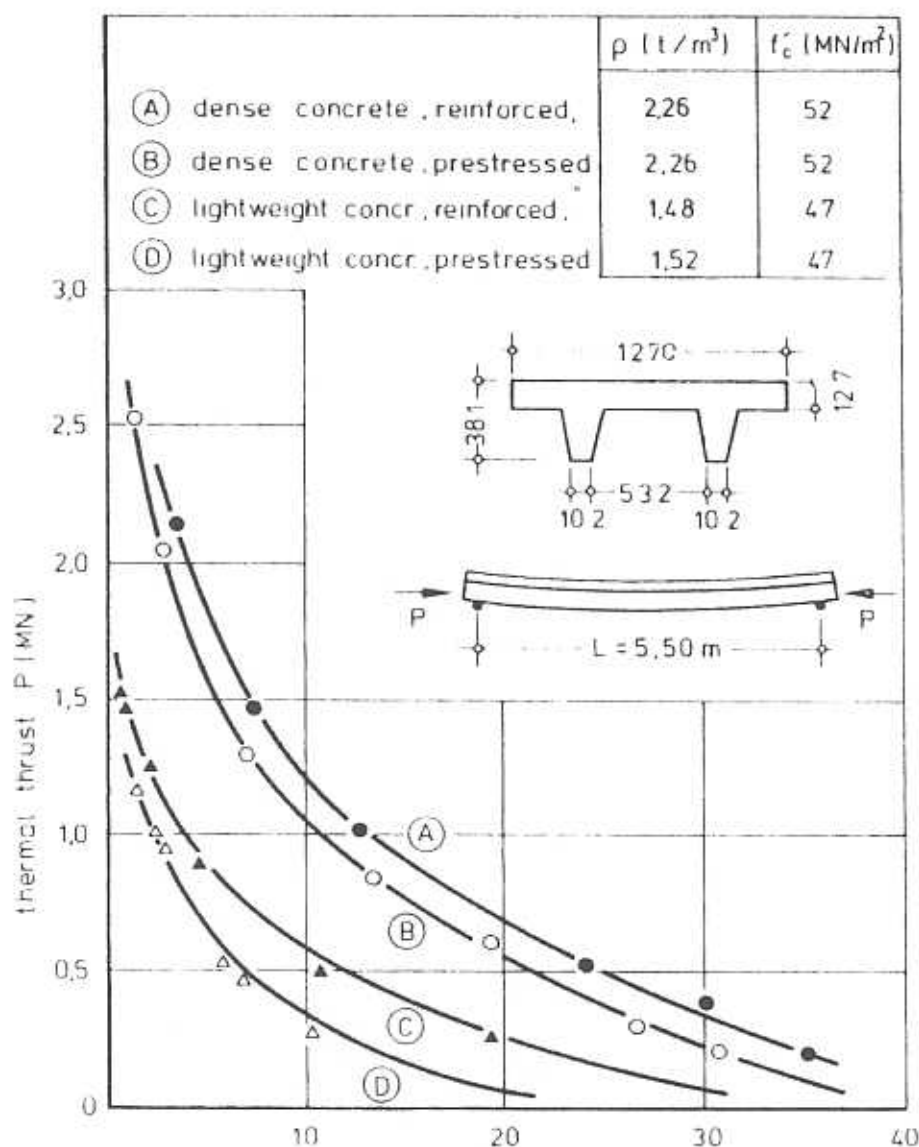


Fig. 7-3: Thermal restraint force on specimens with restricted longitudinal expansion (Issen et al., 1970)

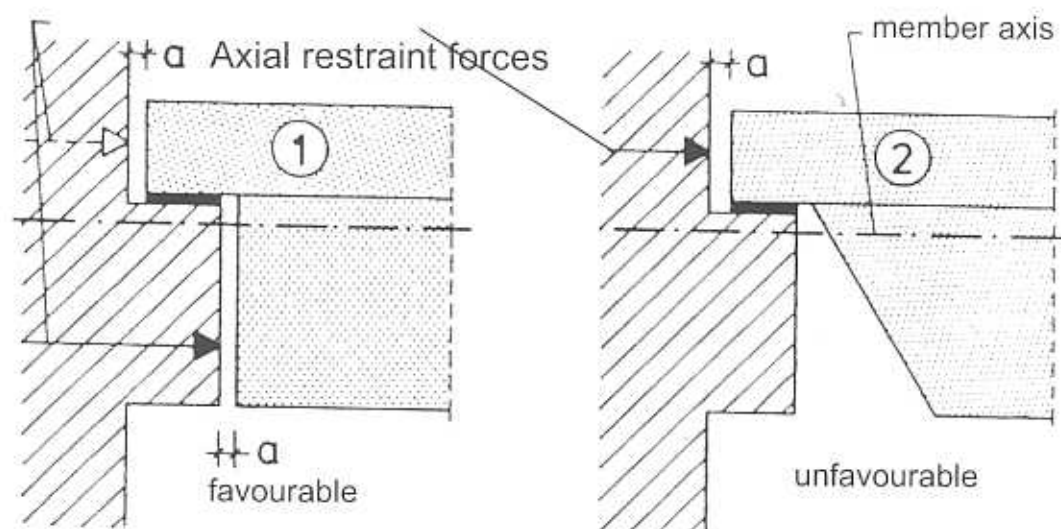


Fig. 7-4: Favourable (1) and unfavourable (2) support geometry with respect to axial restraints (Kordina et al., 1981)

7.8.2 Library in Linköping

In 1996, a similar collapse occurred in the city library of Linköping, Sweden (Anderberg et al., 1996), which was exposed to a severe fire, that spread quite rapidly. The two-storey part of the building collapsed after 30 minutes, although the design fire resistance was 60'. The building consisted of cast-in-situ concrete columns, structurally connected to the beams, to the floor (ground level) and to the roof slabs (2nd storey and part of the 1st storey).

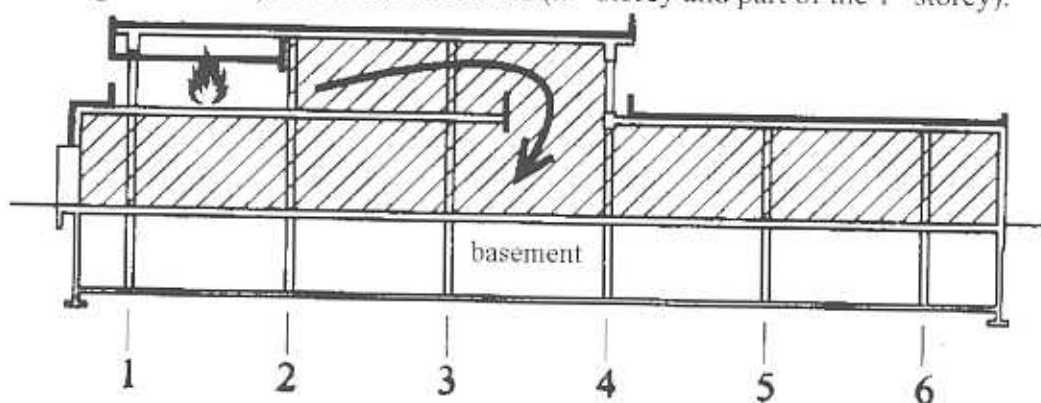


Fig. 7-5: Cross-section of the city library of Linköping (Anderberg et al., 1996)

The main reason for the early collapse is attributed to the fact that there was a 52 m long and 3.6 m wide opening in the slab of the first floor (see cross section in Fig. 7-5) along a one-storey and a two-storey part of the building (Anderberg et al., 1996). Because of this opening, the ground floor and the first floor were combined into one fire compartment (dashed zone in Fig. 7-5). Moreover, the opening acted as a "chimney" during the fire resulting in an intense two- or multi-sided heating of the edge beams along the opening, as well as in the neighbouring parts of the first floor. After the closure of a 30 mm-wide expansion joint, due to the thermal elongation of the adjacent floor parts, the further thermal expansion was restrained. The restraint compressive forces from the floor slab and the roof deformed the columns. Finally the compressive forces caused a shear failure occurring at the top of a critical column (Fig. 7-6) and a sudden shear failure in the main stabilizing walls. These failures led to the progressive collapse of a part of the building.

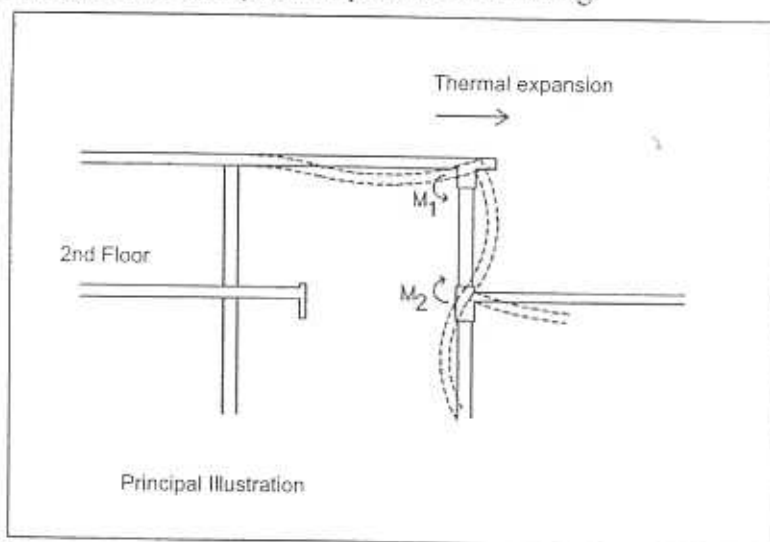


Fig. 7-6a: Illustration of the principle of the impact of roof rotation on the common edge columns

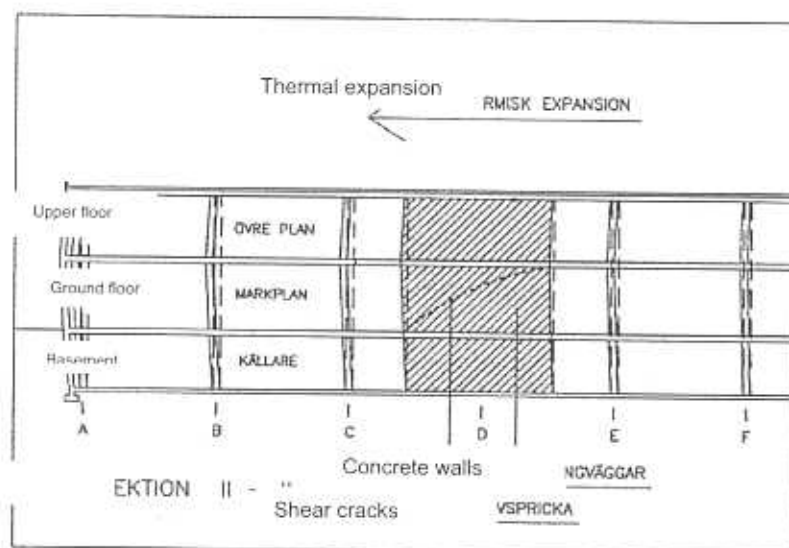


Fig. 7-6b: Thermal expansion in longitudinal direction after shear failure in the horizontally stiffening walls in the ground floor

In Figs.7-7a,b the calculated temperature profile over the cross section of a slab (after 30min of ISO 834 fire), and the calculated mean slab temperature (as a function of fire duration) are plotted. The latter plot shows that after a fire duration of 30min the mean calculated temperature is about 165°C. The normal force is very dependent on the joint gap. Here the gap was presumably about 30 mm and the total horizontal force for the 17 m wide floor was in the magnitude 5000 – 6000 kN, which finally exceeded the shear capacity of the walls. With a joint gap of 70 mm or a modified structural system, it may have been possible to prevent the failure mechanism.

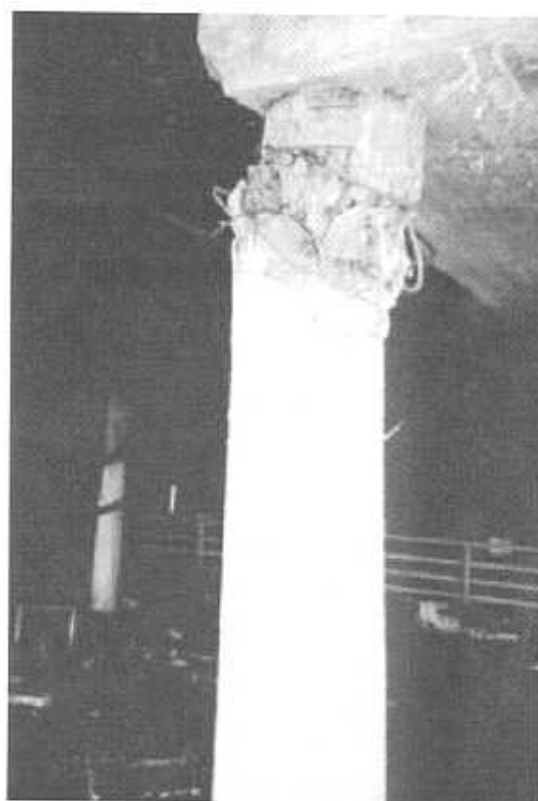


Fig. 7-6: Shear failure at the top of a column (Anderberg et al., 1996)

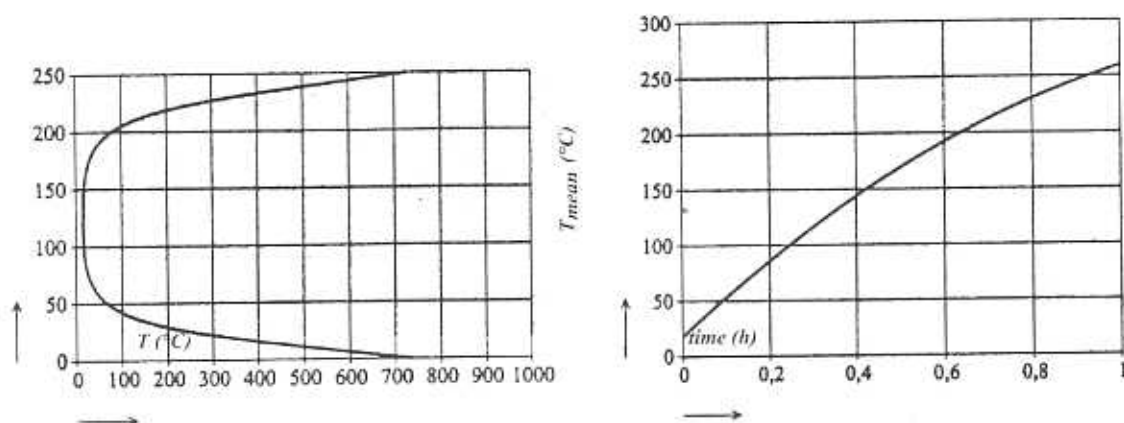


Fig. 7-7: Calculated temperature profiles in a floor slab after 30' fire exposure according to ISO 834 fire (a), and evolution of the mean slab temperature (b) (Anderberg et al., 1996)

7.8.3 Windsor building (Madrid)

The Windsor building, located in the commercial and financial centre of Madrid, was built in 1974-1979 (Fig. 7-8). Among the 37 storeys, 5 were underground and 2 were technical storeys (Fig. 7-10). From the third storey up, the in-plan section was rectangular (40 x 26 m). The tower was built around a central reinforced-concrete core, that housed the lifts and the stairways. The steel columns placed along the façades from the 3rd storey up were supported by the technical storeys. The internal columns were RC columns.



Fig. 7-8: Windsor as built [Calavera et al. (2005)]



Fig. 7-9: Windsor building during the fire of 25 February 2005

The rehabilitation works in progress since August 2003 included the upgrading of the existing structure to meet the most recent fire-safety legislation. Specifically, all the steel parts of the structure were being fireproofed, a precaution that was not required when the tower was designed and built. When the fire broke out, the structural members above the 9th storey and all the floors above the 2nd technical storey had yet to be fireproofed (Calavera et al., 2005).

In the night of 25 February 2005, the fire started at the 21st floor and spread throughout the building until it was extinguished roughly 26 hours later. All the storeys above the 4th storey

were affected. Because of the rehabilitation works, no occupants were inside the building. Figs. 7-9 and 7-11 show the upper part of the building during and after the fire, respectively. The steel columns on storeys 18 to 27 had collapsed. Due to their fall, nearly all the perimeter floor slabs fell down and the debris accumulated on floor 17, i.e. on top of the second technical floor. Since this concrete floor was stiffened by solid RC slabs and deep beams, it was sufficiently strong to resist the impact of the falling debris and of their accumulated weight. Thus complete collapse of the building was avoided.

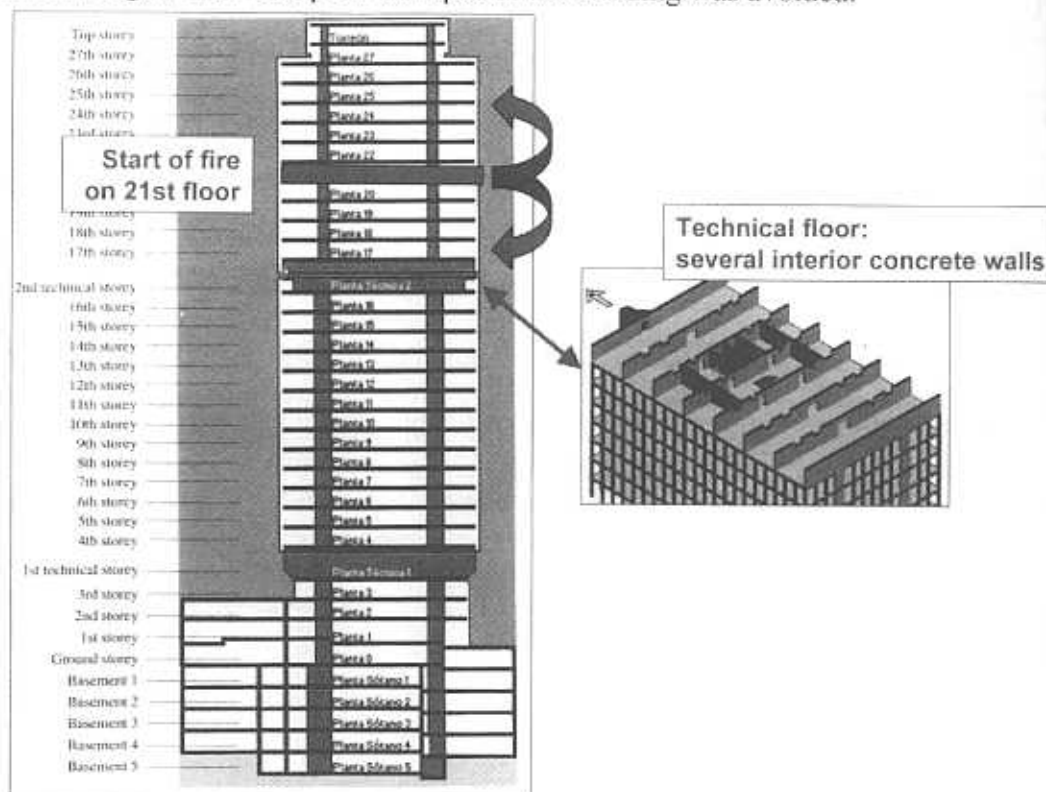


Fig. 7-10: East-west cross-section of the Windsor building (Calavera et al., 2005)

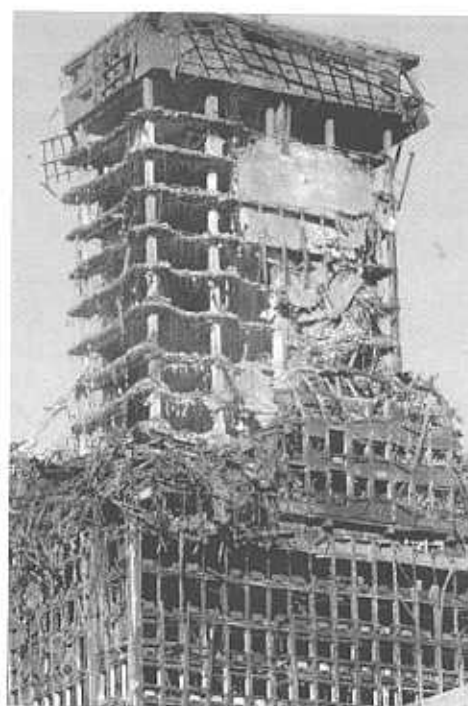


Fig. 7-11: Windsor building after fire

In light of the damage caused by the fire on the buildings, the Municipal Authorities of Madrid decided to demolish the buildings. Intermac was asked to prepare a proposal and to act as specialized consultant for the demolition works (Calavera et al., 2005). It was concluded that the concrete structure performed extraordinarily well under such a severe fire, and that the fireproofing of steel members, to guarantee their fire performance was absolutely necessary.

7.9 Repair of a pretensioned roof girder after a fire

7.9.1 Description of the building

In 1974, a full-scale fire test was performed on an industrial precast hall, designed and built especially for being tested in such severe conditions. This experiment was performed in order to gain first-hand knowledge on the fire behaviour of a typical building, with the structure composed of precast members. After the fire test, the building was dismantled and the structural assessment of several concrete members was performed. This paper focuses on the residual properties of a pretensioned roof girder and on its structural behaviour after being repaired by shotcreting.

A detailed description of the real-scale fire test can be found in (Almey et al., 1974, 1976, 1977). The building measured 12 m x 18 m in plan, had a free height of 6 m under the roof girders and entirely consisted of precast concrete members (Fig. 7-12). The load-bearing structure consisted of three portal frames, each consisting of two columns (cross section 400 mm x 400 mm) with a girder that supported the roof. The columns were anchored to the foundations (Fig. 12). Wood was selected as fuel. The total amount of wood used was 27 tons (125 kg/m^2) sawn in small beams, loosely stacked and dried. From the tests it appeared that the actual value of the calorific value was 14.42 MJ/kg . Consequently, the theoretical fire load was $1.8 \cdot 10^6 \text{ kJ/m}^2$.

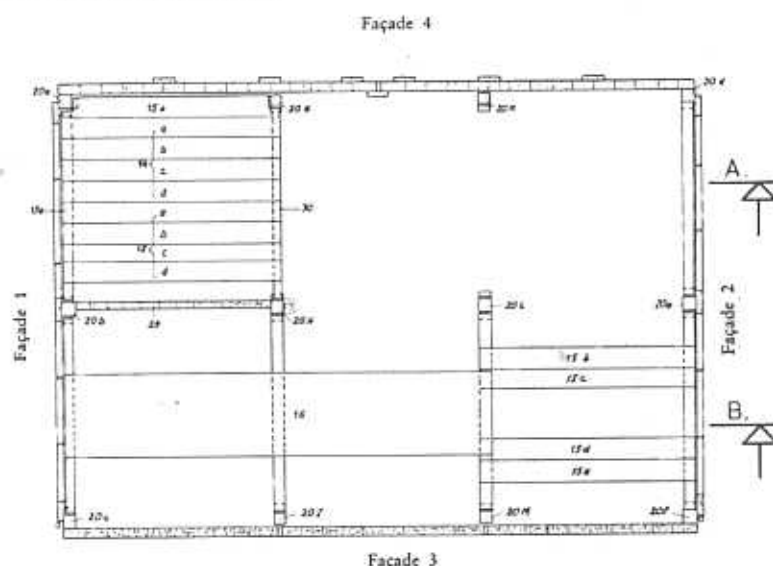


Fig. 7-12: Plan view of the industrial hall

7.9.2 Temperature development during the fire

The ignition of the wood stacks took about 2 minutes. Twenty minutes past the fire ignition, enormous smoke clouds could be observed. Opening of the doors and stirring up the fire by means of four fans was not sufficient to obtain a fully-developed fire. Only after 34 minutes, when the light-domes collapsed, the fire fully developed (with flames escaping

through the roof) and the inside temperature reached the level of the ISO-curve. From that moment on, the fire rate was roughly controlled by the closing and opening of the doors. In spite of the very high amount of fuel and of the ventilation of the building, the heating curve ISO 834 was followed only during 75' (from 20' to 95' after fire ignition, Fig. 7-13). After 70' no flames were seen through the openings of the building, and probably after 95' the average air temperature was decreasing at a relatively fast pace. Since then, reliable measurements were no longer available, because some thermocouples had fallen into the layer of burning charcoal, while in other thermocouples the wiring and the supports were destroyed. The fire died out after 120' by lack of fuel.

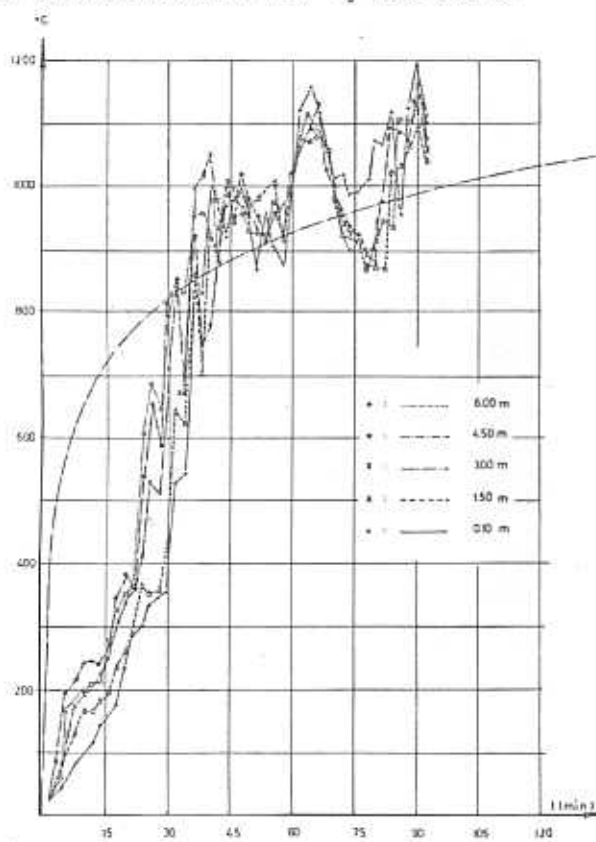


Fig. 7-13: Average air temperature at different levels inside the building

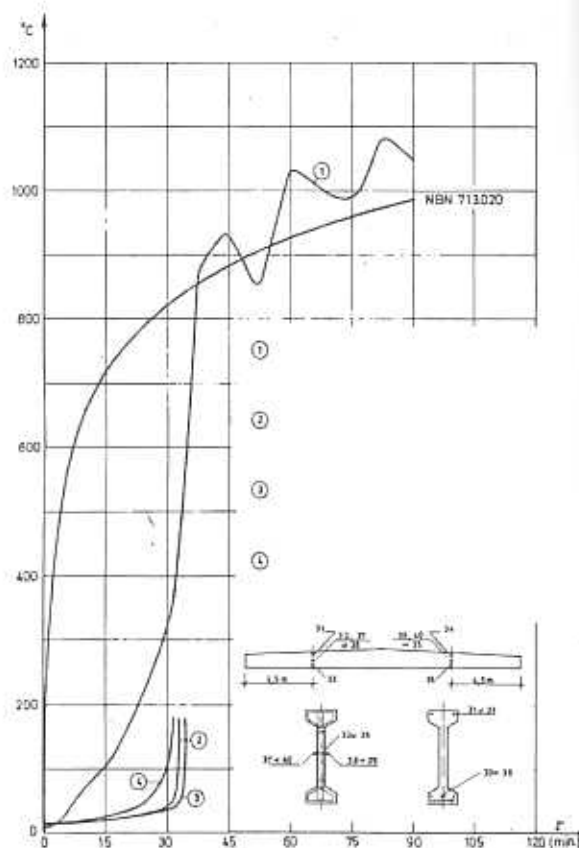


Fig. 7-14: Evolution of the measured temperature in the central roof girder

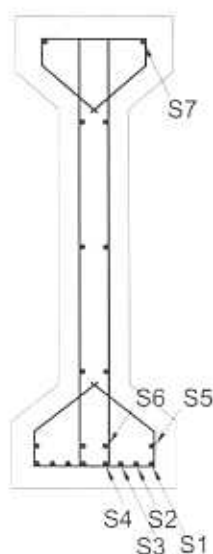
Fig. 7-14 shows the position of the thermocouples and the temperatures measured in the central roof girder of the building. The following comments can be made:

- The surface temperature increases very rapidly after 30' since fire ignition and follows closely the evolution of air temperature inside the building, with a delay that is smaller than for the other elements. (The smaller delay is probably related to the position of the girder inside the building).
- The average temperature of the reinforcement remains almost constant during 35', but then there is a sudden increase, most probably because of the detachment of the cover, due to concrete spalling. (During the test, this sudden increase was attributed to some unknown phenomenon and the measurements were considered to be unreliable).

7.9.3 Characteristics of the roof girder

One of the pretensioned roof girders, with a total length of 18 m, was removed from the test site and brought to Magnel Laboratory. The I-shaped cross-section had the following dimensions: total depth at midspan/at the supports = 1.2/0.75 m; width of the flanges = 0.4 m;

web thickness = 0.12 m (Fig. 7-15). Twelve 7-wire prestressing strands (nominal diameter 12.7 mm) were located in the lower flange and two strands in the upper flange. Fig. 7-16 shows one side of the girder, that was severely damaged during the fire test. Surface spalling could be observed in many zones. In some cases the spalling in the web reached a depth of several centimeters and at one point the very deep spalling led to the formation of a hole through the full thickness of the web (Fig. 7-16). In several zones, both the stirrups and the longitudinal rebars were visible, and in other zones the cover of some strands was also expelled.



Figures 7-15 and 7-16: Pre-tensioned roof girder – Cross section and view after the fire test

In the lab, the surface of the girder was cleaned and prepared by means of a pneumatic hammer and by gritblasting. Afterwards, the girder was repaired by shotcreting (Figs. 7-18 and 7-19).

7.9.4 Test of the roof girder under static loads

The repaired girder was tested under static loads with a span of 17.6 m. Four point loads were applied by means of hydraulic actuators (Figs. 7-17 and 7-20). The design service load of the girder (15 kN/m, dead weight excluded) was achieved by means of 4 equivalent point loads of 66 kN each.

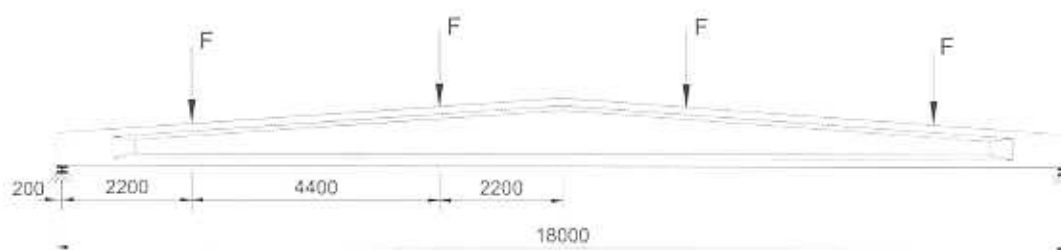


Fig. 7-17: Outline of the girder and of the loads in the static test

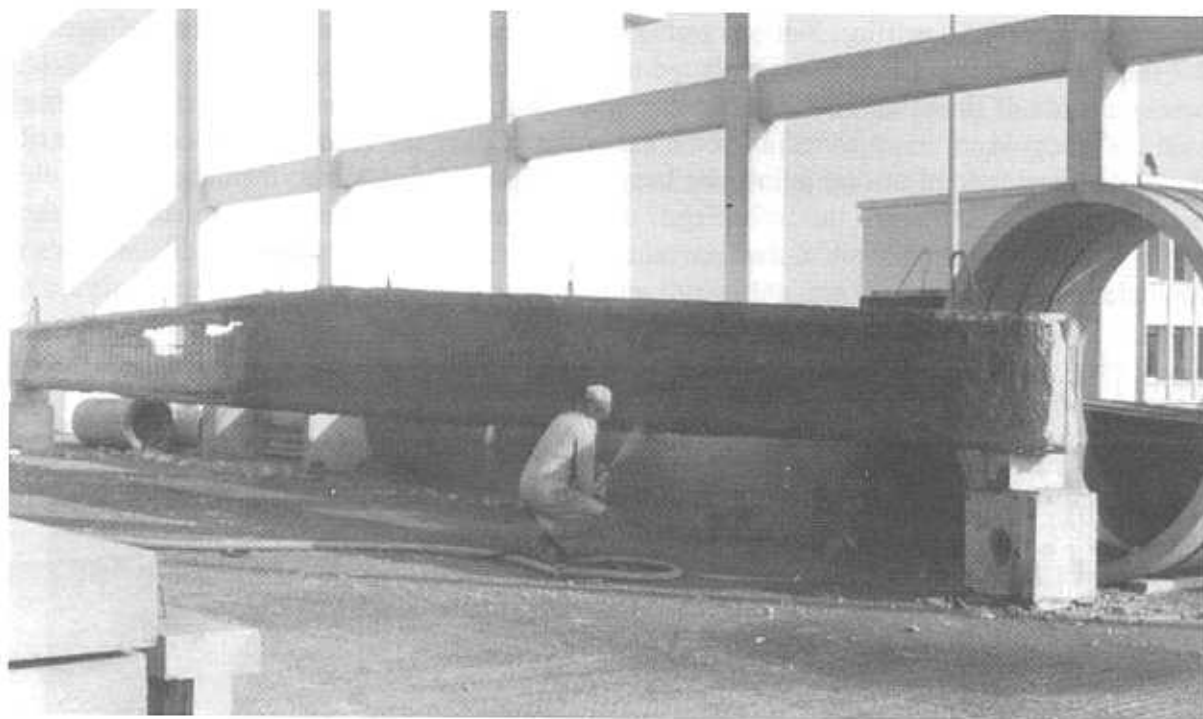


Fig. 7-18: Shotcreting of the damaged girder (Taerwe et al., 2006)



Fig. 7-19: Central part of the girder after shotcreting (Taerwe et al., 2006)

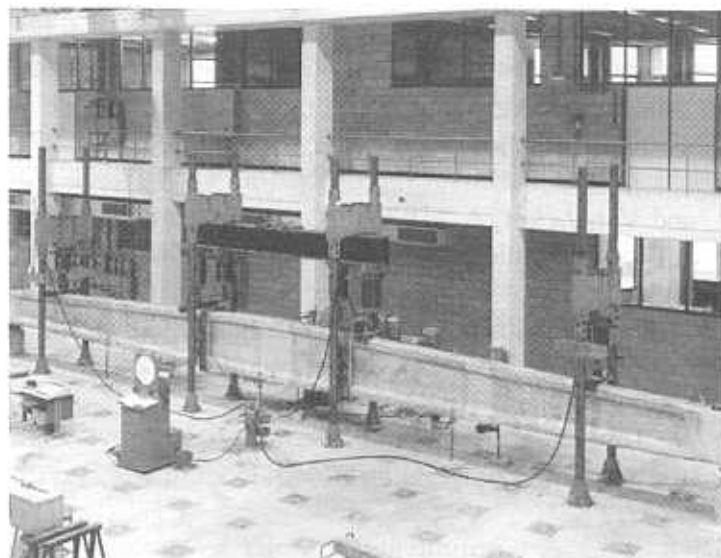


Fig. 7-20: Test set-up

During the static test, the four point loads were increased in steps of 5 kN each, up to the service load of 66 kN. After unloading, the girder was reloaded up to 66 kN with the previous procedure, and then the girder was brought to failure, by increasing the loads by 10 kN at each step.

Failure occurred at a load level of 162 kN per point load, which was 2.45 times higher than the service load. Crushing of the concrete in the upper flange occurred after yielding of the prestressing strands in a cross-section where a broken strand could be observed after the fire test (Fig. 7-21).

The calculated resisting moment of the repaired girder is $M_R = 1616$ kNm. The maximum bending moment in the critical section caused by the four point loads and the dead weight of the beam was $M_E = 1629$ kNm. Hence, a very good agreement is obtained between the maximum applied bending moment during the static test and the calculated value of the residual resisting moment.

This example shows that - although the girder was seriously damaged during the fire - still a sufficient safety margin could be achieved by applying an appropriate repair technique.



Fig. 7-21: Failure zone of the repaired roof girder

References

- Almey, F., Minne, R. and Van Acker, A. (1974). "La cellule d'essai de l'U.A.C.B. en proie aux flammes, De U.A.C.B.-proefcel werd de prooi der vlammen", *Beton*, 27, pp. 52-73.
- Almey, F., Minne, R. and Van Acker, A. (1976). "Les résultats d'un 'incendie pas comme les autres, De resultaten van een 'niet alledaagse brand », *Beton*, 34, pp. 41-50.
- Almey, F., Minne, R. and Van Acker, A. (1977). "Incendie expérimental d'un bâtiment industriel préfabriqué : rapport de synthèse, Experimentele brand van een geprefabriceerd industrieel gebouw : syntheseverslag" *Beton*, 40, pp. 35-69..
- Anderberg, Y. (1983). "Properties of Materials at High Temperatures, Steel", *RILEM Technical Committee 44-PHT*
- Anderberg, Y. and Bernander, K. (1996) "Biblioteksbranden I Linköping den 21 september 1996: studium av orsaken till tidigt ras", *Fire Safety Design AB (FSD)*, Lund.
- Bazant, Z.P. and Kaplan, M.F. (1996) "Concrete at High Temperatures: Material Properties and Mathematical Models", Harlow, Longman.
- Benedeth, A. and Mangon, E. (2004) "Damage Assessment in Actual Fire Situations by Means of Non-Destructive Techniques and Concrete Test", *Proceedings of the Workshop "Fire Design of Concrete Structures": What now? What next?*, Milan University of Technology, Milan..
- Calavera, J., Gonzalez-Valle, E., Cano J.L., Diaz-Lozano, J., Fernandez-Gomez, J., Ley, J., and Izquierdo, J.M. (2005) "Fire in the Windsor Building, Madrid : Survey of the fire resistance and residual bearing capacity of the structure after the fire", *INTEMAC report NIT2-05*.
- Drysdale, D.D. and Schneider, U. (Editors) (1990), "CIP W 14 Report, Repair ability of Fire-Damaged Structures", *Fire Safety Journal*, V. 16, pp. 251-336.
- Felicetti, R. (2004) "Digital Camera Colometry for disessment of Fire-Damaged Concrete", *Proceedings of the Workshop "Fire Design of Concrete Structures: What now? What next?* Milan University of Technology.
- Felicetti, R. (2004) "The Drilling test for the Assessment of the Thermal Damage in Concrete", *Proceedings of the Workshop "Fire Design of Concrete Structures": What now? What next?*, Milan University of Technology.
- Issen, L.A., Gustafarro, A.H. and Carlson, C.C. (1970), "Fire tests on concrete members : an improved method for estimating thermal restraint forces", *Fire test performance*, ASTM, STP 464, pp. 153-185.
- Kordina, K. and Meyer-Ottens, C. (1981), "Beton Brandschutz Handbuch", BetonVerlag, Düsseldorf.

Lennon, T. (2004) "Fire Engineering Design of Concrete Structures", *Proceedings of the Workshop "Fire Design of Concrete Structures": What now? What next?*, Milan University of Technology.

Malhotra, H.L. (1982) "Design of Fire-Resisting Structures", Survey University Press.

Schneider, U., Diederichs, U., Rosenberger, W. and Weiß, R. (1980), "Hochtemperaturverhalten von Festbeton", *Special Research Department 148 „Behaviour of Structural Elements Exposed to Fire"*, Braunschweig University of Technology, Report 1978-1980, Part II.

Schneider, U. (1986) "Properties of Materials at High Temperatures: Concrete", 2nd. Edition, Kassel, RILEM Technical Committee 44-PHT, Technical University of Kassel, 1986.

Schneider, U. (1988) "Concrete at High Temperatures – A General Review", *Fire Safety Journal*, Vol. 13.

Short, N.R., Purkiss, J.A. and Guise, S.E. (2000) "Assessment of Fire-Damaged Concrete", *Concrete Communication Conference 2000*, BCA, Crowthorne, pp. 245-254.

Short, N.R., Purkis, J.A. and Guise, S.E. (2002) "Assessment of Fire-Damaged Concrete using Crack Density Measurements", *Structural Concrete*, Vol. 3, pp. 137-143.

Short, N.R. and Purkiss, J. A. (2004), "Petrographic Analysis of Fire-Damaged Concrete" *Proceedings of the Workshop "Fire Design of Concrete Structures": What now? What next?*, Milan University of Technology.

Taerwe, L., Poppe, A.-M., Annerel E. and Vandeveld, P. (2006) "Structural assessment of a pretensioned concrete girder after fire exposure", *Proceedings of the 2nd fib Congress, Naples (Italy)*, 5-8 June 2006, Condensed Papers (2), ISBN-10: 88-89972-06-08, ISBN-13: 978-88-89972-06-9, pp. 236-237, full paper : CD 12 pp.

A1 Beam-column-floor connections*

A1.1 Introduction

A1.1.1 General

Because of the possible large deformations which may occur during a fire, the connections should be designed for a large ductility and rotation capacity. In precast structures, the solutions are somewhat simpler than in cast in-situ construction, because of the simple support conditions of most connections.

In cast in-situ structures, alternative connections should be studied to cope with the dilatation problem. The aim is to avoid shear failure, as happened in the buildings in Gent and Linköping. The problem is that as far as cast in-situ concrete structures are concerned, there has not been much research on the functioning of their connections under fire circumstances.

For both types of structures, connections are vital parts and shall be designed, constructed and maintained in such a way that they fulfil their functions and perform as required in the case of fire.

An axial restraint force near the top of the beam as shown in Figure A1-1 would lead to a premature failure of the floor system.

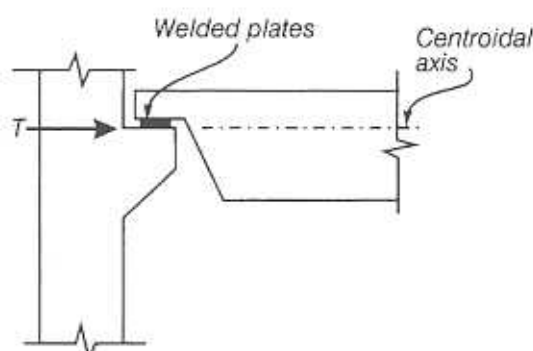


Fig. A1-1: The axial restraint would lead to failure (Buchanan, 2002)

It is essential that the line of thermally induced thrust is below the centroid of the compression region of the beam or the slab (Figure A1-2 (a) & (b)) so that the eccentricity (between the line of action of the thermal thrust and the centroid of the compression block near the top of the beam) has a positive value. In this case, the axial restraint has a beneficial effect.

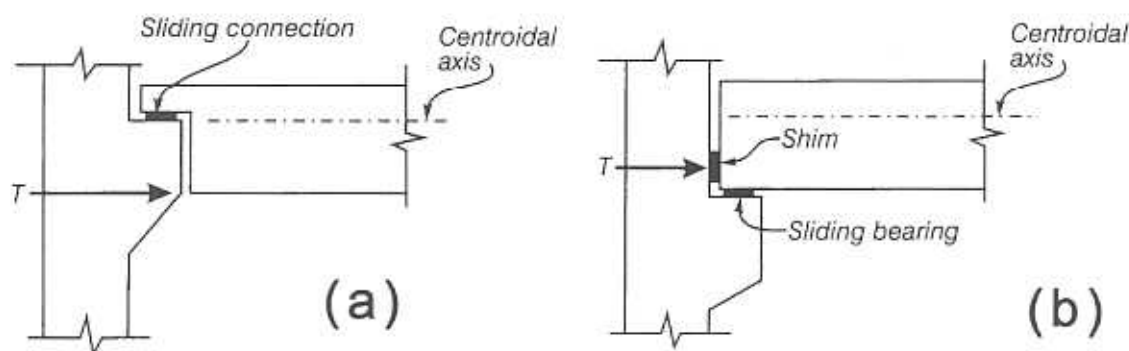


Fig. A1-2: The axial restraint has a beneficial effect (Buchanan, 2002)

* by Yahia Msaad and Fabienne Robert

A1.1.2 Literature review

The Working Group A-1: Element Tests of NIST (1997) recognizes the necessity of testing the connections for a better understanding and a possible prediction of the behaviour of structures under fire. However, tests on entire frames or joints between members are not pursued due to the size and the high cost associated with them.

A joint report from Cembureau/FIP (1979) indicates that the performance of a building structure in an actual fire depends not only on the theoretical functioning of each element, but also on the efficiency of detailing. It is necessary to consider whether the removal of one element can lead to the progressive collapse of the whole structure. It is recommended that:

- all main reinforcing bars should be properly anchored
- the top and bottom reinforcement in continuous beams should be continuous with effective overlaps at the supports (in order to avoid premature failure in the case that tension develops at the bottom steel, which was in compression before the fire occurred)
- some form of fire-stopping should be provided in the service ducts that penetrate a fire compartment wall or floor.

Furthermore, a range of codes examined, including the ISO 834 – Fire Resistance Tests: Elements of Building Construction; ASTM E 119 – Standard Test Methods for Fire Tests of Building Construction and Materials; JIS A 1304 – Method for Fire Resistance Test for Structural Parts of Buildings; New Zealand Building Code; Concrete Structures Standard NZS 3101:1995; Loadings Standard NZS 4203: 1992; Building Code of Australia 1996; Singapore Civil Defence Force Fire Code; and the Eurocodes, do not seem to provide guidelines for the design of connections.

ACI/TMS (1997) suggests that resistance to potential thermal expansion can be achieved when continuous concrete structural topping is used. Members should be designed in such way that flexural tension governs the design. In addition, negative moment reinforcement should be long enough to accommodate the complete redistributed moment (the required lengths of the negative moment reinforcement shall be determined assuming that the span being considered is subjected to its minimum probable load whereas the adjacent span(s) are loaded to their maximum unfactored service loads).

A1.1.3 Connections and fire indirect effects

In general, the normal permanent and variable actions on a structure will not give rise to specific design problems for connections, since the load level is mostly smaller at fire than in the normal situation and also lower safety factors are used for the ultimate state design because of the accidental character of fire. However this is not the case for indirect actions, where important alterations may occur, mainly affecting the connections.

A1.1.3.1 Increase of the support moment for restrained continuous structures

The thermal dilatation of the exposed underside of a beam or a floor, forces the member to curve, which in turn results in an increase of the support moment at the colder top side of the member, see also CEB (1991). The effect on the connection can be important, since the thermal restraint may lead to the yielding of the top reinforcement. However, precast structures are generally designed for simple supporting conditions, where the rotational capacity is large enough to cope with this action.

A1.1.3.2 Forces due to hindered thermal expansion

When a fire occurs locally in the centre of a large building, this expansion will be hindered by the surrounding floor structure, and very large compressive forces will generate in all directions. Experience from real fires learns that the effect of hindered expansion is generally less critical since concrete connections are generally capable to take up large forces. In any case it is recommended to take account of the phenomenon at the design stage.

A1.1.3.3 Large deformations due to cumulated thermal dilatations

When a fire covers a wide building surface and lasts for a long period, it may lead to large cumulated deformations of fire exposed floors and beams at the building structure edge. It is not unrealistic to assume that, in a large open store hall, the cumulated longitudinal deformation of a bay during a long fire, may amount to 100 mm and more. The rotational capacity of the connection between, for example, beams and columns at the edge of a building is a critical parameter for the stability of the entire structure.

A1.1.3.4 Local damage at the support (due to the eccentricity of the support reaction)

The curvature of beams and floors during a severe fire may have an influence of the location of the vertical force transferred through the connection. The edge of the supporting member might split off, when the contact between the supported and the supporting member is moving towards the edge of the latter. The problem can be solved by increasing the thickness of the bearing pad.

A1.1.3.5 Cooling effect after a fire

The cooling of a structure after a long fire may introduce tensile forces on the connections between long structural members. However, these effects are normally not taken into account at the design.

A1.2 Structural fire resistance

It is suggested (PCI, 1985) that many types of connections in precast concrete construction are not vulnerable to the effects of fire and as a result they do not require any special attention. Gravity-type connections, such as the bearings between precast concrete panels and concrete footings or beams that support them, do not generally require special fire protection. If the panels rest on pads made of combustible materials, protection of the pads is not usually needed since deterioration of the pads would not cause collapse.

The principles and solutions valid for the fire resistance of structural concrete components, apply also for the design of connections: minimum cross-sectional dimensions and sufficient cover to the reinforcement. The design philosophy is based on the large fire insulating capacity of concrete. Most concrete connections will normally not require additional measures. This is also the case for supporting details such as bearing pads in neoprene or another material, since they are protected by the surrounding components.

A1.2.1 Dowel connections

Simply support connections perform better during a fire than heavy continuous ones because of their higher rotational capacity. Dowel connections are a good solution to transfer horizontal forces in simple supports. They need no special considerations since the dowel is well protected by the surrounding concrete. In addition, dowel connections can provide additional stiffness to the structure because of the semi-rigid behaviour. After a certain horizontal deformation, an internal force couple is created between the dowel and the surrounding concrete, giving additional stiffness in the ultimate limit state. This is normally not taken into account in the design, but represents nevertheless a reserve in safety.

A1.2.1.1 Connections between superposed columns

Columns are often intervening in the horizontal stiffness of low rise precast concrete structures. This is generally the case for industrial buildings, where the horizontal stability is ensured by portal frames composed of columns and beams. The columns are restrained into the foundations and have a dowel connection with the beams. The horizontal blocking of possible large deformations depends on the stiffness of the column and the rigidity of the restraint. Experience with real fires has shown that such column connections behave rather well in a fire and do not lead to structural incompatibility.

In multi-floor precast structures, columns generally transfer only vertical forces, the horizontal rigidity being assumed by central cores and shear walls. The question to be answered with regard to the fire resistance concerns the choice whether to use single storey columns or continuous columns over several storeys. When a fire occurs at an intermediate floor, the horizontal blocking will be smaller in the case of single storey columns than with continuous columns (Fig. A1-3). The blocking in itself is not so dangerous, since it provides a kind of prestressing to the heat exposed structure, but the forces may lead to shear failure of the column itself. The latter phenomenon has effectively been stated in a real fire on a very rigid cast in-situ structure. The example shows that the connection between superposed columns may influence the indirect actions on the column.

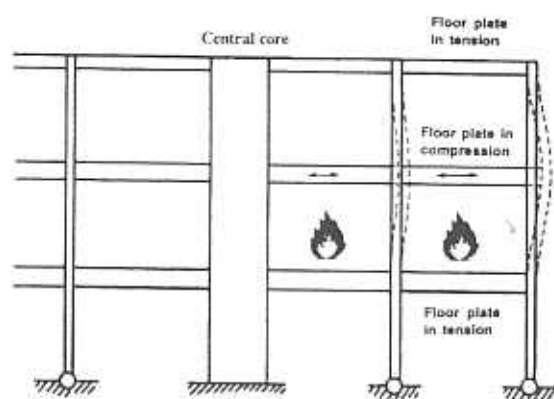


Fig. A1-3: Large horizontal forces on continuous columns due to floor dilatation

A1.2.1.2 Floor-beam connections

The connections between precast floors and supporting beams are situated within the colder zone of the structure, and hence not affected by the fire. The position of the longitudinal tie reinforcement (longitudinal means in the direction of the floor span) should preferably be in the centre of the floor thickness, or a type of hair-pin connecting reinforcement.

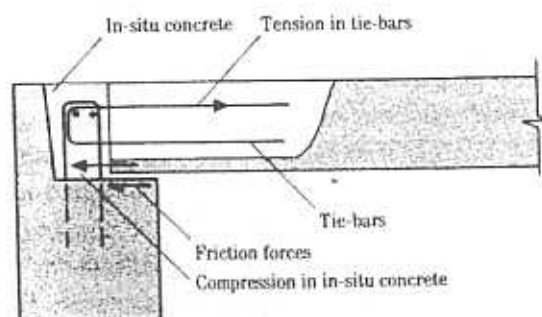


Fig. A1-4: Hair-pin connection at floor-wall or floor-beam support

In case of restrained support connections, the Eurocode prescribes the provision of sufficient continuous tensile reinforcement in the floor itself to cover possible modifications of the positive and negative moments.

A1.2.1.3 Floor-wall connections

Walls exposed to fire at one side will curve because of the differential temperature gradient. At the same time the supported floor will expand in the longitudinal direction. Both phenomena will lead to an increasing eccentricity of the load transfer between floor and wall, with a risk of collapse (Fig. A1-5). For masonry walls, only reinforced or confined masonry should be used, the strength of the wall should be increased, by providing internal ties in the slab that are efficiently connected with the peripheral ties in the walls.

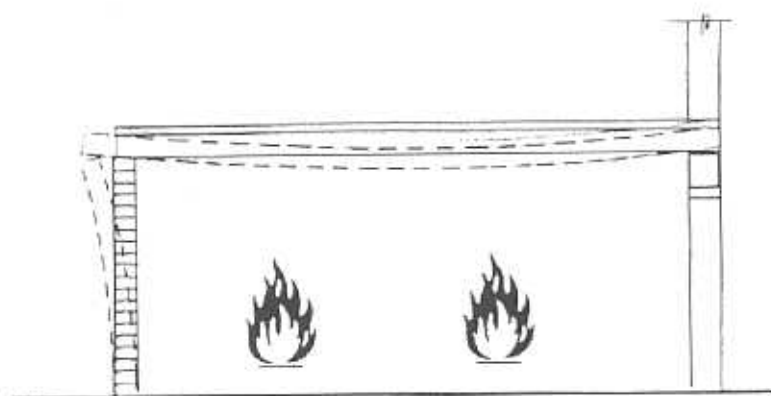


Fig. A1-5: Wall curvature and floor expansion may lead to large support eccentricity

A1.2.1.4 Hollow core slab connections

Experience during fire tests in laboratories has learnt that the structural integrity and diaphragm capacity of hollow core floors through correctly designed connections, which as a matter of fact constitutes the basis for the stability of the floor at ambient temperature, are also essential in the fire situation. Due to the thermal dilatation of the underside, the slab will curve. As a consequence, compressive stresses appear at the top and the bottom of the concrete cross-section and tensile stresses in the middle (explained in Chapter 2 : Fire action and design approach). The induced thermal stresses may lead to internal cracking. In principle, cracked concrete sections can take up shear as good as non-cracked sections. In

fact, the crack borders are rough and shear forces can be transmitted by shear friction and aggregate interlocking (Fig A1-6), on condition that the transversal wedging forces are balanced by an adequate reinforcement or by the surrounding structure. In hollow core floors, this transversal force is taken up by the transversal tie reinforcement at the support.

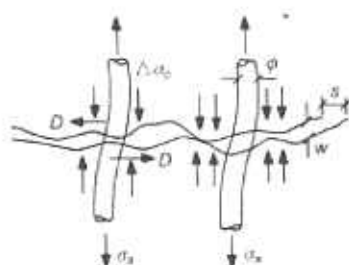


Fig. A1-6: Shear friction and interlocking

Also in fire conditions, the same principle remains valid. The decrease of the concrete strength at higher temperatures is hardly playing a role. Such temperatures appear only at the bottom part of the concrete section, and much less in the centre. From the foregoing, it appears that at the design stage, provisions are to be taken to realize the necessary coherence between the units in order to obtain an effective force transfer through cracked concrete sections. The fact that shear failures never were observed in real fires shows that there exist enough possibilities to realize this coherence between the units. As a matter of fact, this has also been proven in numerous fire tests in different laboratories. The possible design provisions are explained here after:

A1.2.1.5 Reinforcing bars in cast open cores

The reinforcing bars are in the first place designed to connect the floor units at the support construction. The reinforcing bars are placed in the central part of the cross-section, there where the thermal stresses appear. They are keeping the cracks closed. The effectiveness of such reinforcement in the preservation of the shear capacity of the units, at fire, has been proven over during tests in different laboratories.

A1.2.1.6 Reinforcing bars in the longitudinal joints

This is a variant solution of the above given connection with open cores. To transfer forces in an effective way, provisions must be taken to ensure a good anchorage of the bars in the joints. This presupposes that the joints remain closed, which can be realized through a good peripheral tie reinforcement. The real function of the latter is to ensure the diaphragm action of the floor and the lateral distribution of concentrated loadings, even through cracked joints. Indeed, the interlocking effect ensures the force transfer. The anchorage capacity of steel bars in cracked longitudinal joints between hollow core units has been extensively studied at Chalmers University of Technology, Göteborg [Engström (1992)]. It is recommended to limit the diameter of the bars to maximum 12 mm and to provide a larger anchorage length than normally needed, e.g. 1.50 m for a bar of 12 mm. When the above conditions are met, the reinforcing bars in the joints ensure the interlocking effect of the possible cracked concrete section, and hence the shear capacity of the units at fire. Also this case has been proven repeatedly in many ISO fire tests.

A1.2.1.7 Peripheral ties

As already mentioned, peripheral ties are playing an essential role with respect to the diaphragm action of the floor and the transversal distribution of concentrated loads. The peripheral ties are also contributing in a positive way to the preserving of the shear capacity of the units when exposed to fire. Indeed, the peripheral beam obstruct directly and indirectly the expansion of the floor, at one side through the rigidity of the tie beam itself together with the supporting construction, and at the other through the coherence between neighbouring floor units.

When fire occurs in the central part of a large floor, the thermal expansion of the units will be practically completely blocked by the rigidity of the surrounding floor. The blocking will mobilize important compressive forces in the fire exposed floor units. As a matter of fact, this has been stated at real fires, where sometimes large spalling occurred under the high compressive forces. In such cases, the central part of the cross-section will certainly not be cracked any longer through the differential thermal stresses, but the whole section will be subjected to compression. The shear capacity will therefore be unaffected.

A1.2.1.8 Steel connections

Steel connections, such as steel corbels and similar, shall be protected against the effect of fire, either by encasing them into concrete or by an adequate fire insulation. The concrete cover should be at least 30 mm for 1 hour of fire resistance and 50 mm for two hours. Precautions are to be taken to prevent spalling of the concrete cover by adequate reinforcement.

In case of partially encased steel profiles, for example in slim floor structures, the temperature rise in the steel profile will be slower than in non-encased unprotected profiles, due to the effect of the thermal conductivity of the surrounding concrete. However, it is recommended to protect the exposed steel flange by a fire insulating material.

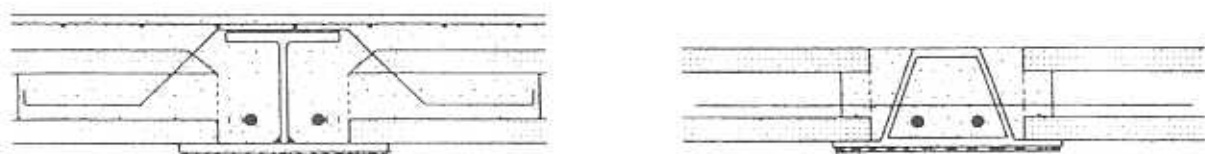


Fig. A1-7: Examples of slim floor structures

A1.3 Separating function

Requirements with respect to the separating function are expressed as limit states of thermal insulation and structural integrity against fire penetration. They apply mainly for joints between prefabricated floors, walls, or walls and columns, which should be constructed to prevent the passage of flames or hot gases.

Longitudinal joints between precast floor elements generally do not require any special protection. The precondition for thermal isolation and structural integrity is a minimum section thickness (unit plus finishing) according to the required fire resistance. Minimum dimensions are given in the Table 1.1 (according to CEB Bulletin n° 208 "Fire design of concrete structures"). The joint should also remain closed. The latter is realised through the

peripheral tie reinforcement. When the section is too small, for example due to the limited thickness of flanges of TT-floor elements, a special fire insulating joint strip can be used.

Table A1-1: Minimum joint thickness h_j

Standard fire resistance	Minimum joint thickness (mm)	
	Dense aggregate concrete	Lightweight aggregate concrete (1.2 t/m ³)
R 30	60	60
R 60	80	65
R 90	100	80
R120	120	95

Joints between walls and columns can be made fire tight by either a connecting reinforcement at half height, or through a special profile of the column cross-section (Fig. A1-8).



Fig. A1-8: Connection between column and wall

References

- ACI/TMS (1997). Committee 216, Standard Method for Determining Fire Resistance of Concrete and Masonry Construction Assemblies, ACI 216.1-97 / TMS 0216.1-97, American Concrete Institute.
- Buchanan A.H. (2002). "Structural design for fire safety". University of Canterbury, New Zealand.
- CEB (1991). Fire design of concrete structures. Bulletin 208, CEB, July 1991.
- CEMBUREAU/FIP (1979). Concrete for Fire Resistant Construction.
- EN1992-1-2: "Eurocode 2: Design of Concrete Structures - Part 1-2: General rules - Structural Fire Design", December 2004, 97 pp.
- NIST (1997). International Workshop on Fire Performance of High-Strength Concrete. Proceedings, Special Publication 919, Gaithersburg, MD, February 13-14.
- PCI (1985) Design Handbook: Precast and Prestressed Concrete, 3rd Edition.
- Phan L.T. (1996). "Fire Performance of High-Strength Concrete: A Report of the State-of-the-Art". NISTIR 5934, Building and Fire Research Laboratory, National Institute of Standards and Technology, December 1996.
- Van Acker A. (2003). "Shear resistance of prestressed hollow core floors exposed to fire". Structural Concrete, Vol. 4, No. 2, pp. 65-74.

A2 Fastenings*

A2.1 Introduction

With the increasing use of fasteners in civil engineering, the need for estimating their fire resistance is becoming more and more important. However, there are no unified test procedures at the moment and no commonly-accepted design rules for fastening systems exposed to fire, this being a clear demonstration of the difficulties of the problem. Moreover, the different failure modes exhibited by loaded fasteners in fire conditions have not been so far investigated in a systematic way.

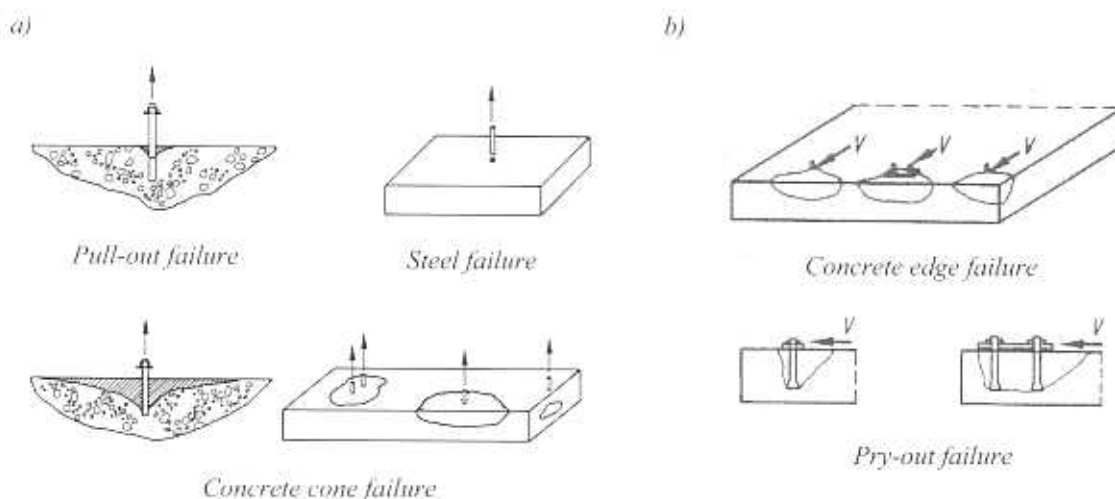


Fig. A2-1: Typical failure modes of fasteners; (a) under tensile load; and (b) under shear load (Eligehausen et al., 2006)

The typical failure modes observed during the tests on anchors loaded in tension or shear or in combined tension and shear in ordinary environmental conditions are schematically shown in Fig. A2-1 (Eligehausen et al., 2006). Under tensile load (Fig. A2-1a) fasteners usually fail after the formation of a concrete cone (concrete-cone failure mode), which means that concrete tensile strength is the controlling parameter, together with the (limited) toughness of the material. The ultimate anchor resistance depends on the embedment depth and on concrete fracture properties. The second typical failure mode is due to shank yielding, when the steel capacity is exhausted. This failure mode is typical of large embedment depths and of small-diameter anchors. A third possible failure mode ensues from the pull-out of the anchor, when the mechanical interlock or the chemical adhesion between the shank and the concrete fails, and in so doing the load transfer from the anchor to the concrete is no longer possible.

Under shear loading anchors can fail because of concrete edge-failures, pry-out failures and steel failures. Concrete edge-failure has some similarities with concrete-cone failure under tensile loading (see Fig. A2-1b). The ultimate failure load depends mainly on the distance from the edges and on concrete strength. In the case of anchors having a small embedment depth and a large distance from the nearest edge, either steel failure or pry-out failure occurs. In case of pry-out failure, anchors may exhibit a sufficient rotation capacity to produce a concrete cone failure 'behind' the point of load application.

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A2.2 Behaviour of fasteners under fire

In a number of fire tests, mainly performed by various manufacturers to test their specific products, the most frequently-observed failure mode was due to steel failure. However, in most of these tests concrete failure modes (concrete-cone failure and pull-out failure, in case of tension load, or concrete pry-out and edge failures in the case of shear load, see Fig. A2-1) were deliberately excluded, for instance by adopting relatively-large embedment depths. However, some recent experimental investigations (Reick, 2001) and numerical simulations based on 3D thermo-mechanical FE modelling (Ozbolt et al., 2005) have demonstrated that concrete-related failure modes can also take place. This is especially the case for (a) anchors with relatively-small embedment depths, (b) anchors installed close to an edge and (c) anchors and anchor groups made of high-quality steel (for instance stainless steel).

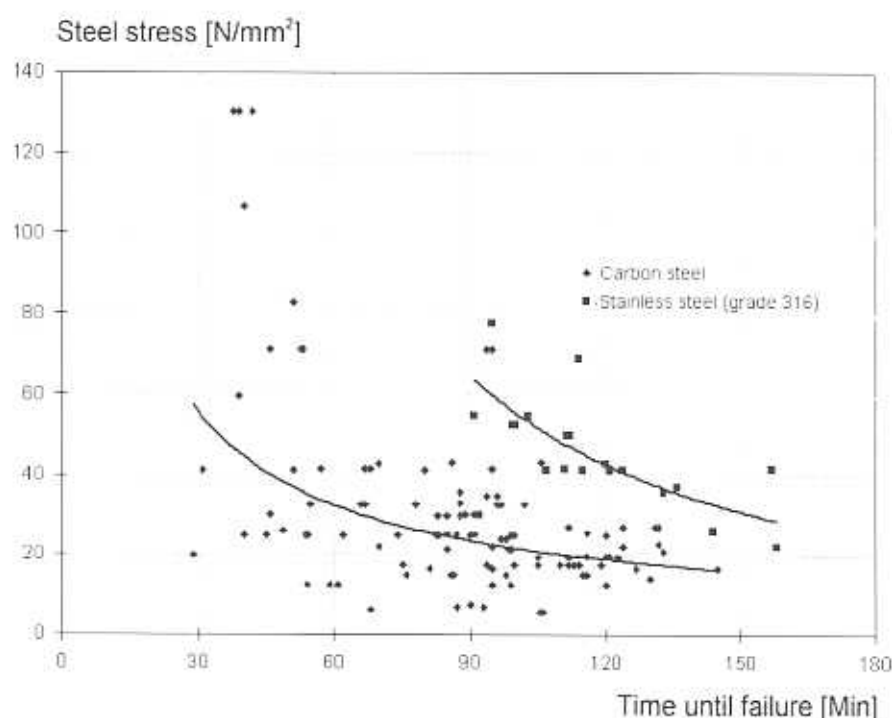


Fig. A2-2: Ultimate steel stress as a function of time to failure under a standard fire (Reick, 2001)

According to the tests where steel failure was observed, there are basically two parameters that play a major role in the steel failure of a fastener: (a) type of steel; and (b) diameter of the anchor. Figure A2-2 shows the results of more than 300 fire tests on different anchor types and different anchor sizes. The tests were performed in different laboratories, by using different fixture geometries, and are summarized by Reick (2001). The anchors were installed in both un-cracked and cracked concrete, and then a sustained tensile load was applied. The collapse was caused by rupture of the anchor bolt (i.e. anchor shank) or by stripping of the threads. In Fig. A2-2 each data point represents the ultimate steel stress, calculated from the applied load at the onset of failure. The curves shown in Figure A2-2 represent the average behaviour and demonstrate that the larger the fire duration, the smaller the ultimate steel stress, with stainless steel in a much better position than galvanized steel.

The very large scatter of the test results is mainly due to the different anchor sizes used in the tests. Therefore, in Fig. A2-3 the test results concerning the galvanized anchors mentioned in Fig. A2-2 are shown for different anchor sizes. Note that the ultimate steel stress increases with the anchor diameter. However, the influence of the anchor diameter is very pronounced for any fire duration below 60 minutes, since the larger the diameter, the smaller the mean temperature inside the bolt.

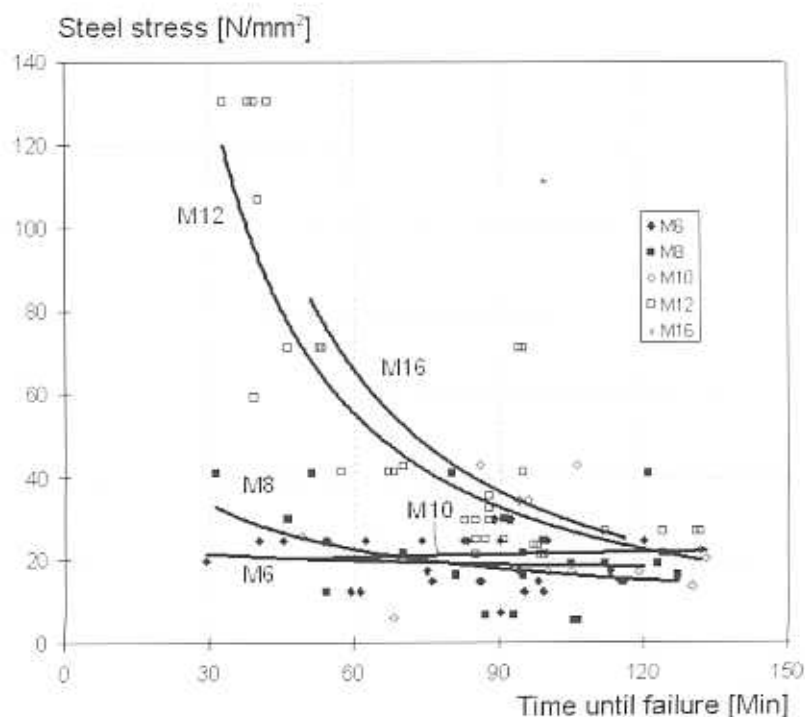


Fig. A2-3: Ultimate steel stress as a function of time to failure for anchors M6 to M16 made of galvanised carbon steel (Reick, 2001)

The scatter of the test results turns out to be rather large for any given diameter, particularly for small values of the fire duration, the reason being that some anchors were installed in cracked concrete, while others were installed in solid concrete. During the first 30 minutes of any fire test, water evaporates from the concrete, and the evaporation is definitely larger along the pre-existing cracks. However, evaporating water temporarily cools the fastener. Especially when a fastener is installed in a relatively small concrete mass, extensive water evaporation can be observed during the tests. As a consequence, the steel temperature decreases to about 100 °C. Water evaporation diminishes after ≈ 60 minutes of the fire duration and so effects the behaviour of the fastener.

To minimize the scatter of the results obtained in different laboratories, the test procedures for anchors in fire conditions should be standardised. A proposal is contained in the current CEN Technical Document (European Committee for Standardisation, 2006). On the basis of the results shown in Fig. A2-2, the characteristic steel strengths for fasteners under fire were defined and the results are summarised in CEN/TS - 2006 (compare Tables A2-1 and A2-2).

Table A2-1: Characteristic tension strength for galvanised carbon-steel fasteners exposed to standard fire (CEN/TS, (2006))

Diameter of anchor bolt or thread [mm]	Anchorage depth [mm]	Characteristic tension strength of unprotected galvanised carbon-steel fasteners (fire resistance class R) : $\sigma_{Rk,t,fire}$ [N/mm²]			
		30 min (R 15 to R30)	60 min (R45 and R60)	90 min (R90)	120 min ($\leq$ R120)
6	≥ 30	10	9	7	5
8	≥ 30	10	9	7	5
10	≥ 40	15	13	10	8
≥ 12	≥ 50	20	15	13	10

Table A2-2: Characteristic tension strength for stainless-steel fasteners (A4, grade 316) exposed to standard fire [CEN/TS (2006)]

Diameter of anchor bolt or thread [mm]	Anchorage depth [mm]	Characteristic tension strength of unprotected stainless-steel fasteners (fire resistance class R) : $\sigma_{Rk,s,fire}$ [N/mm ²]			
		30 min (R 15 to R30)	60 min (R45 and R60)	90 min (R90)	120 min (≤ R120)
6	≥ 30	10	9	7	5
8	≥ 30	20	16	12	10
10	≥ 40	25	20	16	14
≥ 12	≥ 50	30	25	20	16

A limited number of test results indicate that under fire exposure the shear and tension strength of an anchor are similar. Therefore in CEN/TS (2006) it is recommended that the values given in Tables A2-1 and A2-2 can also be used for the characteristic shear resistance of fasteners exposed to standard fire.

The pull-out of the fasteners (Fig. A2-1a) is mainly caused by the splitting cracks in concrete that reduce the friction between the anchor and the concrete. Since there are very few experimental data on this failure mode, the proposed design load according to CEN/TS is based on theoretical considerations. Because of the thermal strains in the concrete, compressive stresses are generated in the surface layer (approximately 35 mm deep) of a concrete member. Since these compressive stresses cause the closure of the splitting cracks, heating has initially a positive influence on the pull-out capacity of the anchors. However, when severe sagging in a concrete member subjected to bending – like a beam or a slab – occurs because of fire, a fastener may exhibit a pull-out failure, shortly before the collapse of the member, due to rapid propagation of bending cracks. However, the critical moment for the pull-out failure would probably occur after the fire, since concrete damage increases during the cooling of the concrete member. With regard to this point, recent finite element studies (Ožbolt et al., 2005) demonstrate that - because of irreversible thermal strains in the concrete closest to the anchor (load-induced thermal strains) - additional cracks are generated in the concrete, with a reduction of the friction at the interface with the anchor. In the CEN/TS document, the reduction of the pull-out capacity after 90 minutes of standard fire (ISO 834) is evaluated as 25 % of the pull-out resistance at room temperature.

As previously mentioned, the typical concrete-related failure mode of a fastener loaded in tension is controlled by the formation of a concrete cone (concrete-cone failure). According to the available experimental data (Reick, 2001) and to recently-performed numerical simulations (Ožbolt et al., 2005), the reduction of the ultimate resistance in tension depends mainly on the embedment depth (anchors with small embedment depths exhibit reductions up to 60 % of the ultimate resistance at room temperature). However, for anchors with large embedment depths, small or even no reduction at all has been observed (Fig. A2-4). For relatively-small embedment depths, the whole shank of the anchor is surrounded by very hot concrete, that is severely damaged (concrete tensile and compressive strengths, as well as Young's modulus are significantly reduced). For anchors with large embedment depths, the head of the anchor (in undercut fasteners) and a sizable part of the shank (in expansion fasteners) are relatively far from the heated surface, in a zone of lower temperatures, where the concrete is less damaged and its properties are still good. Such conditions contribute to the relatively low reduction of the concrete-cone resistance. In the CEN/TS document, for anchors with embedment depths up to $h_{ef} = 200$ mm, the reduction of the characteristic resistance ($N_{Rk,c}$) after 90 minutes of heating is formulated as a linear function of the embedment depth – $N_{Rk,c}(90) = (h_{ef}/200) \cdot N_{Rk,c}$. For anchors with embedment depths larger than 200 mm, no reduction is proposed (Fig A2-4.)

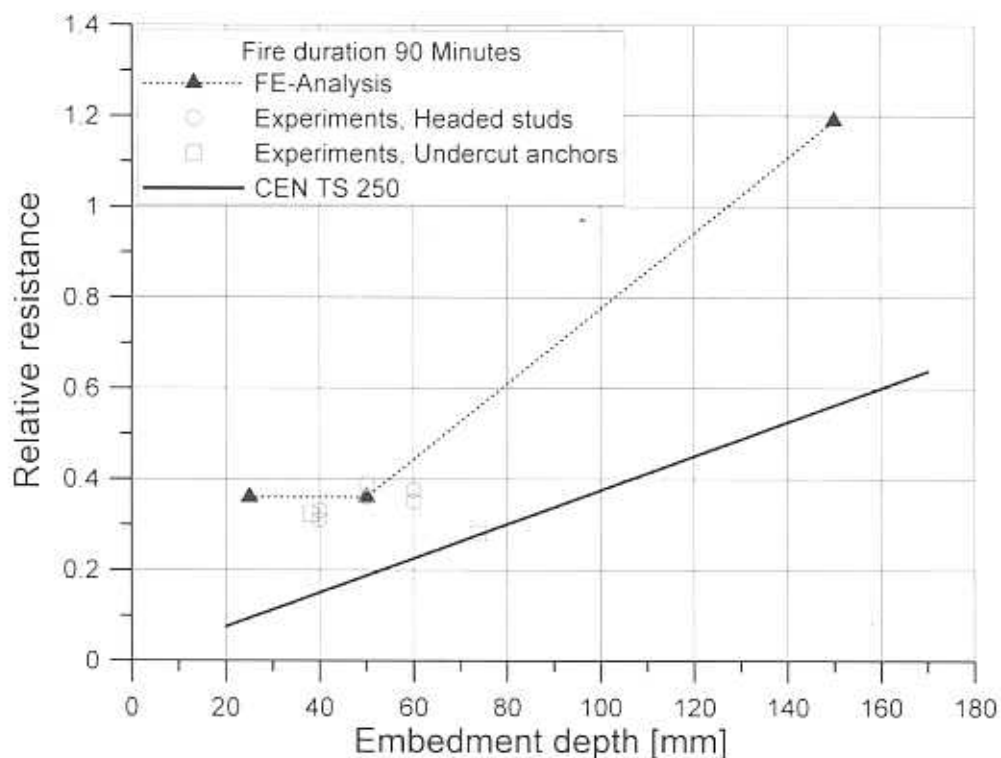


Fig. A2-4: Relative concrete-cone resistance of anchors, as a function of the embedment depth (Ozbolt et al., 2005)

Concrete cone failure is particularly critical in the case of group anchors, especially when the anchor spacing is small. In these cases the steel capacity is k -times the capacity of a single anchor (k = number of the anchors in a group), whereas the concrete cone capacity depends on anchor spacing and varies between 1 - and k -times the capacity of a single anchor (Eligehausen et al., 2006). Consequently, the possibility for anchors with large embedment depths to fail according to the concrete-cone mode cannot be ruled out. Furthermore, any explosive spalling in the concrete layers closest to the heated surface would additionally weaken concrete-cone resistance, for either single or group anchors. Unfortunately, there are no experimental results and theoretical predictions on this issue.

All the presented and commented so far are about anchor behaviour at high temperature and/or under the standard fire. However, since concrete behaviour tends to worsen during and after cooling, anchors should be designed considering also the post-fire situation, for at least two reasons. Firstly, new anchors may be installed in thermally-damaged concrete (whose deterioration is not always evident), and secondly the anchors installed before the fire may be unsafe after the fire. In both cases, the most typical failure mode (under pure tension and far from the edges) is that related to concrete-cone formation, since the fastener is either undamaged (first case) or recovers most of its initial mechanical properties after the cooling process. With reference to this context, small- and medium-diameter fasteners – that necessarily have a rather small instalment depth – are likely to fail because of the formation of a concrete cone after a fire, even if in ordinary conditions (before the fire) they have been designed to fail because to shank yielding. Only scanty attention has been devoted so far to the post-fire situation of an anchor, but some recent results may be cited (Bamonte et al., 2007), with reference to undercut fasteners installed in a concrete block, slowly heated along one of the faces ($\Delta T/\Delta t = 1\text{--}2^\circ\text{C}/\text{minute}$, see Fig. A2-5a). In Fig. A2-5b the capacity in tension of an undercut medium-diameter fastener (shank diameter $\varnothing = 10$ mm, nominal suggested instalment depth $h_{ef,nom} = 10\varnothing = 100$ mm, actual instalment depth $h_{ef,act} = 0.8 h_{ef,nom} = 80$ mm)

is plotted against the temperature reached underneath the head, at the reference depth $h^* = 8\varnothing$ ($= 80$ mm), for 3 concrete grades.

Depending on the grade, at a temperature between 150°C and 220°C the failure mode shifts from shank yielding to concrete-cone formation, and for higher temperatures the capacity of the fastener is greatly affected by the temperature. Of course, under a real fire characterized by much higher heating rates ($\Delta T/\Delta t = 100^\circ\text{C}/\text{minute}$), the capacity is even more affected by the temperature. For the same fastener type shown in Fig. A2-5a and for different instalment depths, a rather qualitative diagram of the reduction factor is plotted in Fig. A2-5c. This factor takes care of the much larger thermal damage that occurs in a rapidly-heated concrete mass, compared to a slowly-heated mass, for the same temperature reached at a prefixed depth.

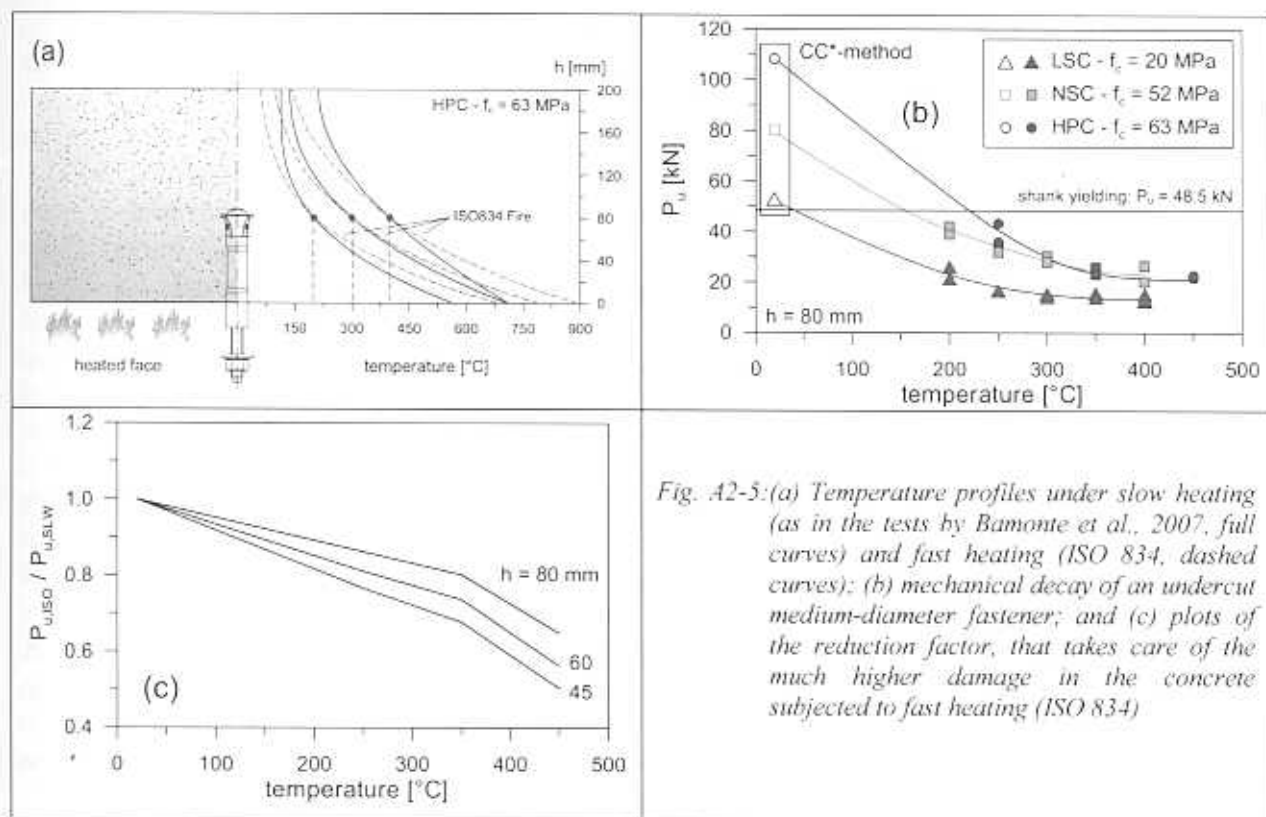


Fig. A2-5: (a) Temperature profiles under slow heating (as in the tests by Bamonte et al., 2007, full curves) and fast heating (ISO 834, dashed curves); (b) mechanical decay of an undercut medium-diameter fastener; and (c) plots of the reduction factor, that takes care of the much higher damage in the concrete subjected to fast heating (ISO 834)

Summing up, it can be concluded that further theoretical and experimental investigations of fasteners in fire conditions are needed. Furthermore, the anchor behaviour discussed so far is valid only if the exposure to the fire is limited to one face (i.e. one side). However, if the exposure involves two or more sides, and the distance of the anchor from an edge is relatively small, there is an additional fire influence. In order to investigate these cases and to avoid – at least partly – very expensive experiments, 3D numerical simulations of concrete members are needed. In these numerical simulations, the so-called transport phenomena (mostly related to moisture and vapour migration inside the concrete) should be modelled in order to predict the possible occurrence of explosive spalling. There is an obvious need for developing realistic hydro-thermo-mechanical models, with full coupling of these three different domains. To verify these models, theoretical investigations must be supported by specific tests, whose results will be instrumental in improving the current design codes for fasteners and in formulating new, more rational and possibly simpler design rules.

References

- Bamonte P.F., Gambarova P.G., Bruni M., Rossini L.: Ultimate Capacity of Undercut Fasteners Installed in Thermally-Damaged High-Performance Concrete, Proc. 6th Int. Conf. on Fracture Mechanics of Concrete Structures – FraMCoS-6, Catania (Italy), June 18-21, 2007.
- CEN/TS 250: Design of Fastenings for Use in Concrete. European Committee for Standardization, 2006.
- Eligehausen, R., Mällée, R. and Silva J.F.: Anchorage in concrete construction. Ernst & Sohn, Berlin, 2006.
- Ožbolt, J., Kožar, I., Eligehausen, R. and Periškić, G.: Three-dimensional FE analysis of headed stud anchors exposed to fire. Computers & Concrete, 4(2), 249-266, 2005.
- Reick, M.: Brandverhalten von Befestigungen mit großem Randabstand in Beton bei zentrischer Zugbeanspruchung. Mitteilungen des Institut für Werkstoffe im Bauwesen (Fire Behaviour of Axially-Loaded Fasteners Installed in Concrete Blocks far from the Sides), Technical Report 2001/4, IWB, Stuttgart, Germany, 2001.

A3 Integrity of compartmentation *

A3.1 Introduction

The fire resistance of loadbearing and non-loadbearing components that form compartment walls and floors are typically assessed in isolation, using standard fire test procedures according to EN1363 Part 1 or the relevant national standards. It is then assumed that the construction will provide the appropriate level of resistance in an actual fire in a real building. However, the mode of failure may be different to that experienced in the isolated tests. In the case of loadbearing walls and infill masonry panels, horizontal thermal expansion of the surrounding structure could cause instability of the wall, leading to premature failure. For non-loadbearing walls, the vertical displacement of the structure during a fire is not directly considered when assessing its performance and may lead to premature failure when used in actual buildings.

A3.2 Regulatory requirements and standard fire tests

Compartmentation has traditionally been assumed based on the concept of fire resistance and measured in relation to the resistance to collapse, resistance to fire penetration, and resistance to the transfer of excessive heat.

The purpose of sub-dividing spaces into separate fire compartments is twofold. Firstly to prevent rapid fire spread which could trap occupants of the building and secondly to restrict the overall size of the fire. According to the UK guidance there should be continuity at the junctions of the fire resisting elements enclosing a compartment. Typically this would be the junction between a wall (either loadbearing or non-loadbearing) and a floor. The general method for elements of structure (including compartment floors and walls) is to rely on prescribed values in the regulations. The values relate to a minimum period for which the element must survive in the standard fire test measured against the relevant performance criteria of stability, integrity and insulation. Given that the standard test relates to single elements it is difficult to see how such a reliance can achieve the requirement related to the provision of continuity at the junction between two elements.

The principal area of concern is related to separating elements required to satisfy the criteria of integrity and insulation in addition to loadbearing capacity where appropriate. It is therefore necessary to investigate the methods used to assess performance against the defined criteria for both floor and wall elements.

A3.3 Loadbearing capacity

A3.3.1 Floors

For horizontal members failure in a standard test is assumed to have occurred when the deflection reaches a value of $L/20$ where L is the clear span of the specimen or where the rate of deflection (mm/min) exceeds a value of $L^2/9000d$ where d is the distance from the top of the section to the bottom of the design tension zone (mm). The rate of deflection criteria only applies once the deflection has reached a value of $L/30$.

The origin of the deflection limits are unclear but they, at least in part, are based on the limitations of test furnaces and the requirement to avoid damage to the furnace. This is not a logical basis on which to assess loadbearing capacity. The full-scale tests carried out at Cardington have demonstrated that loadbearing capacity can be maintained when deflections

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much greater than those used to measure "failure" in a standard test have been mobilised. For concrete floor elements failure is generally a function of the insulation capacity rather than loadbearing capacity.

A3.3.2 Walls

For vertical loadbearing elements failure of the test specimen is deemed to occur when the specimen can no longer support the applied load. There is no clear definition of failure in relation to the standard test. Laboratories are only required to provide for maximum deformations of 120mm and values over and above this limit would require the test to be terminated. The state of failure is characterised by a rapid increase in the rate of deformation tending towards infinity. It is therefore recommended that laboratories monitor the rate of deformation to predict the onset of failure and support the test load.

A3.4 Integrity

A3.4.1 Floors and walls

The basic criteria for integrity failure of floor and wall elements is the same. An integrity failure is deemed to occur when either collapse, sustained flaming or impermeability have occurred. Impermeability, that is the presence of gaps and fissures, should be assessed using either a cotton pad or gap gauges. After the first 5 minutes of heating all gaps are subject to periodic evaluation using a cotton pad 100mm square by 20mm thick mounted in a wire holder which is held against the surface of the specimen. If the pad fails to ignite or glow the procedure is repeated at intervals determined by the condition of the element. For vertical elements where the gaps appear below the neutral pressure axis position gap gauges will be used to evaluate the integrity of the specimen. If the 25mm gauge can penetrate the gap to its full length (25mm + thickness of the specimen as a minimum value) or the 6mm gauge can be moved in any one opening for a distance of 150mm then integrity failure is recorded. The cotton pad is no longer used when the temperature of the unexposed face in the vicinity of the gap exceeds 300°C. At this point the gap gauges are used.

Again the origins of the measures used to determine performance are unclear.

A3.5 Insulation

A3.5.1 Floors and walls

The basic criteria for insulation failure of floors and wall elements is the same. Insulation failure is deemed to occur when either the mean unexposed face temperature increases by more than 140°C above its initial value or the temperature at any position on the unexposed face exceeds 180°C above its initial value.

The effect of these localised temperature rises on the unexposed face is unclear. For timber products ignition by a pilot flame can occur between 270°C and 290°C whilst spontaneous ignition (required if there is no integrity failure) occurs between 330°C and 500°C depending on species. These figures suggest that the temperatures used to define insulation failure may be too low particularly for structural elements passing through compartment walls where storage of combustible materials on the unexposed side is unlikely.

The UK test standard states specifically that the standard test method is not applicable to assemblies of elements such as wall and floor combinations. There is some limited guidance to suggest that the test method may be used as the basis for the evaluation of three-dimensional constructions with each element loaded according to the practical application and each element monitored with respect to compliance with the relevant criteria.

A3.6 Results from standard tests

A study has been undertaken based on a series of fire resistance tests carried out by the UK Fire Research Station [Davey, Ashton (1953)]. There are issues to be considered about the allowable deflection to be accommodated in relation to fire resisting construction on the fire floor itself, the floor below and the floor above. Compartment walls are often built under existing lines of compartmentation. For residential buildings where the requirements for compartmentation are particularly stringent the building layout is generally regular with compartment walls running continuously from floor to floor. In such cases the anticipated deflection is likely to be quite small where structural elements span from compartment wall to compartment wall. However, there is no guarantee that compartment walls will always be located in such an advantageous arrangement and there is nothing in the regulations to prevent a compartment wall being constructed immediately underneath or immediately above the mid-span of the supporting element. A useful starting point would therefore be a review of the likely range of deflections to be accommodated for a number of different forms of construction both in terms of standard fire tests and measured results from natural fire tests.

Figure A3.1 shows the spread of results for the maximum deflection of tested reinforced concrete floors in the centre of the span. In general the fire resistance of concrete floors in the absence of spalling is governed by the insulation requirement. Therefore, excluding those values where overall collapse took place and limiting the results to those elements that either survived for the entire duration of the test or failed by an insulation failure the displacement at the centre of the slab is shown in Figure A3.2.

There is an assumption that the current method of meeting the regulatory requirement provides acceptable results. In general the tests referred to above were carried out on specimens spanning 4m. Limiting the deflection to a value of $L/20$ should exclude results greater than 200mm for a 4m span. The values quoted are for the maximum deflection recorded and do not provide any information on the time-deflection history throughout the test.

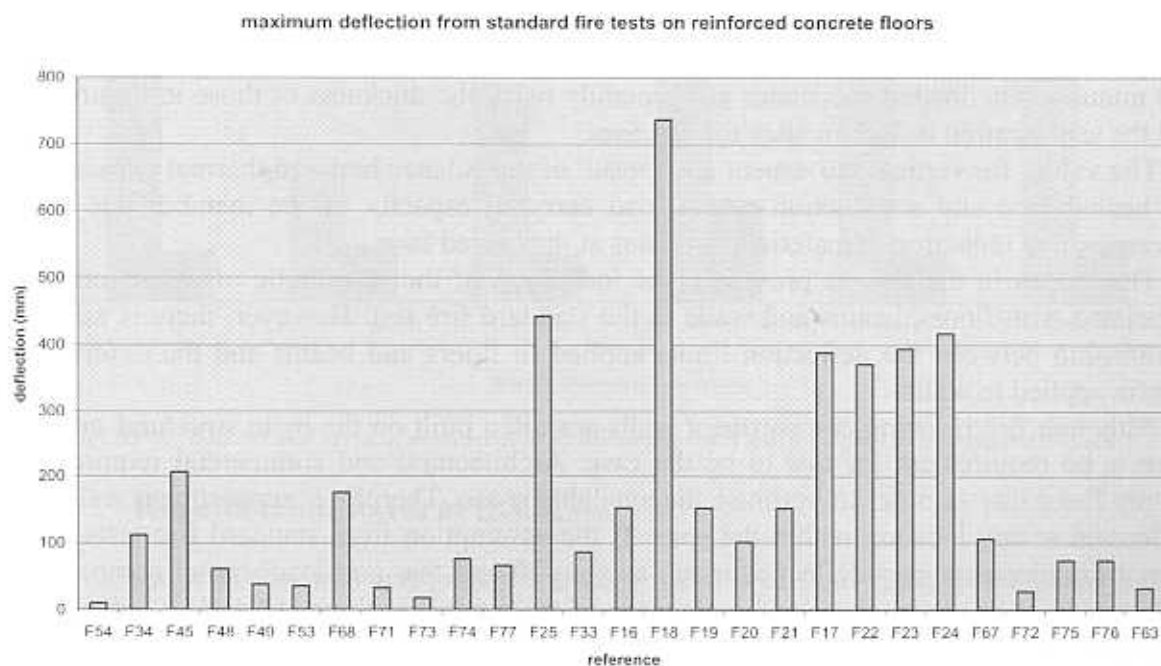


Fig. A3-1: Maximum mid-span deflection of reinforced concrete floors in standard fire tests

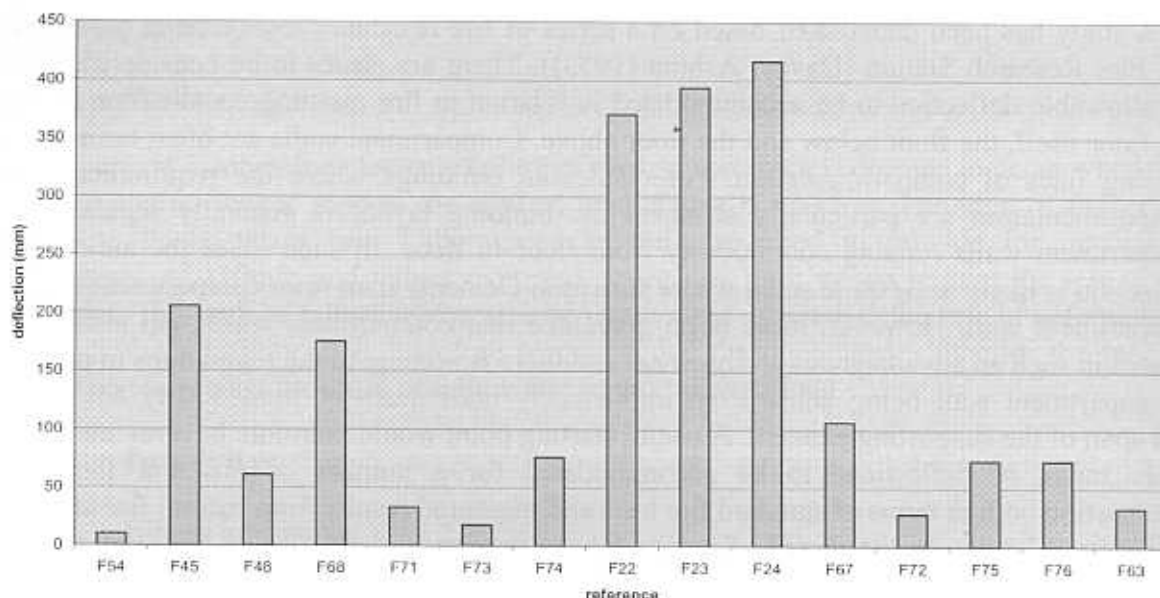


Fig. A3-2: Maximum deflection of reinforced concrete slabs excluding loadbearing and integrity failure

The “allowable” deflection of floor slabs and beams should be seen alongside the requirements for both loadbearing and non-loadbearing walls and partitions. For loadbearing walls there is a requirement to measure vertical deformation and lateral deflection. For non-loadbearing wall elements (partitions) there is a requirement to measure the lateral movement and record the maximum value. The nature of the deformation of walls in standard tests is very much a function of the test set-up. For non-loadbearing walls they are restrained in a frame and therefore can only move laterally due to thermal bowing. For loadbearing walls they are restrained along the free edges but free to move in the direction of load.

The results for non-loadbearing and loadbearing brick walls in standard tests are shown in Figure A3.3 and Figure A3.4. For Figure A3.3 the results generally relate to a time period of 120 minutes. The loaded specimens are generally twice the thickness of those in Figure A3.3 and the test duration is 360 minutes for all cases.

The values for vertical movement are a result of the balance between thermal expansion of the heated face and a reduction in the load carrying capacity of the member due to the corresponding reduction in material properties at the heated face.

The values in the figures provide some indication of the magnitude of the deformation associated with floors, beams and walls in the standard fire test. However, there is no direct relationship between the deflection limits applied to floors and beams and the deformation criteria applied to walls.

Although fire resisting compartment walls are often built on the main structural gridlines there is no requirement for this to be the case. Architectural and commercial requirements require flexibility in order to optimise the available space. Therefore compartment walls may be located at any location within the span. If the assumption from standard fire tests is that supporting elements may deflect as much as $\text{span}/20$ and that non-loadbearing compartment walls can be located anywhere within the span of the beam then there is clearly a potential for premature failure of compartmentation. This potential for failure applies to existing prescriptive methods (i.e a reliance on the results from standard fire tests) of providing the necessary fire resistance to ensure the integrity of compartmentation.

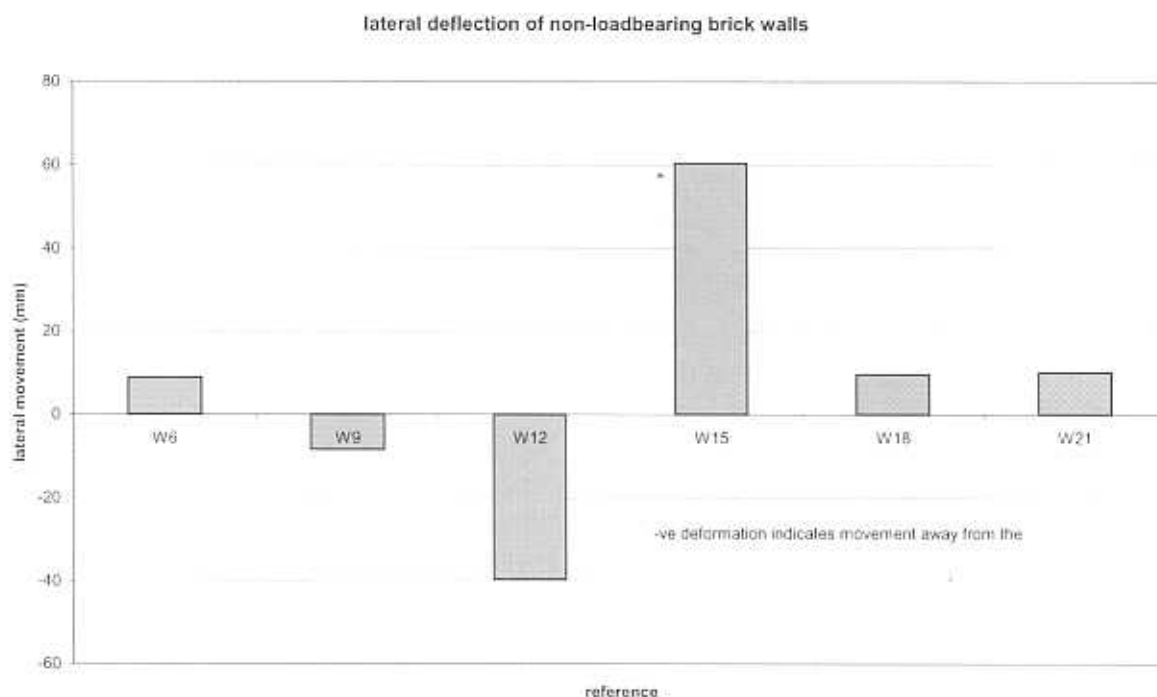


Fig. A3-3: Lateral movement of non-loadbearing brick walls subject to a standard fire curve

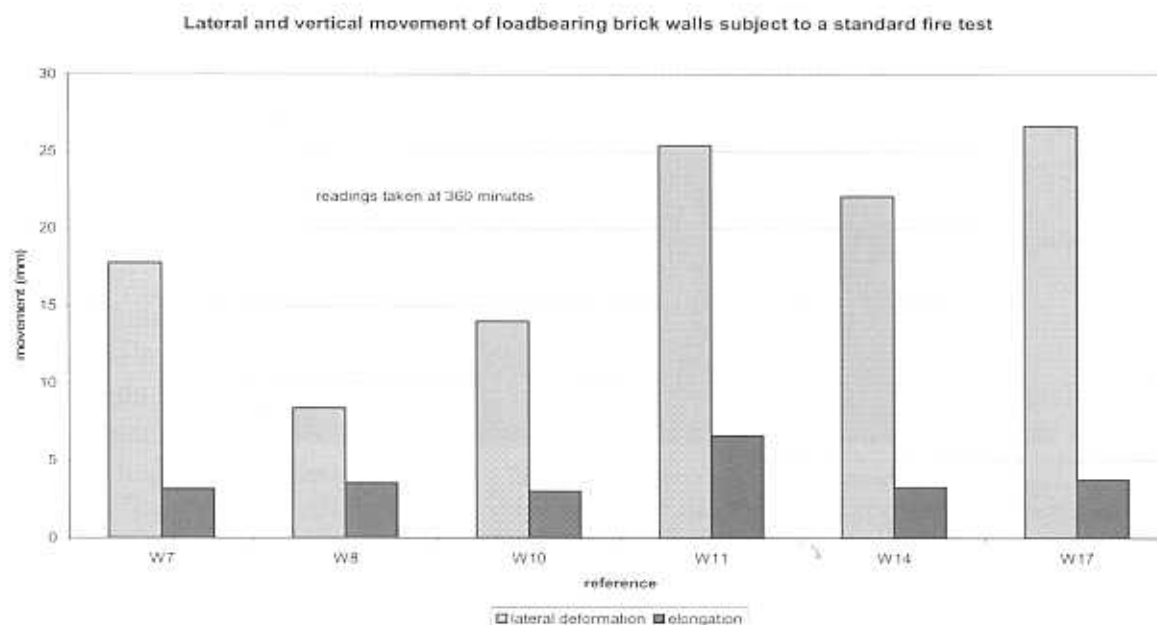


Fig. A3-4: Lateral and vertical movement of loadbearing brick walls subject to a standard fire test

A3.7 Results from natural fire tests

The effect of the thermal and mechanical deformations of floor slabs on the performance of compartment walls is an issue that has been highlighted through the programme of full-scale fire tests carried out at BRE's Cardington test facility. The design guidance [Newman et al (2000), ECSC (2002)] produced as a result of the tests concluded that, wherever possible, compartment walls should be located beneath and in line with the main building gridlines. The effects of deformation of the floor slab on compartment walls built up to the underside of the supporting beams (Figure A3.5) and offset from the main gridlines (Figure A3.6) is shown below.

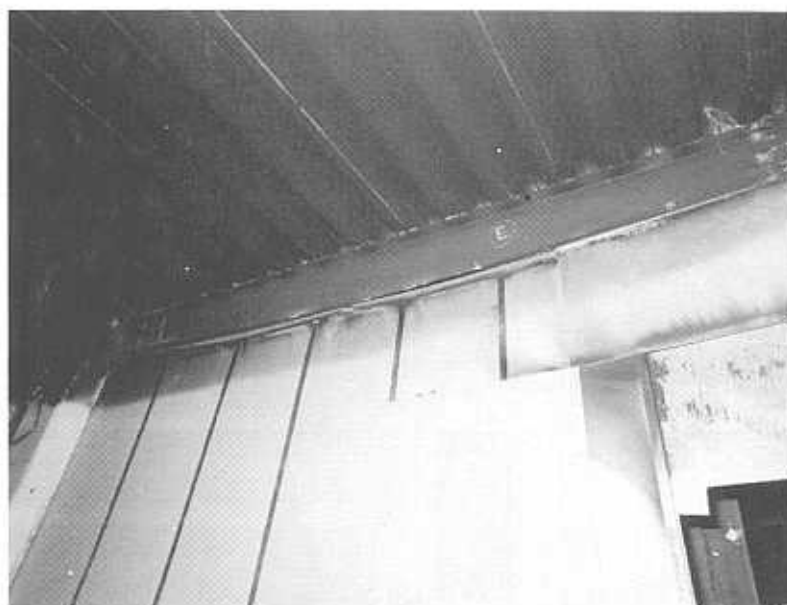


Fig. A3-5: Maintenance of integrity of compartment walls – BRE corner fire test



Fig. A3-6: Integrity failure of compartment wall – BRE large compartment test

For compartment walls made from lightweight plasterboard systems, the manufacturer can supply a range of standard details to accommodate movement from the floor above. A range of such standard details may be found in Appendix C. In general, the deflection heads are there to accommodate movement at ambient temperature and have not been designed for the large levels of vertical deflection typically occurring during fires.

The limited guidance available [Newman et al (2000), ECSC (2002)] on maintaining the integrity of compartmentation during a fire makes mention of deformable blanket and sliding joints without providing any specific details of how to design or install such products whilst maintaining the required insulation and integrity characteristics of the wall.

The UK code of practice for the use of masonry [BS 5628 (2001)] mentions that consideration should be given to the interaction of the whole structure of which the masonry forms a part. The connections of other elements with the walls should be sufficient to transmit all vertical and horizontal loads. For internal walls and partitions not designed for imposed loading, the code provides guidance on the ratio of length to thickness and height to thickness dependent on the degree of restraint present.

If the wall is restrained at both ends but not at the top (a common scenario for non-loadbearing walls), then $t > L/40$ and $t > H/15$ with no restriction on the value of L . Where restraint is present at both the ends and the top, then the same restriction on length to thickness applies and there is a restriction on the height to thickness ratio of 30 with no restriction on the value of L . If the wall is restrained at the top but not at the edges then the height to thickness ratio should be greater than 30.

Where a wall is supported by a structural member it is suggested that a separation joint may be included at the base of the wall or bed joint reinforcement should be included in the lower part of the wall.

Where a partition is located below a structural member and is not designed to carry any vertical load from the structure above it should be separated by a gap or by a layer of resilient material to accommodate deformation. Mention is also made of the need to consider lateral restraint and fire integrity in such situations.

For masonry walls whether loadbearing or not one of the most important aspects of behaviour in fire is the impact of thermal bowing. Some guidance is available in BRE Information Paper 21/88 [Cooke (1988)]. This is the basis of the calculation of the thermal bowing component of the displacement criteria adopted by Bailey (2003) who applied a calibration factor based on the results from the full-scale fire tests to apply the equation to composite floor slabs. The original equations apply to metallic elements.

Concrete, brickwork and blockwork have a lower thermal conductivity than steel and the temperature distribution is therefore highly non-linear with a large thermal gradient across the section. Cooke (1988) presented data for free-standing (cantilever) walls subject to a standard fire exposure. Two thicknesses of wall were tested (225mm and 337mm) with corresponding slenderness (height/thickness) of approximately 13 and 9. The horizontal deflections at the top of the wall were 70mm and 55mm after just 30 minutes fire exposure. Given that the thickness of the walls was well within the limits set by the code of practice BS 5628 (2001) this provides some cause for concern. Walls built to the limits of the code would deflect considerably more than the test values.

A number of design factors can be used to alleviate the effects of thermal bowing. These include:

- The choice of a material with a low coefficient of thermal expansion
- Increasing the thickness of the element
- Providing restraint at the top wherever possible (even for non-loadbearing walls) as the mid-span deflection of simply supported members is a quarter of that at the free end
- Providing edge support

Cooke's paper also pointed to the importance of the thermal exposure in determining the extent of thermal bowing highlighting the need to consider time/temperature regimes other than the standard curve.

References

- Davey, Ashton (1953): Davey N and Ashton L A, National building Studies Research Paper No. 12, Investigations on Building Fires, Part V, Fire Tests on Structural Elements, HMSO, London, 1953
- Newman et al (2000): Newman G M, Robinson J T and Bailey C G, Fire Safe design: A New Approach to Multi-Storey Steel-Framed Buildings, SCI Publication P288, The Steel Construction Institute, Ascot, 2000
- ECSC (2002): Design recommendations for composite steel framed buildings in fire, ECSC Research Project 7210PA, PB, PC, PD112, December 2002

BS 5628 (2001): BS 5628-3:2001, Code of practice for use of masonry – Part 3: Materials and components, design and workmanship, British Standards Institution, London

Cooke (1988): Cooke G M E, Thermal bowing in fire and how it affects building design, BRE Information Paper 21/88, Garston, December 1988

Bailey (2003): Bailey C G, New fire design method for steel frames with composite floor slabs, FBE Report 5, Foundation for the Built Environment, BRE Bookshop, January 2003

A4 Complete results of the parametric study on continuous beams and frames discussed in chapter 4*

MATERIALS: Concrete C30/37 Reinforcing steel B500B		CROSS SECTIONS		
STRUCTURAL MODELS		RECTANGULAR SECTION	ONE WAY SLAB	RECTANGULAR SECTION + SLAB
TRANSLATIONAL STIFFNESS k				
LOADS Rectangular section Rectangular section + slab One way slab DL = 36 kN/m DL = 36 kN/m DL = 7.25 kN/m ² LL = 12 kN/m LL = 12 kN/m LL = 4 kN/m ²	0	B = 35 cm H = 50 cm SECTION A-A 4φ20 + 2φ20	B = 125 cm H = 25 cm SECTION A-A 6φ12	B = 100 cm H = 40 cm h _r = 10 cm b _w = 30 cm SECTION A-A 5φ20 + 2φ20
	EA/3L	SECTION B-B 2φ20 + 2φ20	SECTION B-B 6φ12	SECTION B-B 2φ20 + 3φ20
	EA/L	SECTION A-A 7φ20 + 3φ20	SECTION A-A 8φ16	SECTION A-A 7φ20 + 3φ20
LOADS Rectangular section Rectangular section + slab One way slab DL = 54 kN/m DL = 54 kN/m DL = 9.75 kN/m ² LL = 18 kN/m LL = 18 kN/m LL = 4 kN/m ²	∞	B = 55 cm H = 75 cm SECTION A-A 7φ20 + 3φ20	SECTION A-A 6φ16	SECTION A-A 7φ20 + 3φ20
	EA/3L	SECTION B-B 2φ20 + 5φ20	SECTION B-B 6φ16	SECTION B-B 2φ20 + 5φ20
	EA/L	SECTION A-A 2φ20 + 5φ20	SECTION A-A 6φ16	SECTION A-A 2φ20 + 5φ20

Fig. A4-1: Parametric study of the beams

* by Paolo Riva

MATERIALS:
Concrete C30/37
Reinforcing steel B500B

STRUCTURAL MODELS

CROSS SECTIONS		
RECTANGULAR SECTION	ONE WAY SLAB	RECTANGULAR SECTION + SLAB
 LOADS: Rectangular section: DL = 36 kN/m, LL = 12 kN/m Rectangular section + slab: DL = 36 kN/m, LL = 12 kN/m One way slab: DL = 7.25 kN/m², LL = 4 kN/m² RECTANGULAR SECTION: B = 35 cm H = 50 cm SECTION A-A: 3φ20 + 2φ20 SECTION B-B: 2φ20 + 3φ20	 LOADS: Rectangular section: DL = 36 kN/m, LL = 12 kN/m Rectangular section + slab: DL = 36 kN/m, LL = 12 kN/m One way slab: DL = 7.25 kN/m², LL = 4 kN/m² ONE WAY SLAB: B = 125 cm H = 25 cm SECTION A-A: 6φ12 SECTION B-B: 6φ12	 LOADS: Rectangular section: DL = 36 kN/m, LL = 12 kN/m Rectangular section + slab: DL = 36 kN/m, LL = 12 kN/m One way slab: DL = 7.25 kN/m², LL = 4 kN/m² RECTANGULAR SECTION + SLAB: B = 100 cm, h_r = 10 cm H = 40 cm, b_s = 30 cm SECTION A-A: 4φ20 + 2φ20 SECTION B-B: 2φ20 + 3φ20

Fig. A4-2: Parametric study of the frames

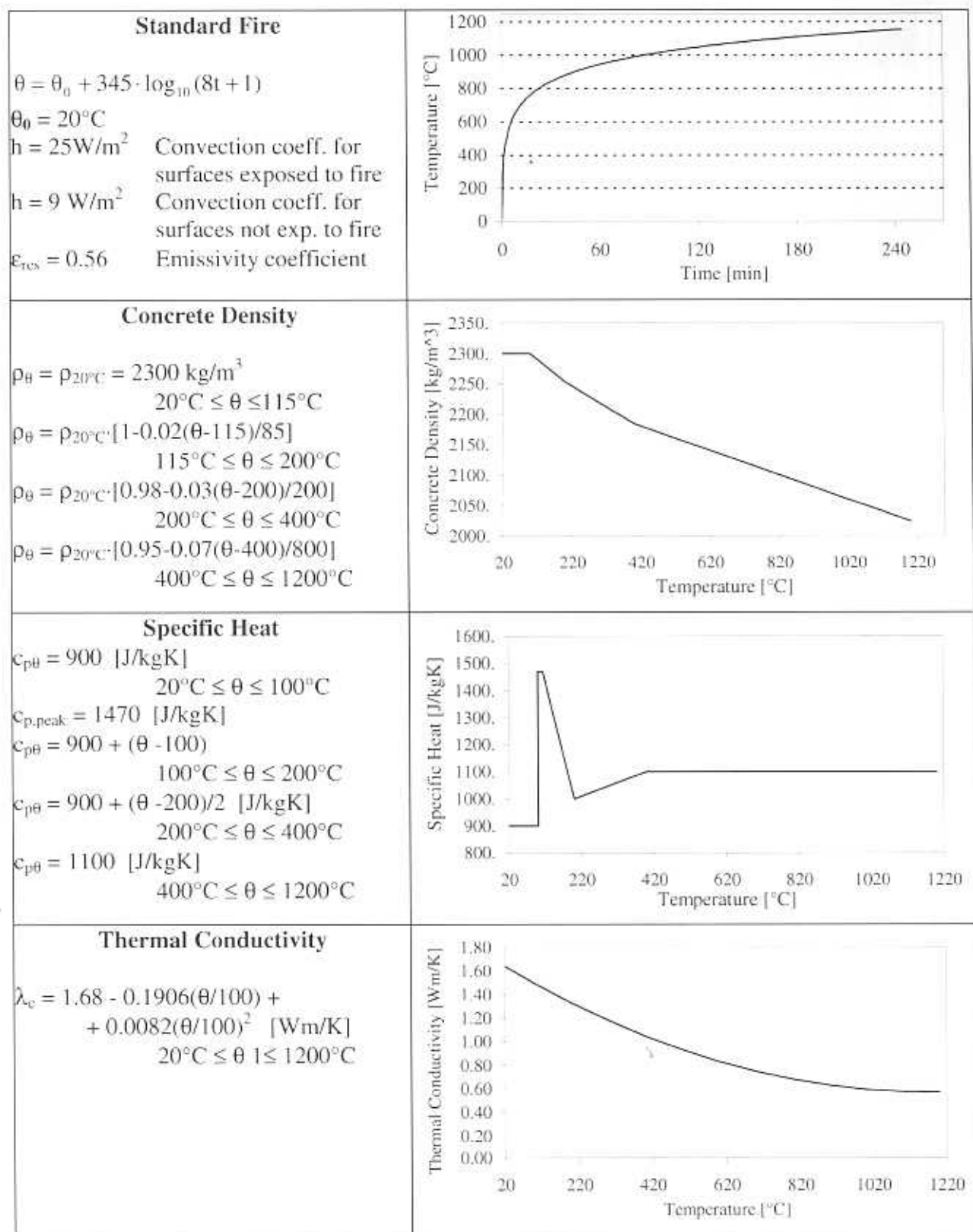


Fig. A4-3: ISO 834 fire and concrete thermal properties (EC2 2005)

6m BEAMS	RECTANGULAR SECTION ADIABATIC 	ONE WAY SLAB AIR 20°C 	T-SECTION AIR 20°C
	RECTANGULAR SECTION ADIABATIC 	ONE WAY SLAB AIR 20°C 	T-SECTION AIR 20°C
	COLUMNS FIRE ON ONE SIDE Air T=20°C ADIABATIC 	COLUMNS FIRE ON THREE SIDES ADIABATIC 	

Fig. A4-4: Thermal analysis boundary conditions.

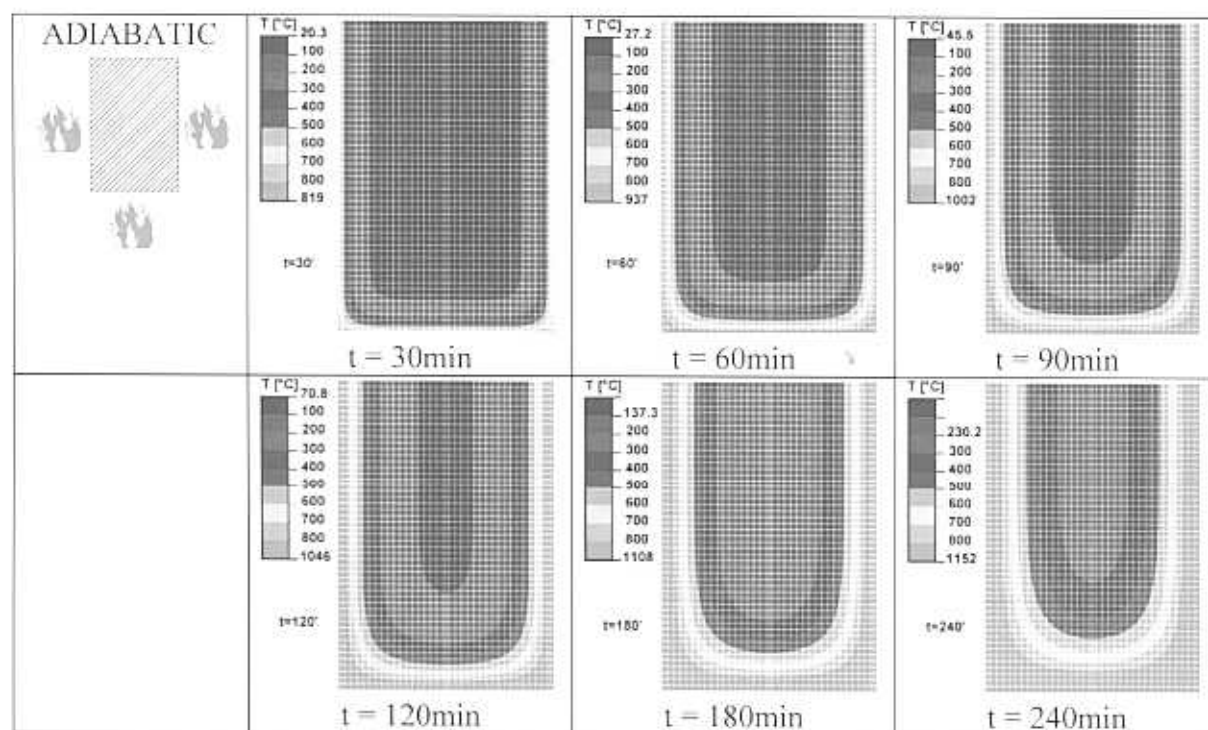


Fig. A4-5: Thermal analysis results of the 6m span rectangular beam [$^{\circ}\text{C}$]

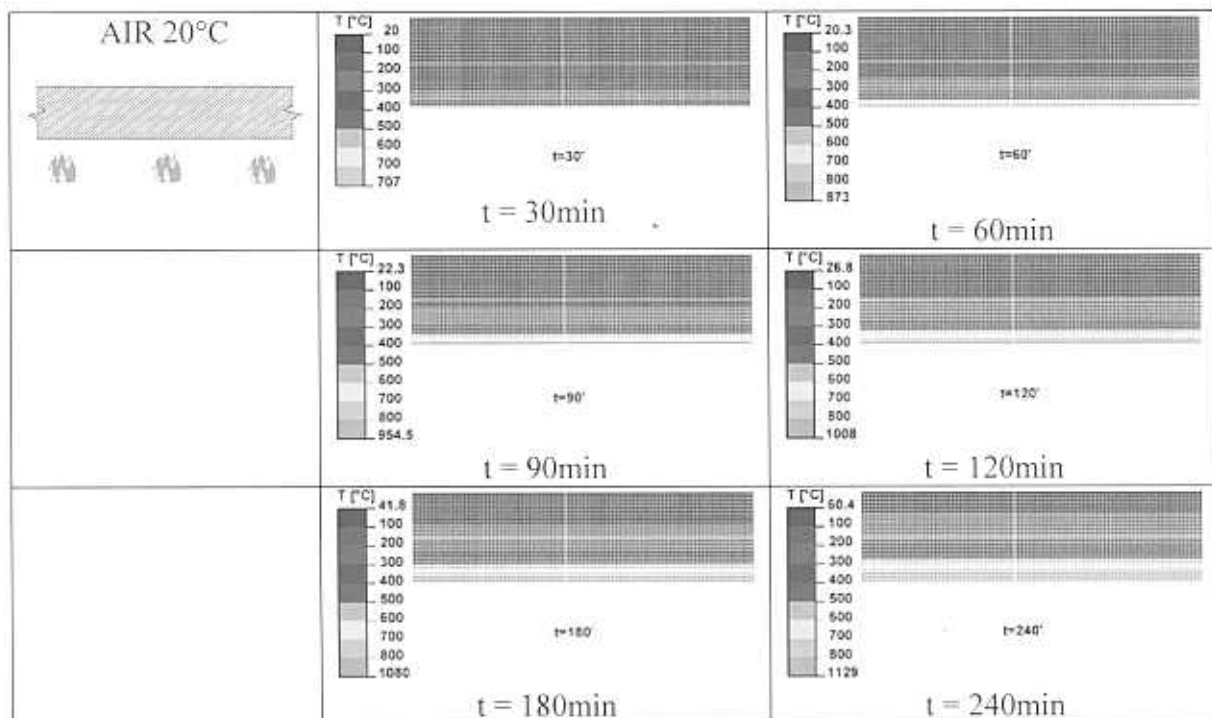


Fig. A4-6: Thermal analysis results of the 6m span one-way slab [°C]

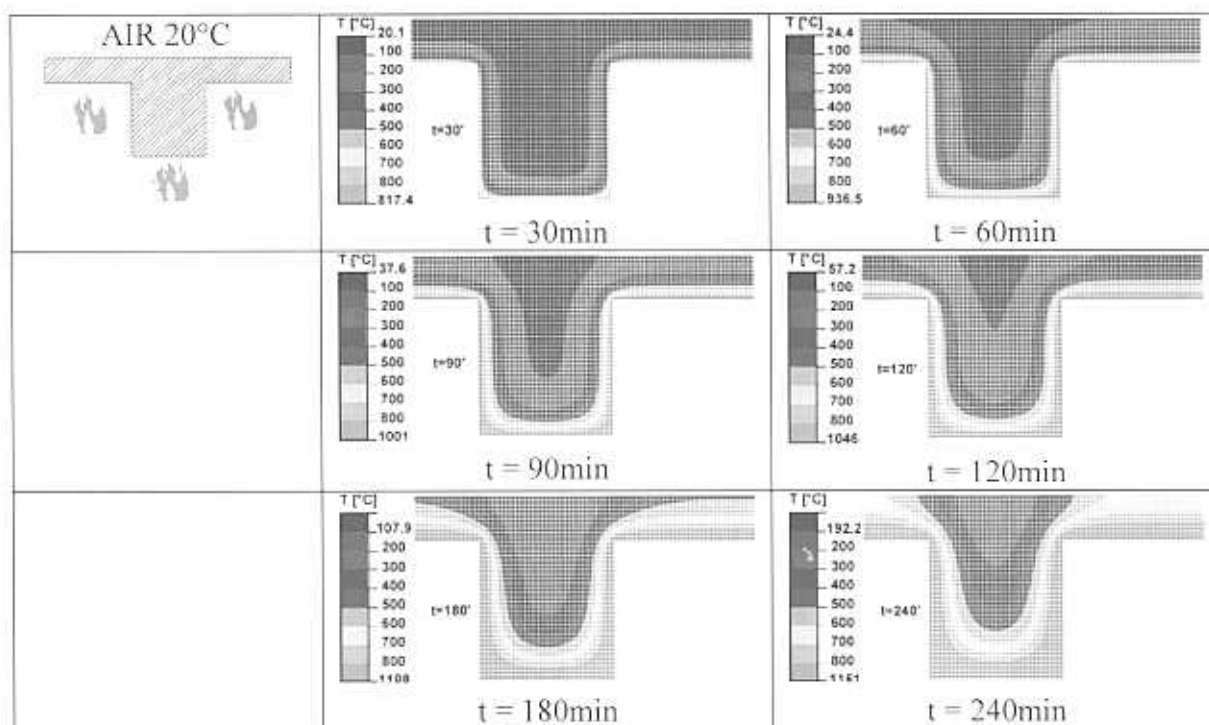


Fig. A4-7: Thermal analysis results of the 6m span T-beam [°C]

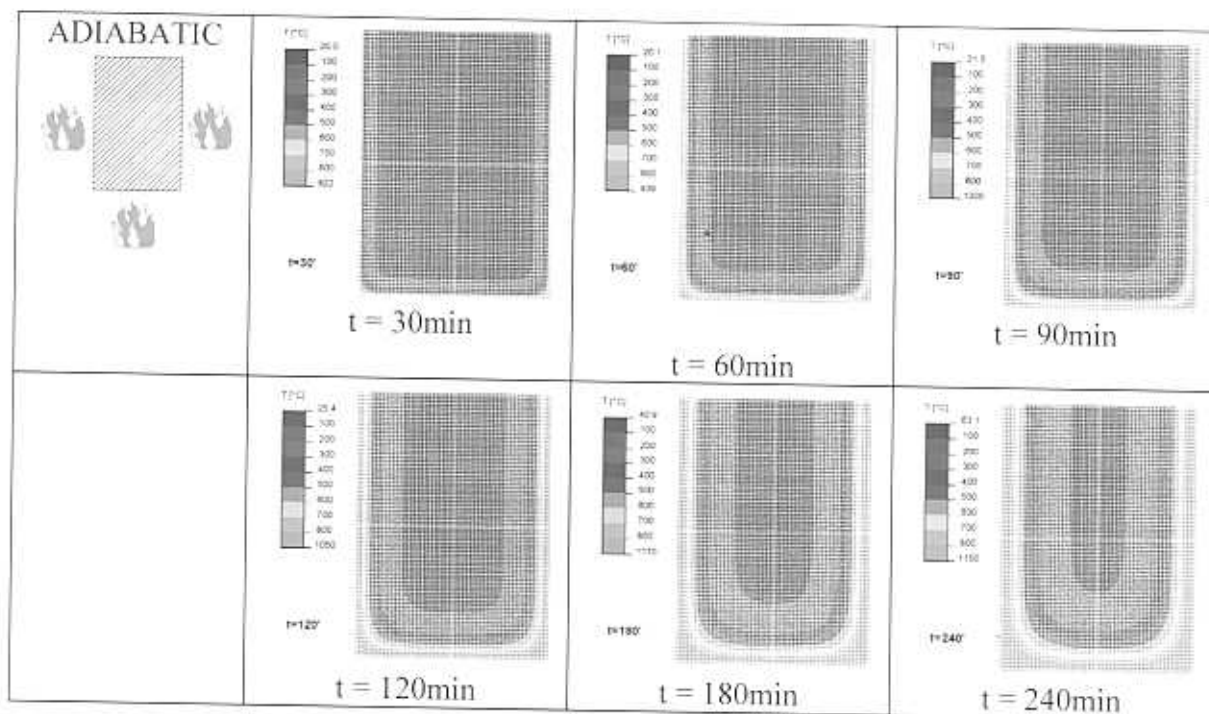


Fig. A4-8: Thermal analysis results of the 9m span rectangular beam [$^{\circ}\text{C}$]

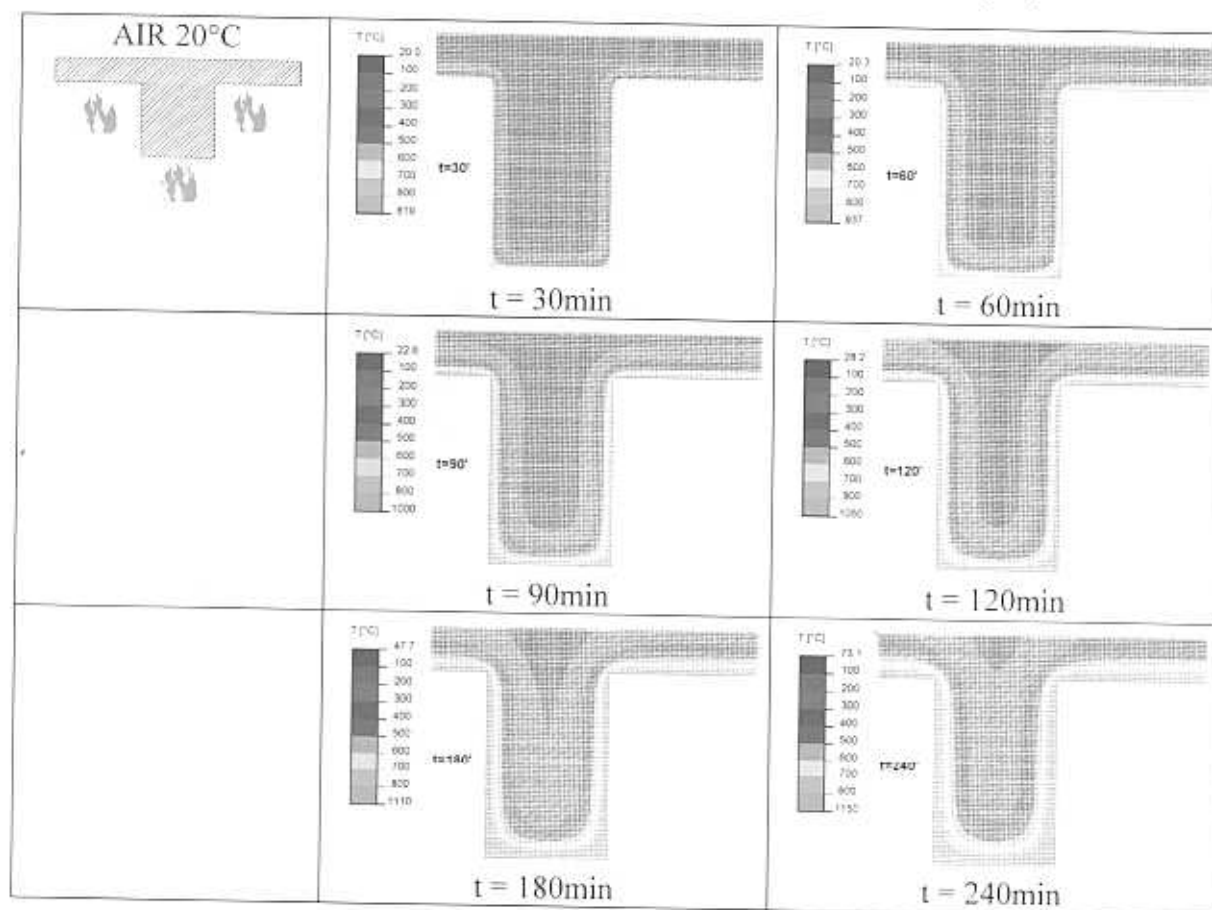


Fig. A4-9: Thermal analysis results of the 9m span T-beam [$^{\circ}\text{C}$]

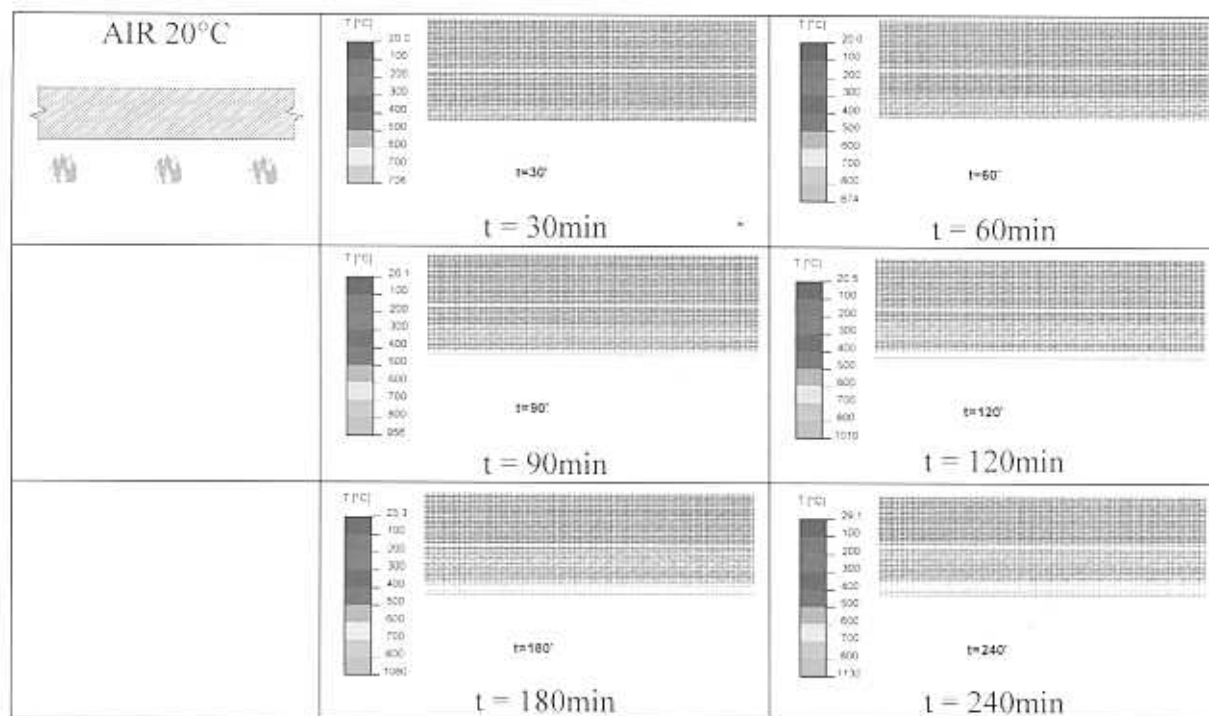


Fig. A4-10: Thermal analysis results of the 9m span one-way slab [°C]

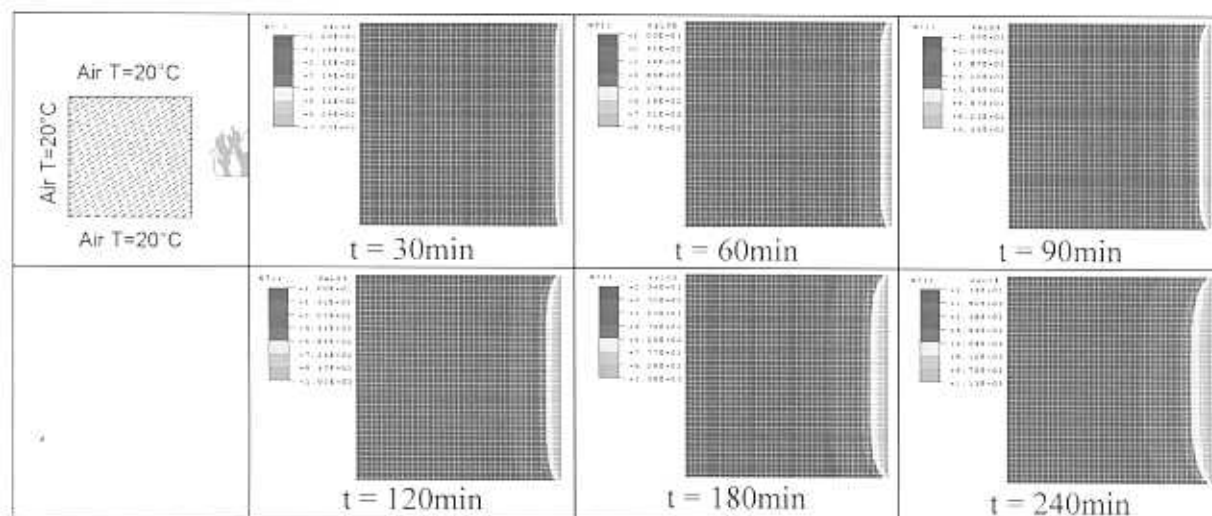


Fig. A4-11: Thermal analysis results for columns heated on one side [°C]

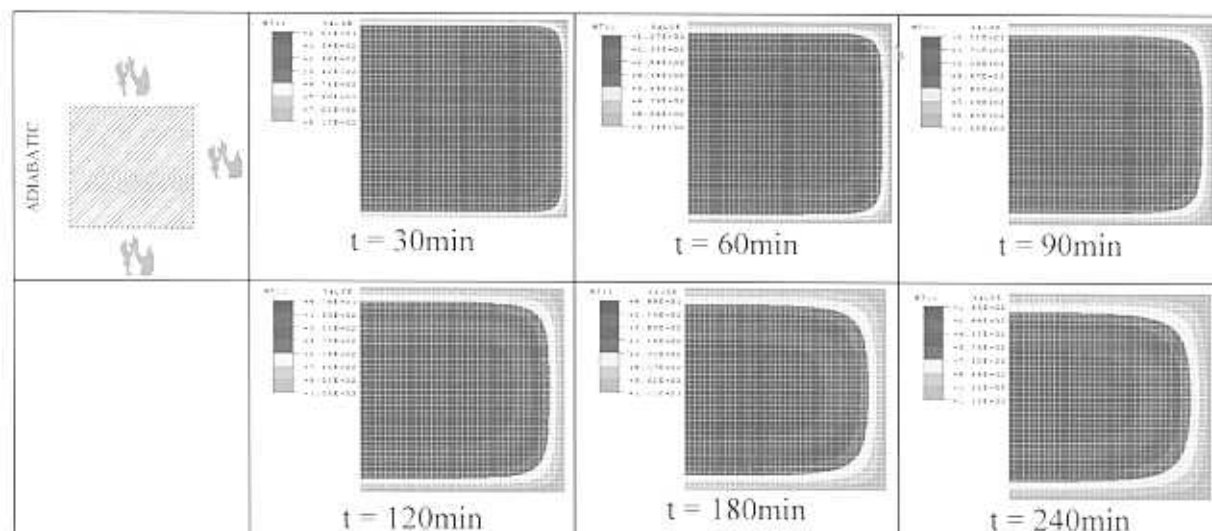


Fig. A4-12: Thermal analysis results for columns heated on three sides [°C]

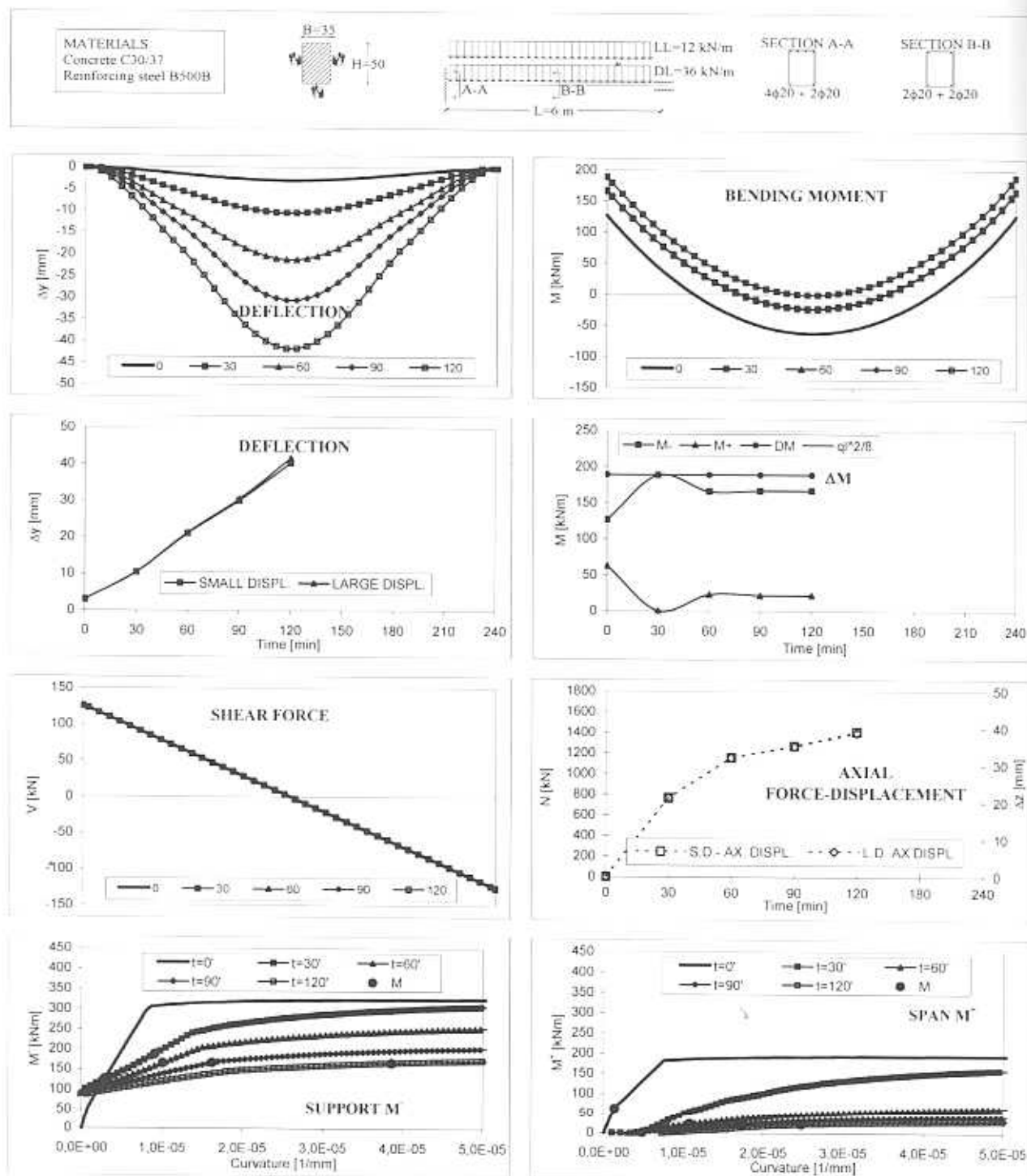


Fig. A4-13: Behaviour of a 6m span rectangular beam with axial restraint of stiffness $k=0$

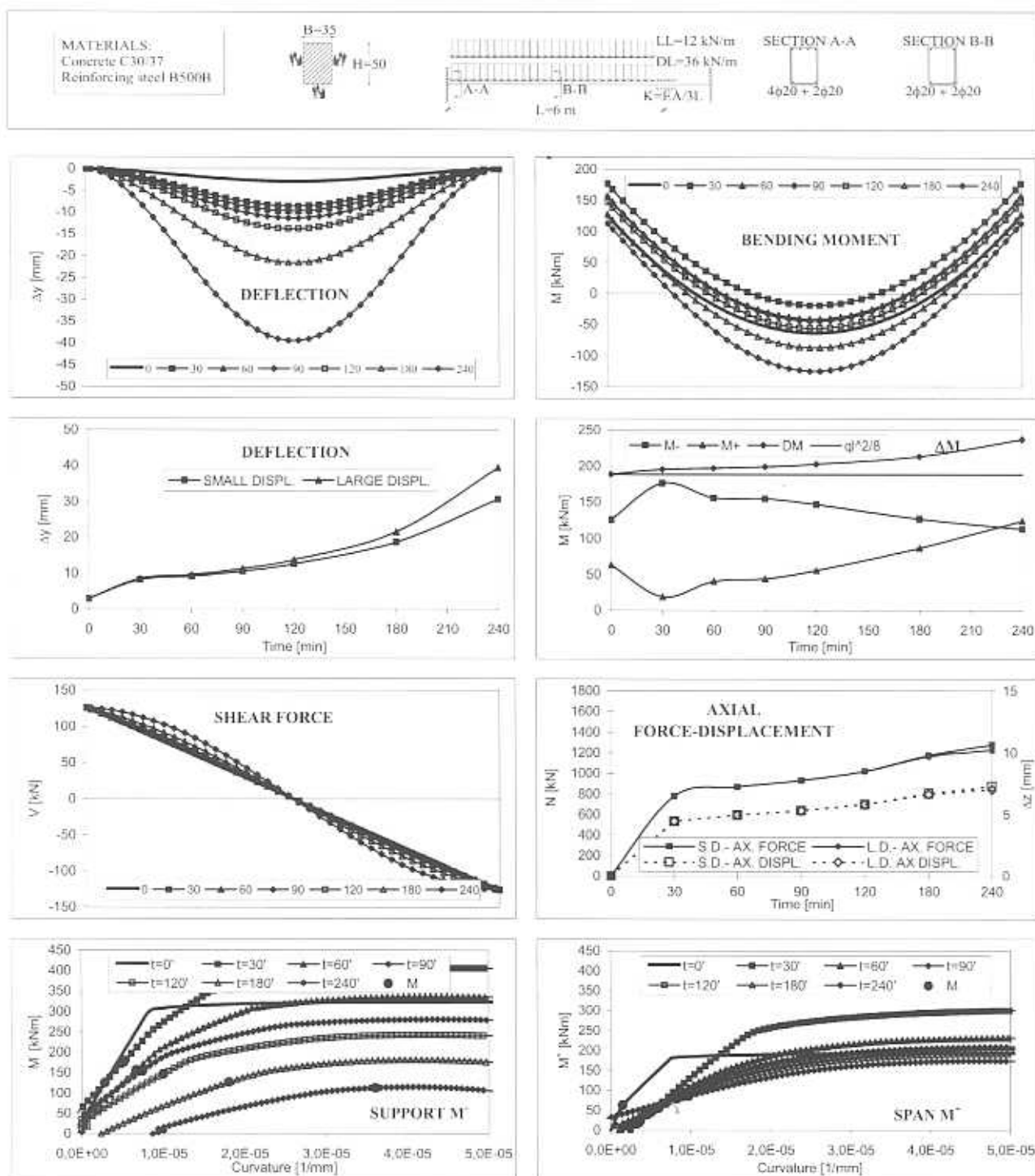


Fig. A4-14: Behaviour of a 6m span rectangular beam with axial restraint of stiffness $k = EA/3L$

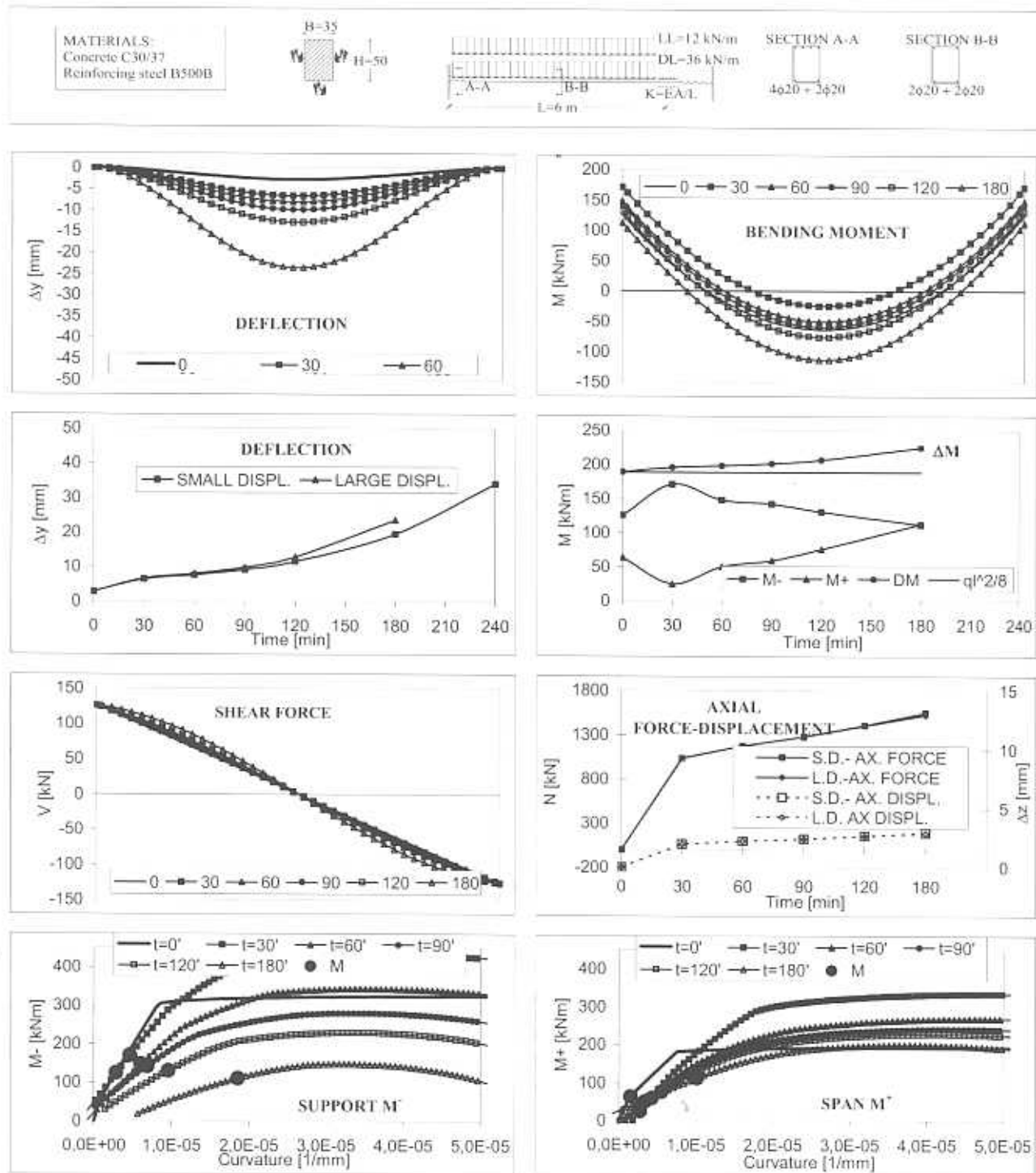


Fig. A4-15: Behaviour of a 6m span rectangular beam with axial restraint of stiffness $k = EA/L$

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



LL=12 kN/m
DL=36 kN/m

SECTION A-A
4ø20 + 2ø20

SECTION B-B
2ø20 + 2ø20

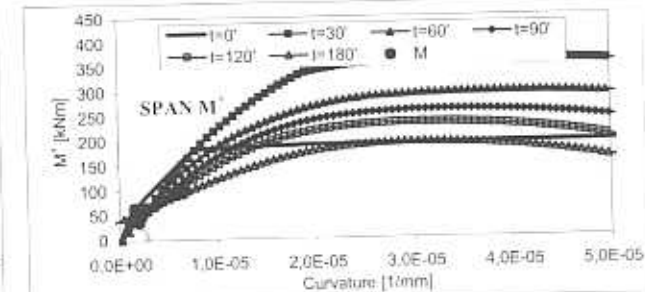
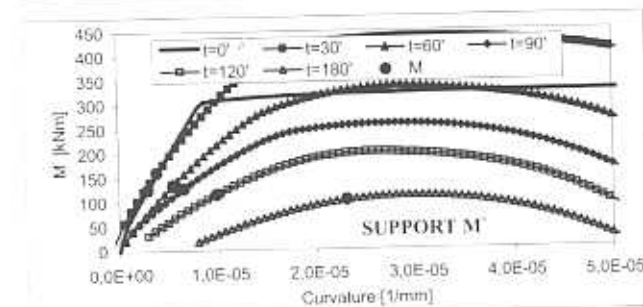
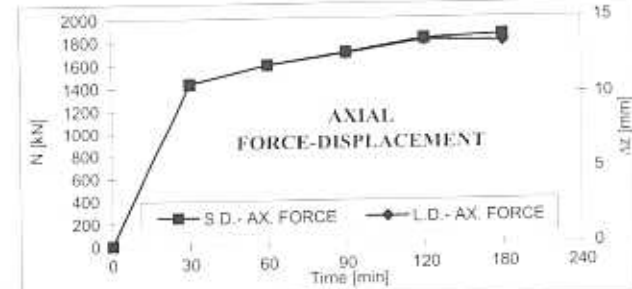
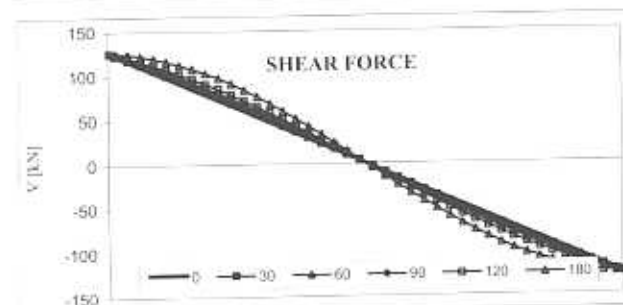
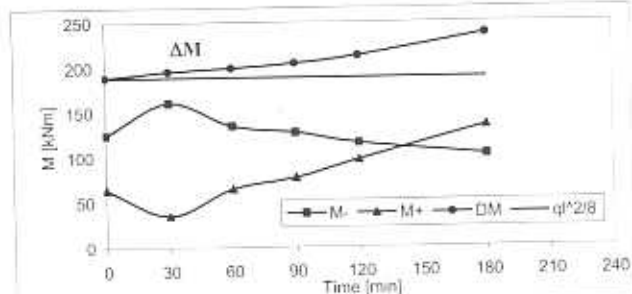
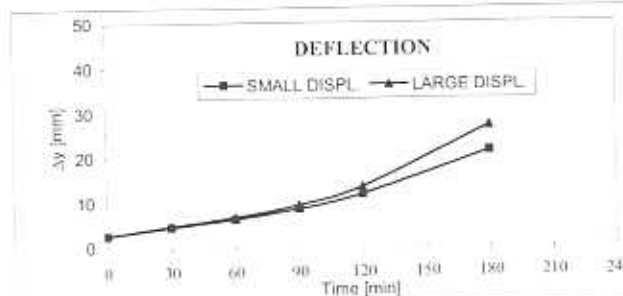
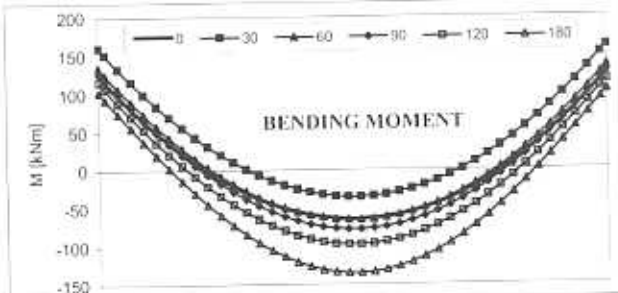
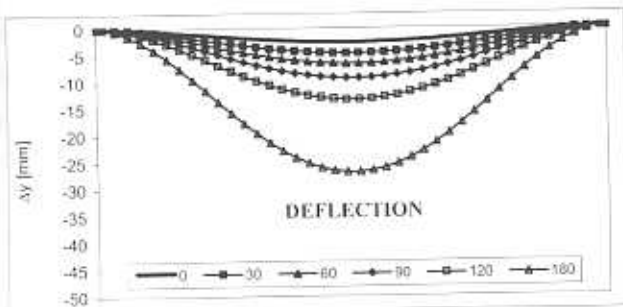


Fig. A4-16: Behaviour of a 6m span rectangular beam with axial restraint of stiffness $k = \infty$

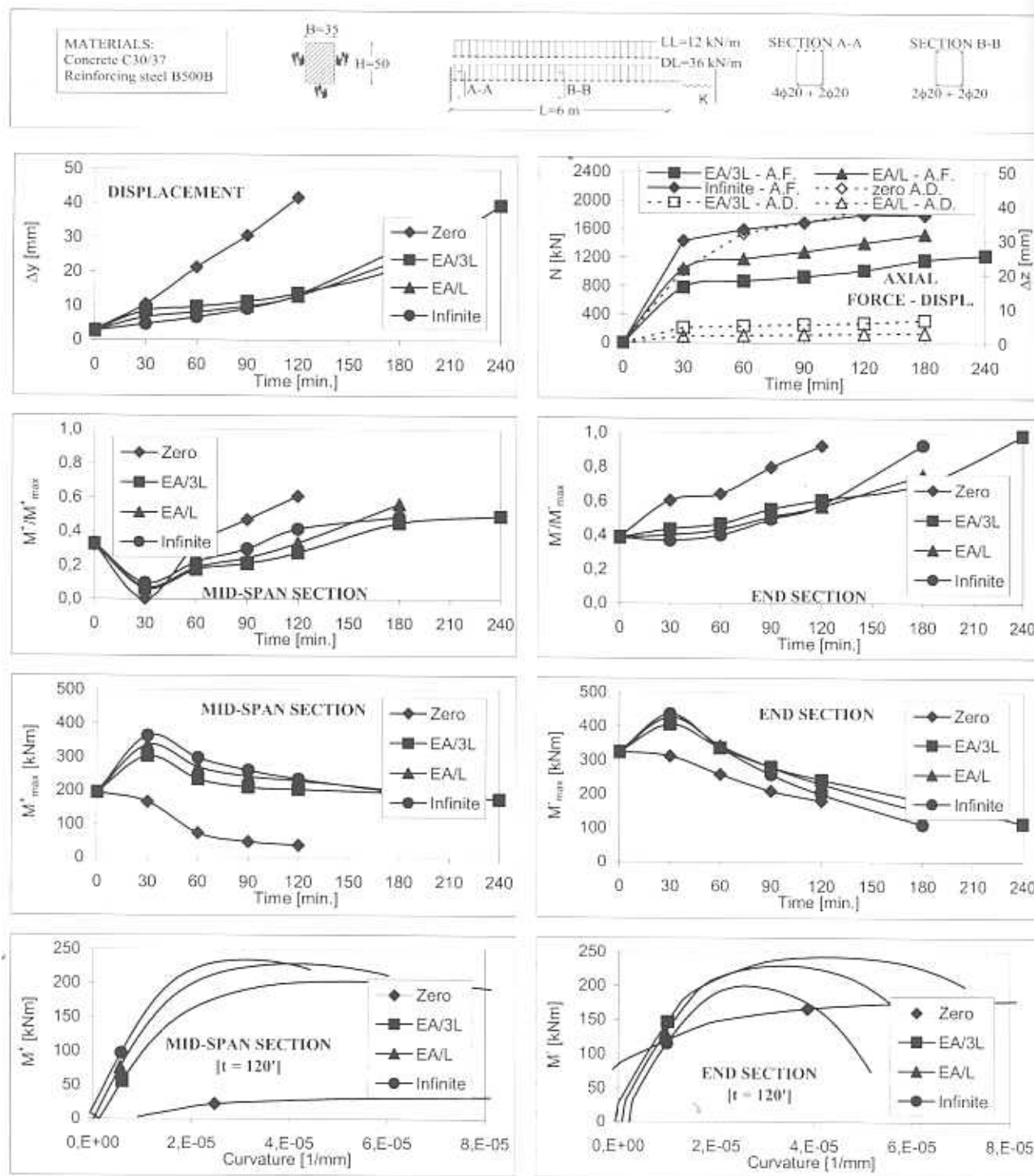


Fig. A4-17: Effect of the axial restraint stiffness on the behaviour of a 6m span rectangular beam

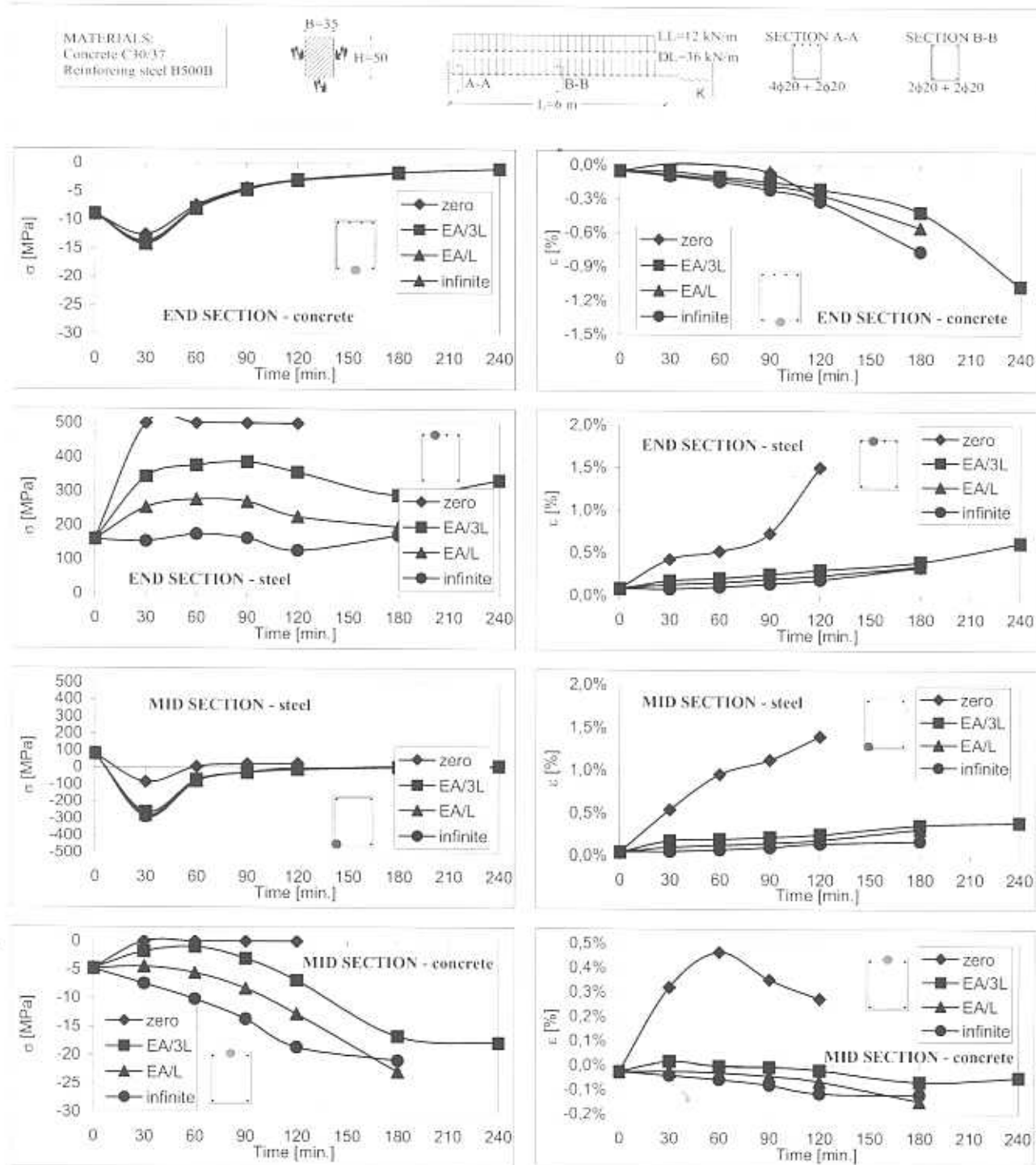


Fig. A4-18: Effect of the axial restraint stiffness on the stress and strain time-histories for a 6m rectangular beam

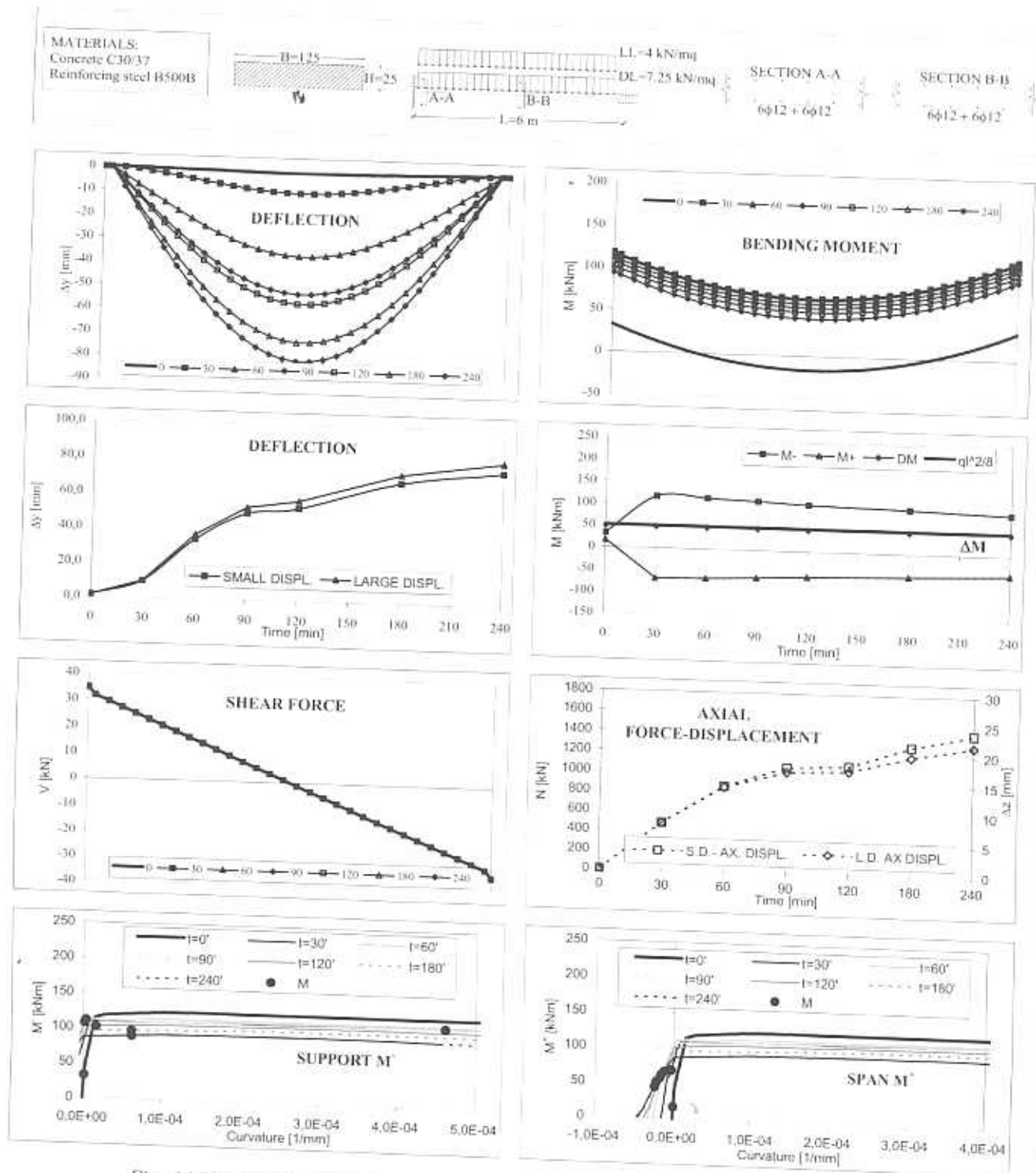
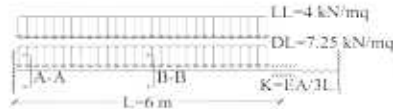


Fig. A4-19: Behaviour of a 6m span one-way slab with axial restraint of stiffness $k=0$

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



SECTION A-A

SECTION B-B

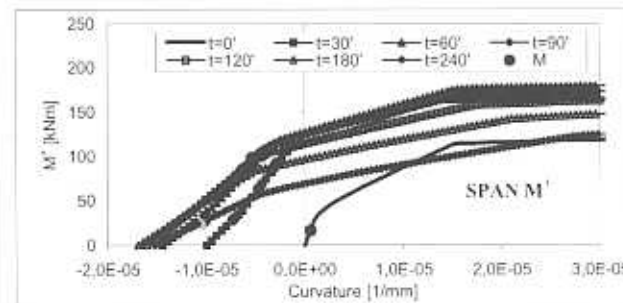
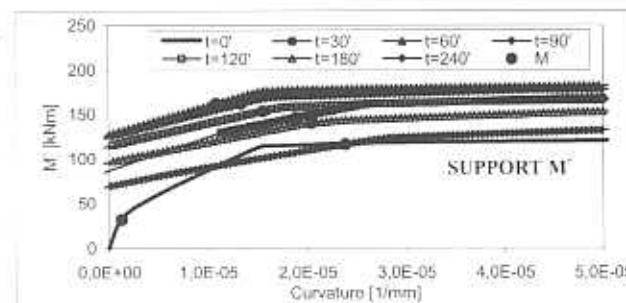
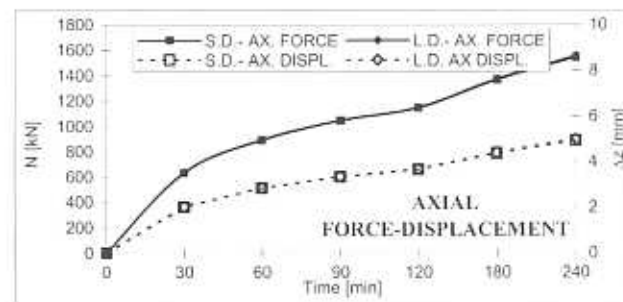
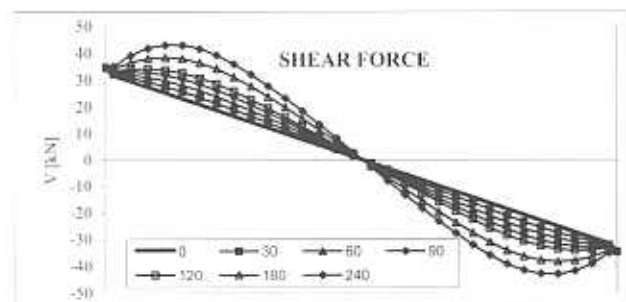
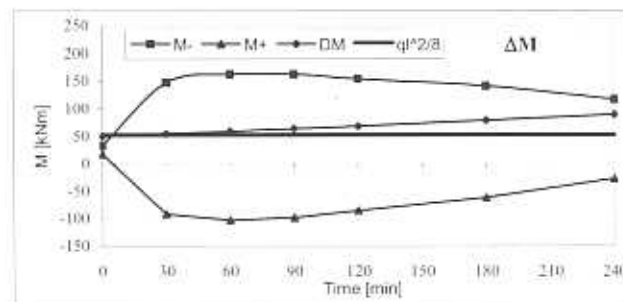
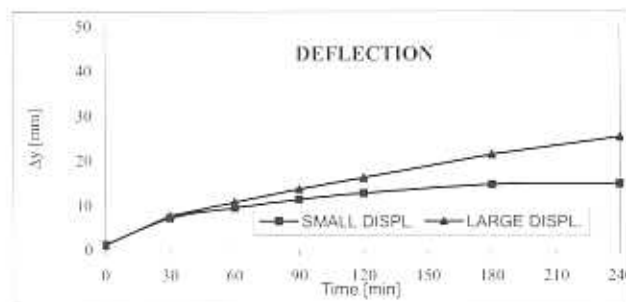
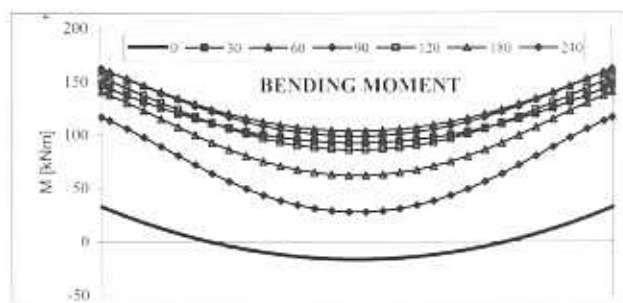
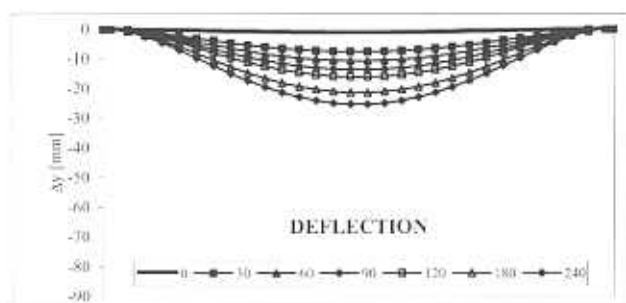


Fig. A4-20: Behaviour of a 6m span one-way slab with axial restraint of stiffness $k = EA/3L$

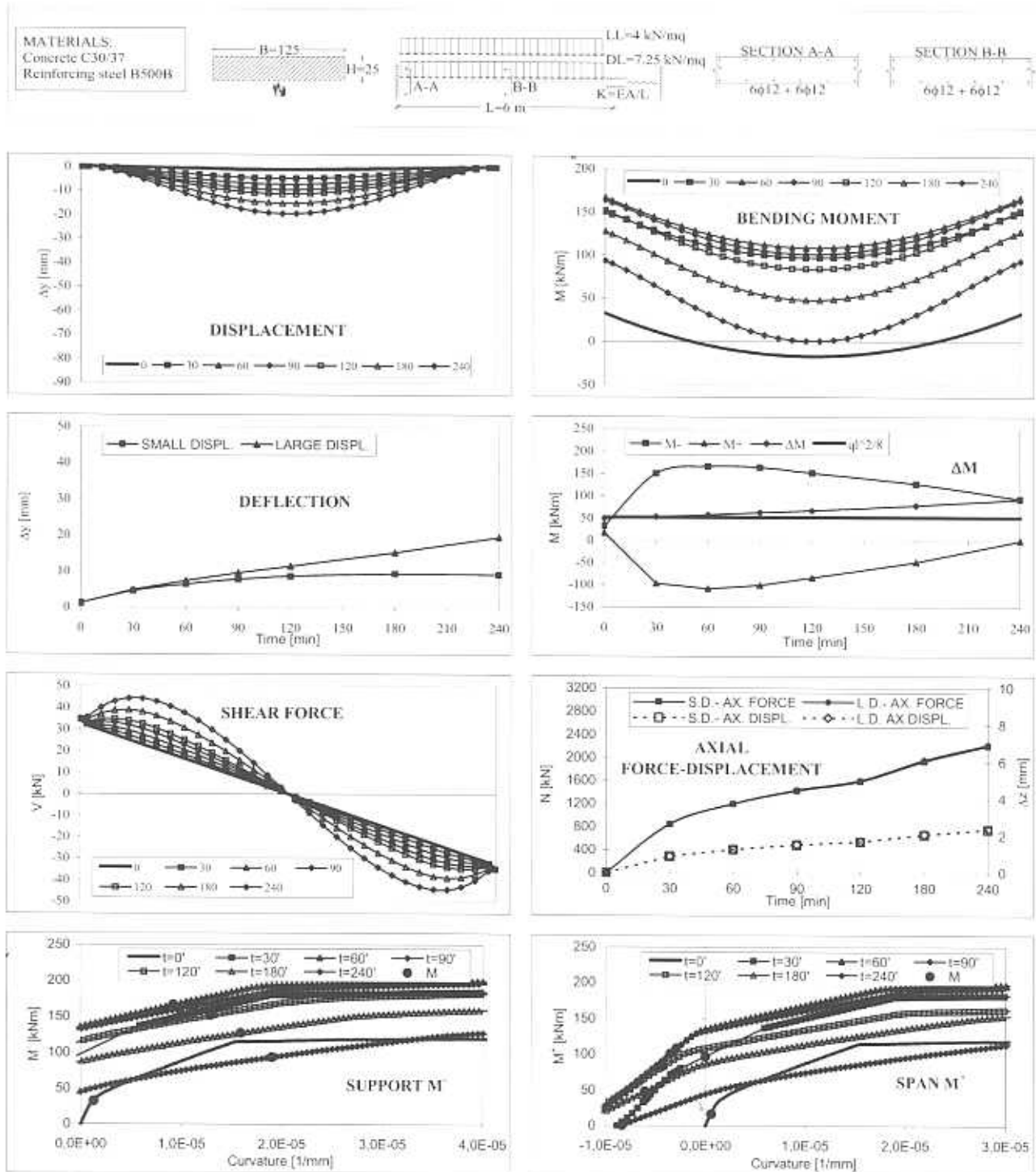
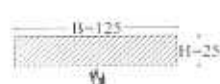


Fig. A4-21: Behaviour of a 6m span one-way slab with axial restraint of stiffness $k= EA/L$.

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



LL=4 kN/mq

DL=7.25 kN/mq

SECTION A-A

6 ϕ 12 + 6 ϕ 12

SECTION B-B

6 ϕ 12 + 6 ϕ 12

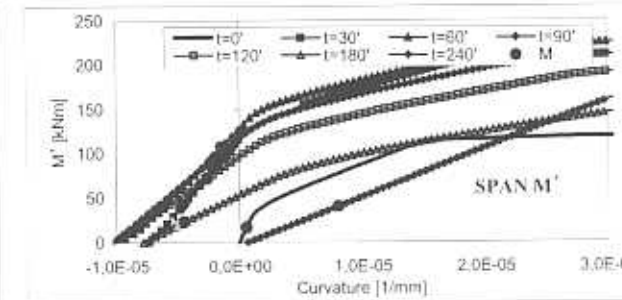
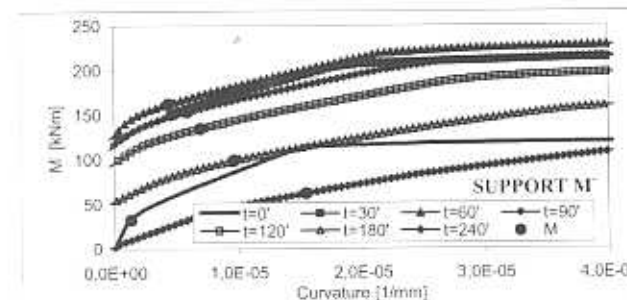
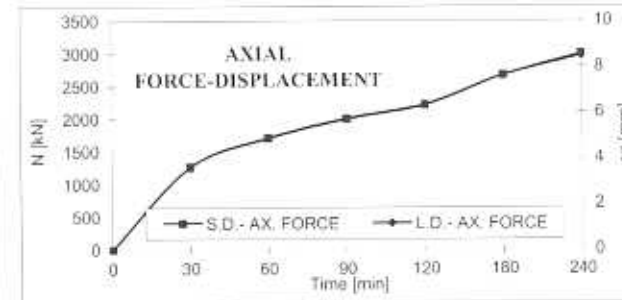
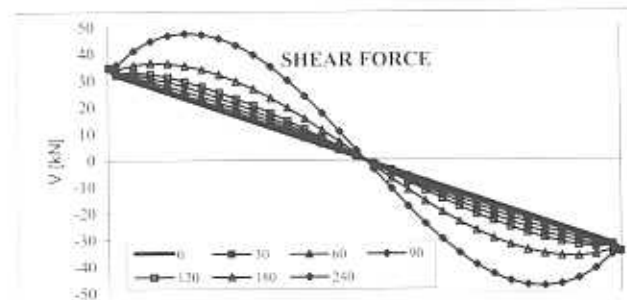
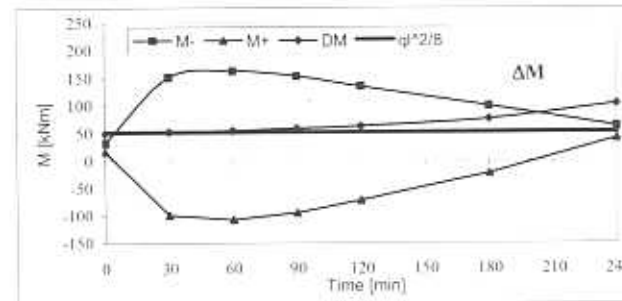
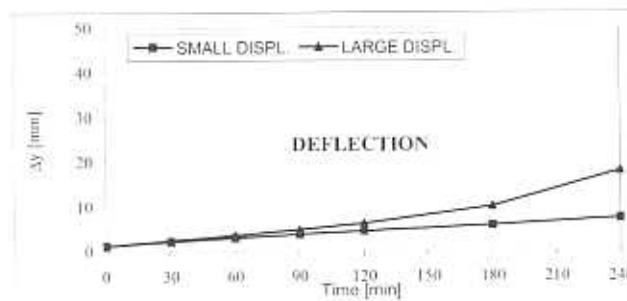
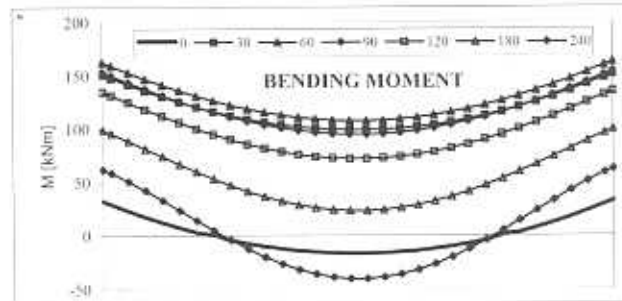
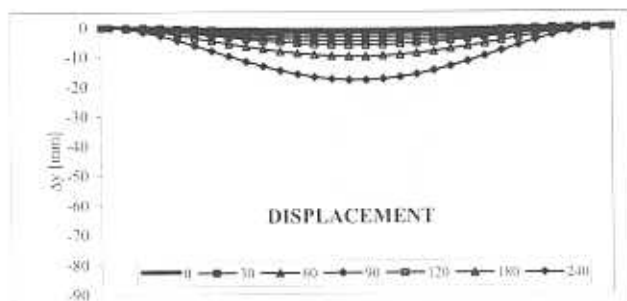


Fig. A4-22: Behaviour of a 6m span one-way slab with axial restraint of stiffness $k = \infty$

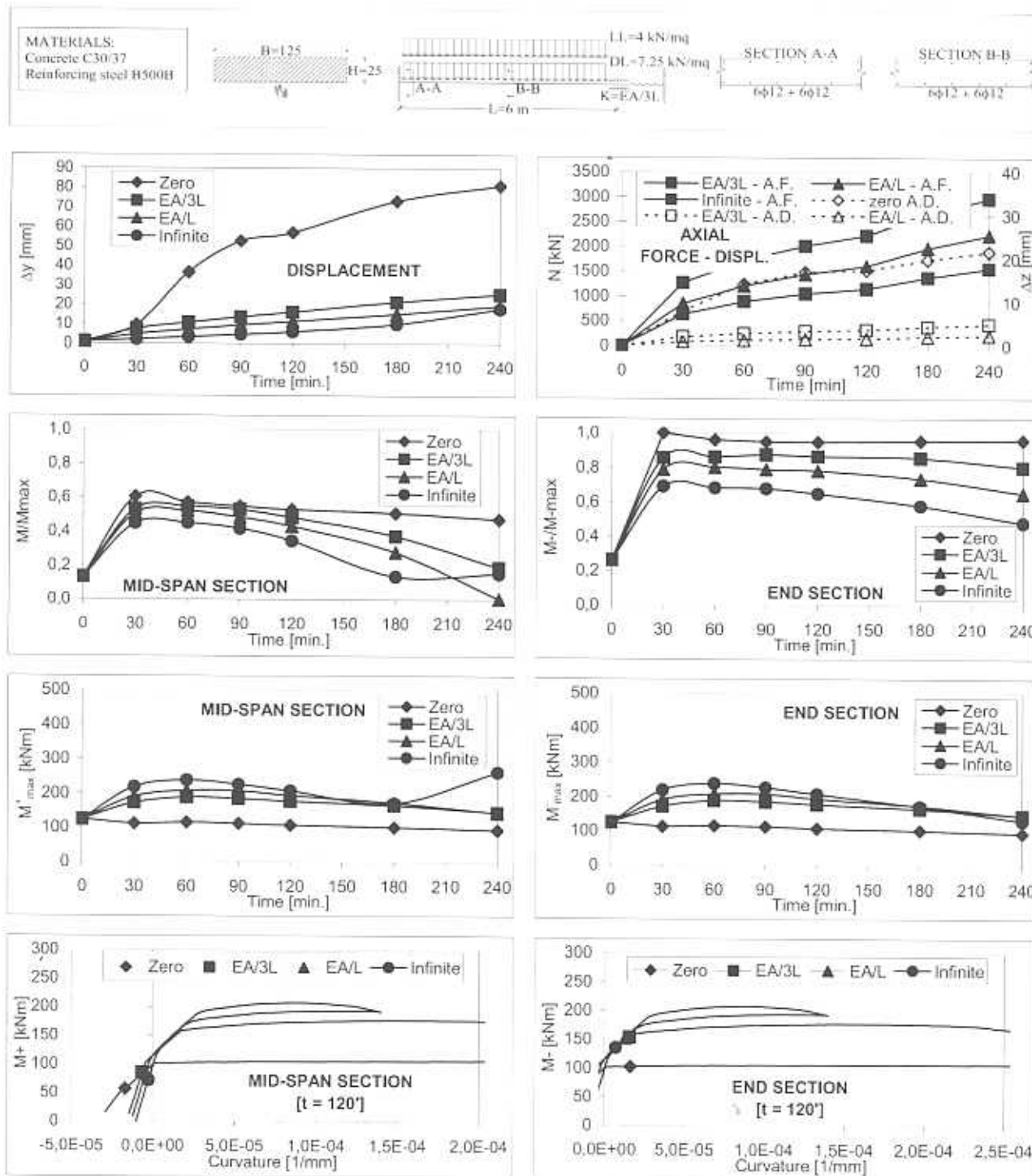


Fig. A4-23: Effect of the axial restraint stiffness on the behaviour of a 6m span one-way slab

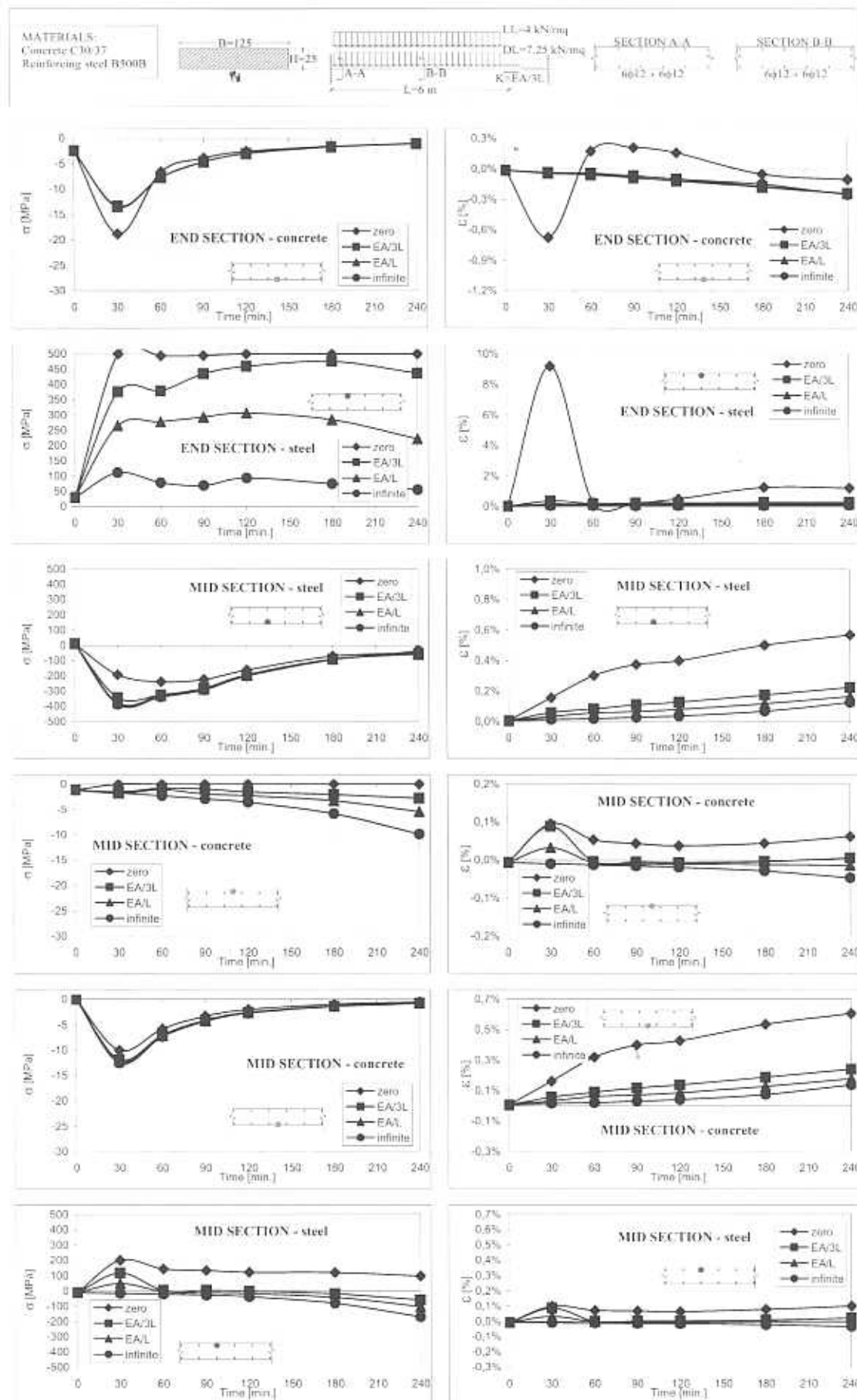


Fig. A4-24: Effect of the axial restraint stiffness on the stress and strain time-histories for a 6m one-way slab

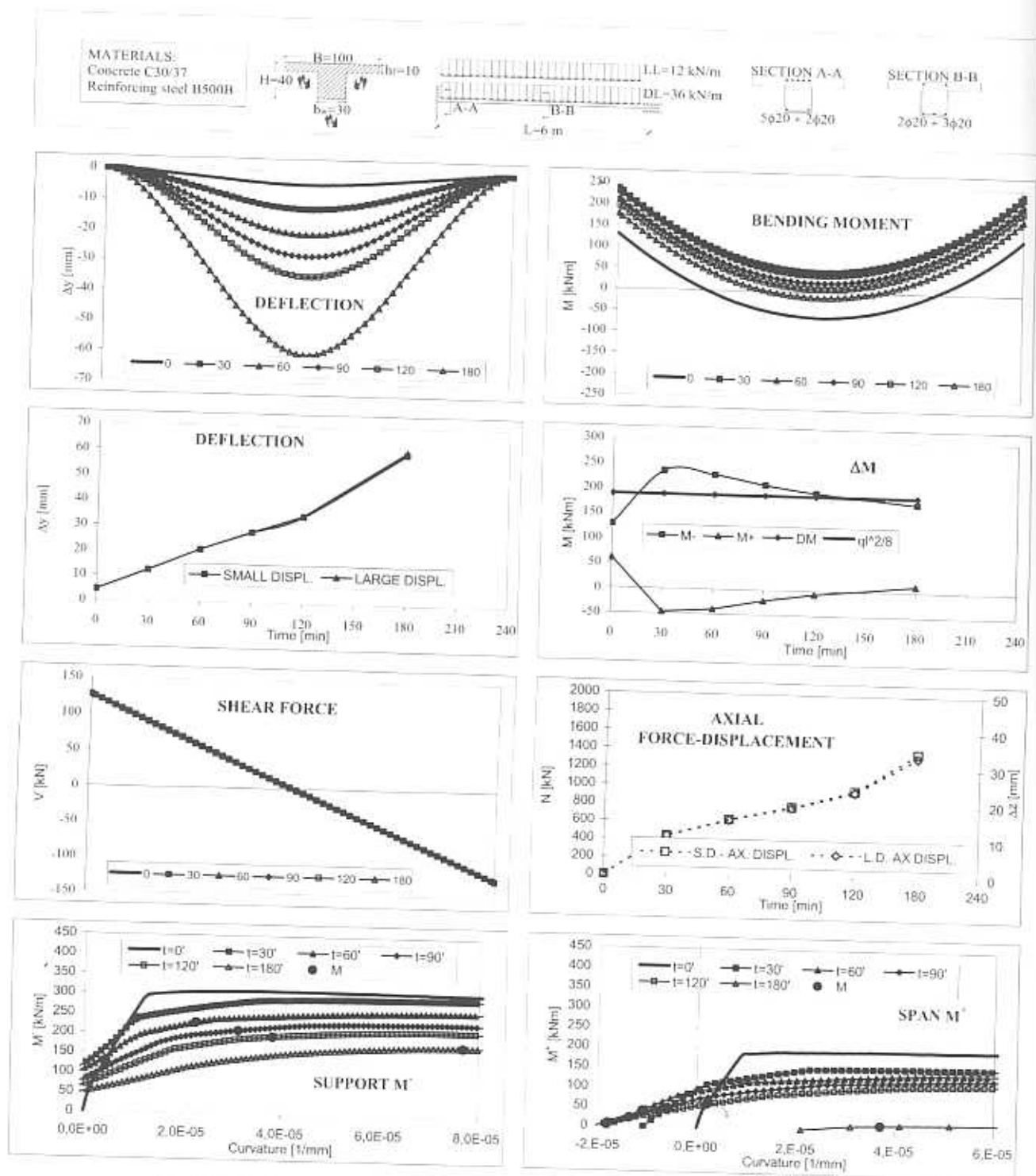


Fig. A4-25: Behaviour of a 6m span T-beam with axial restraint of stiffness $k=0$

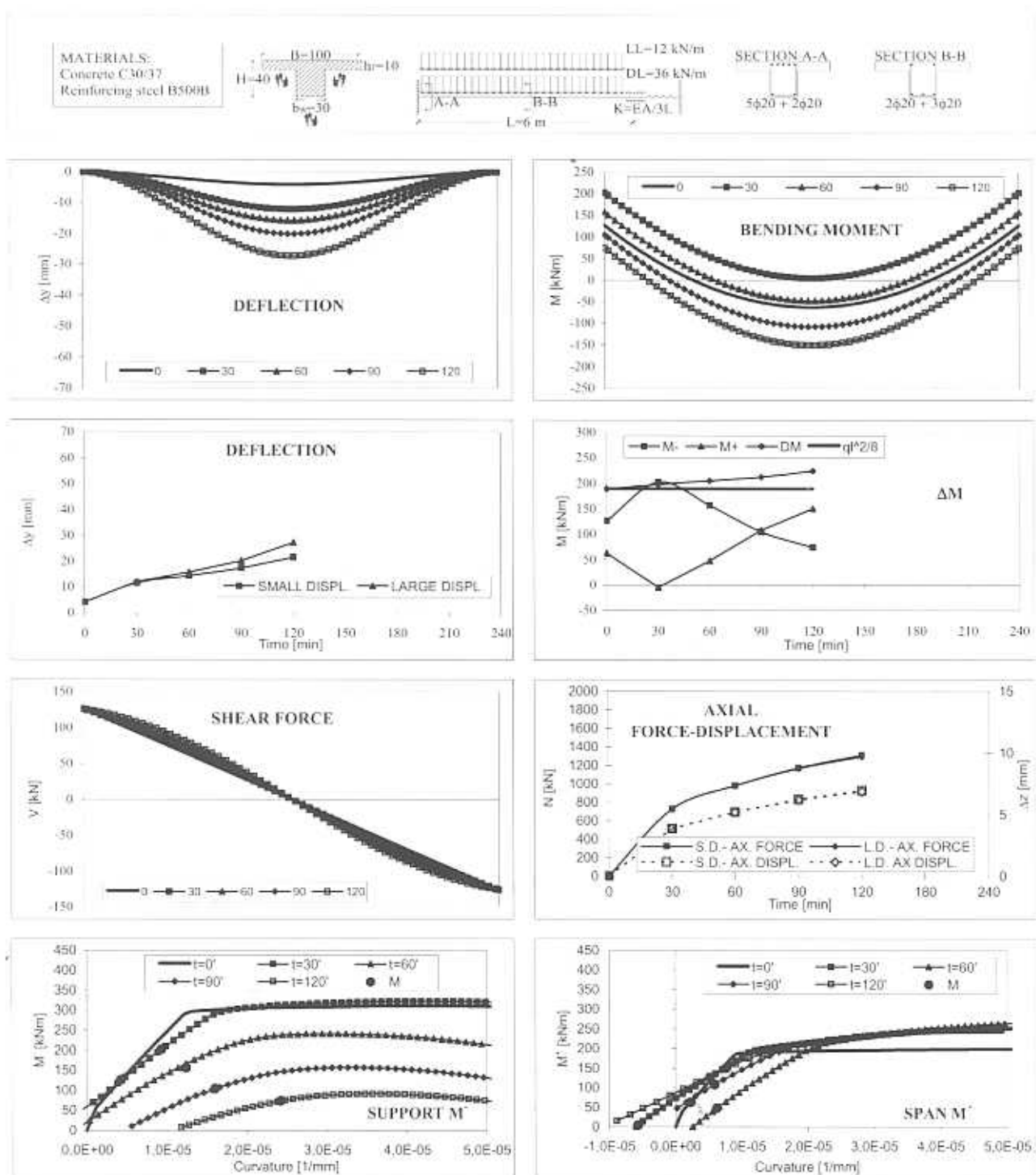


Fig. A4-26: Behaviour of a 6m span T-beam with axial restraint of stiffness $k= EA/3L$.

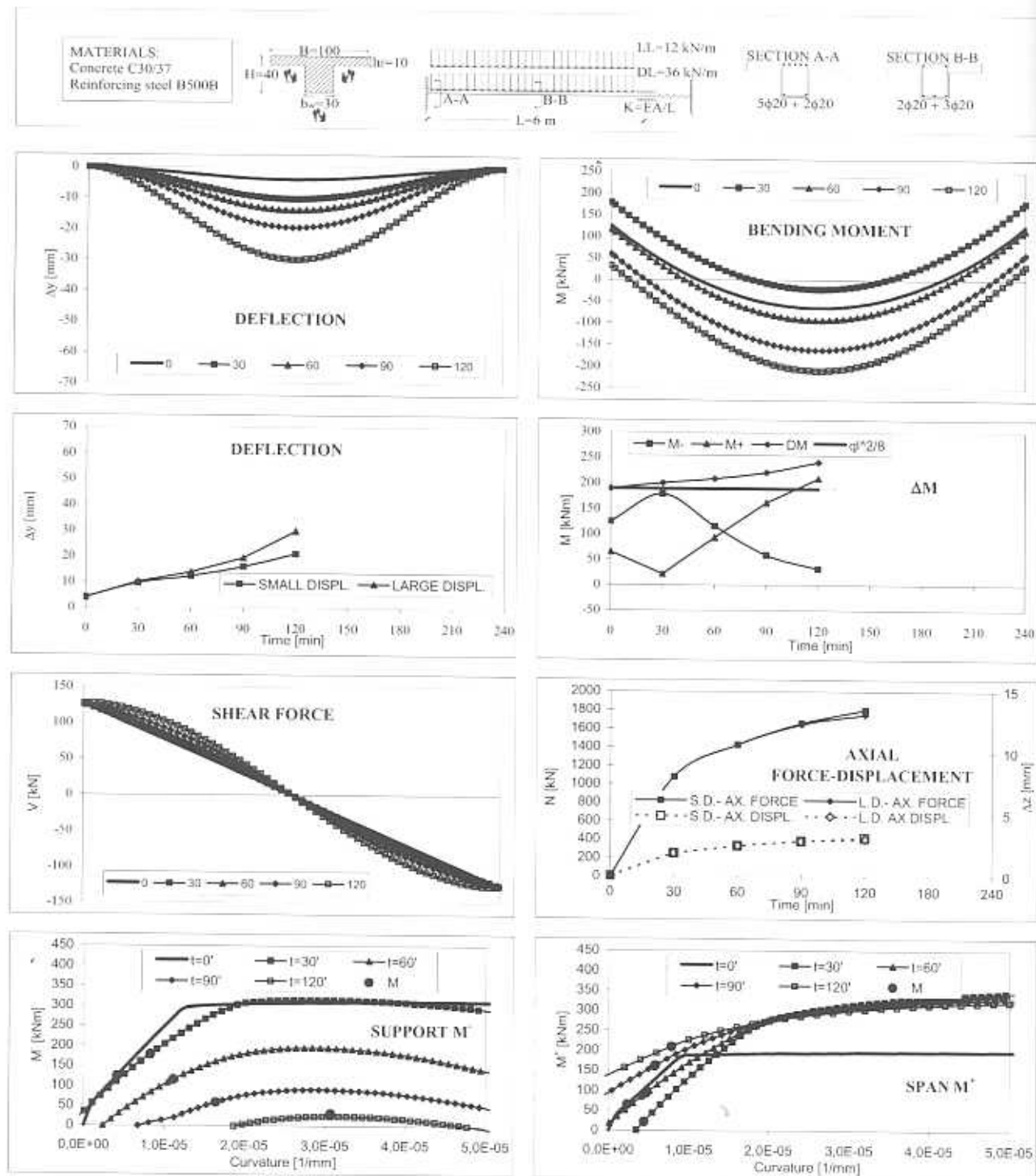


Fig. A4-27: Behaviour of a 6m span T-beam with axial restraint of stiffness $k= EA/L$

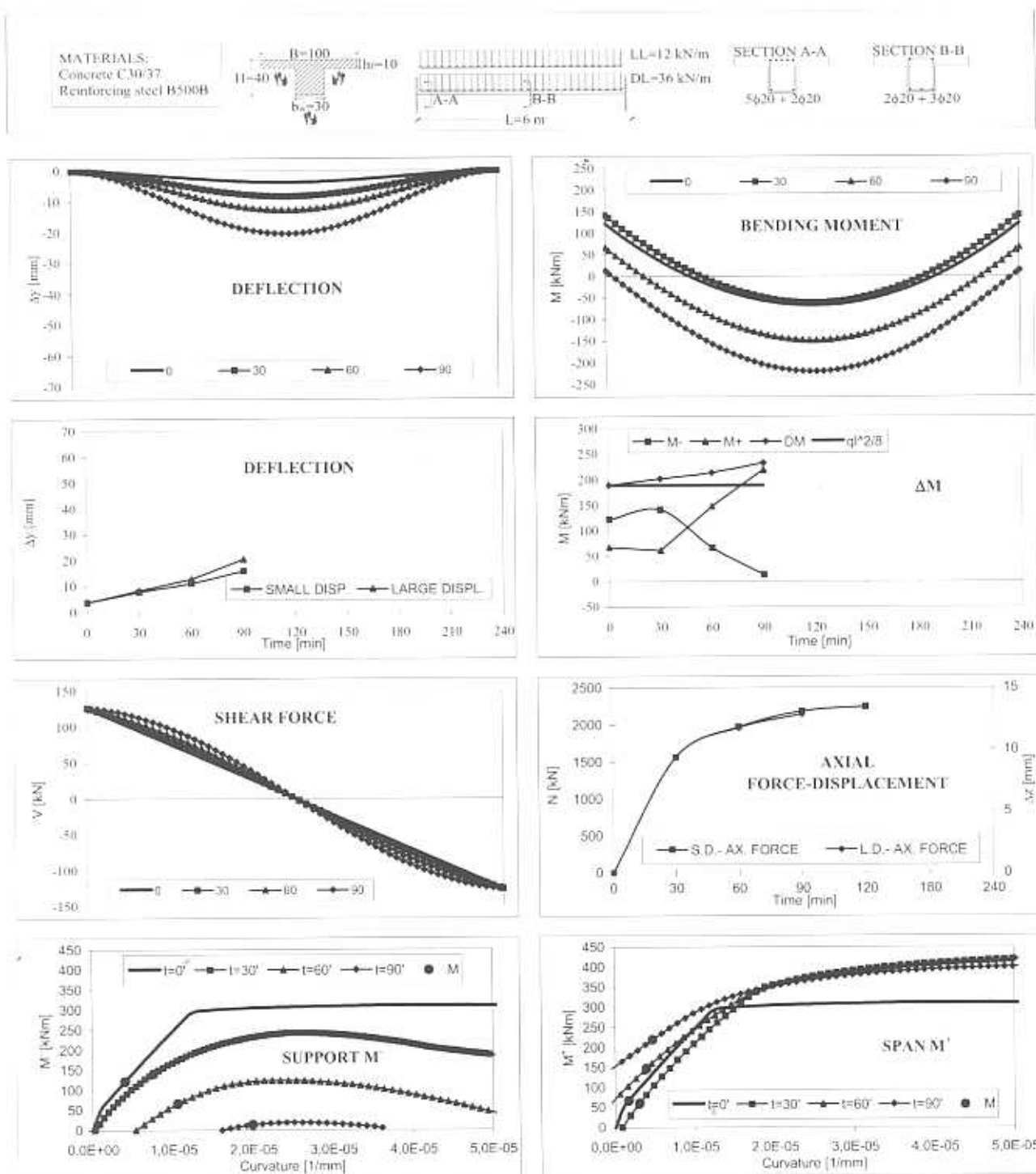


Fig. A4-28: Behaviour of a 6m span T-beam with axial restraint of stiffness $k = \infty$.

MATERIALS:
Concrete C30/37
Reinforcing steel B500B

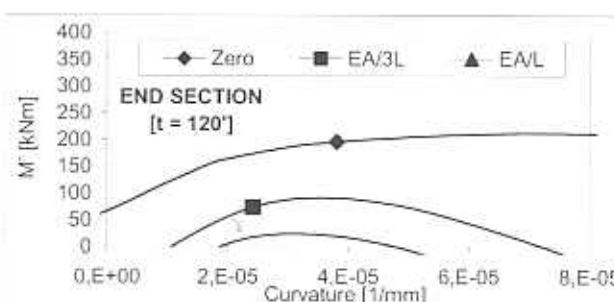
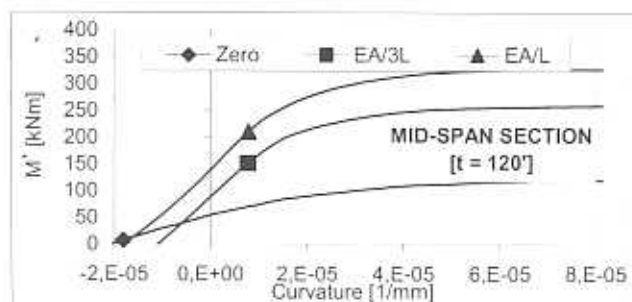
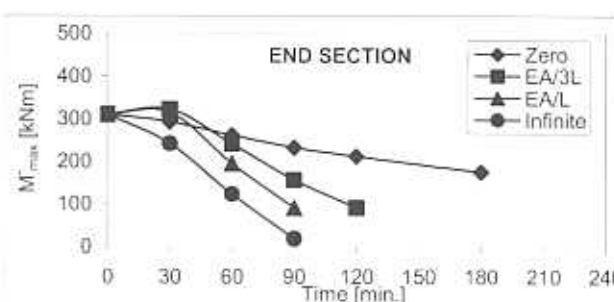
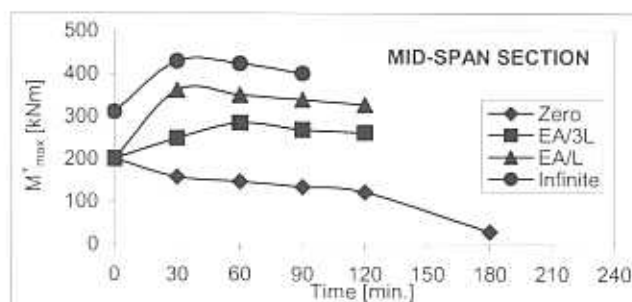
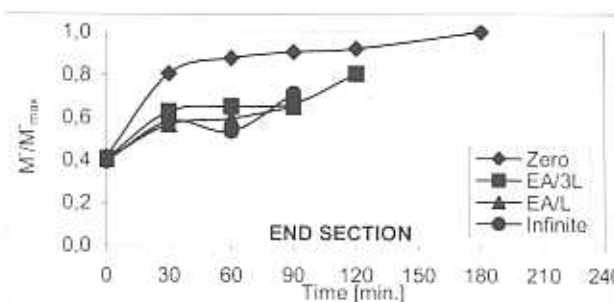
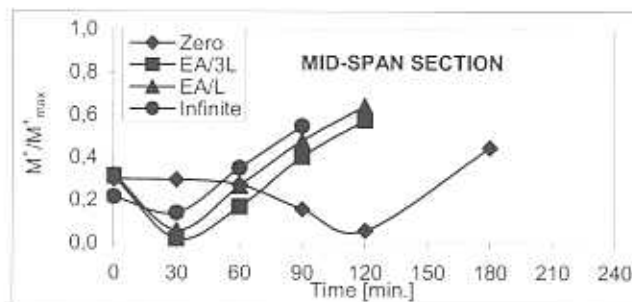
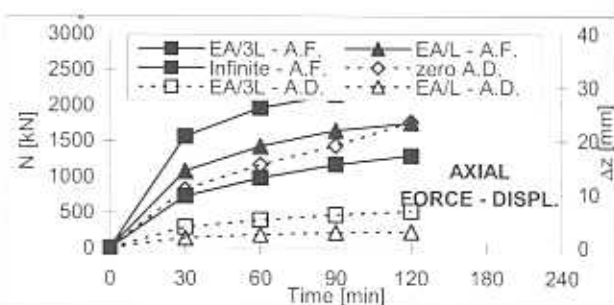
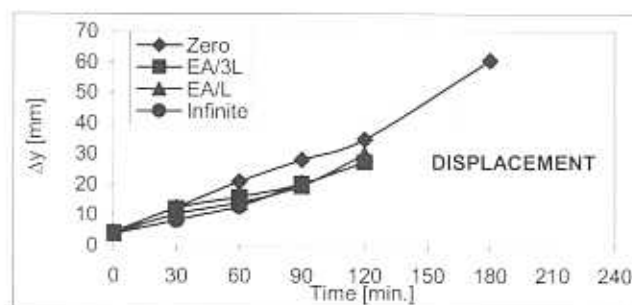
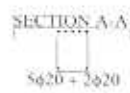
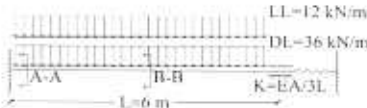
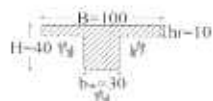


Fig. A4-29: Effect of the axial restraint stiffness on the behaviour of a 6m span T-beam

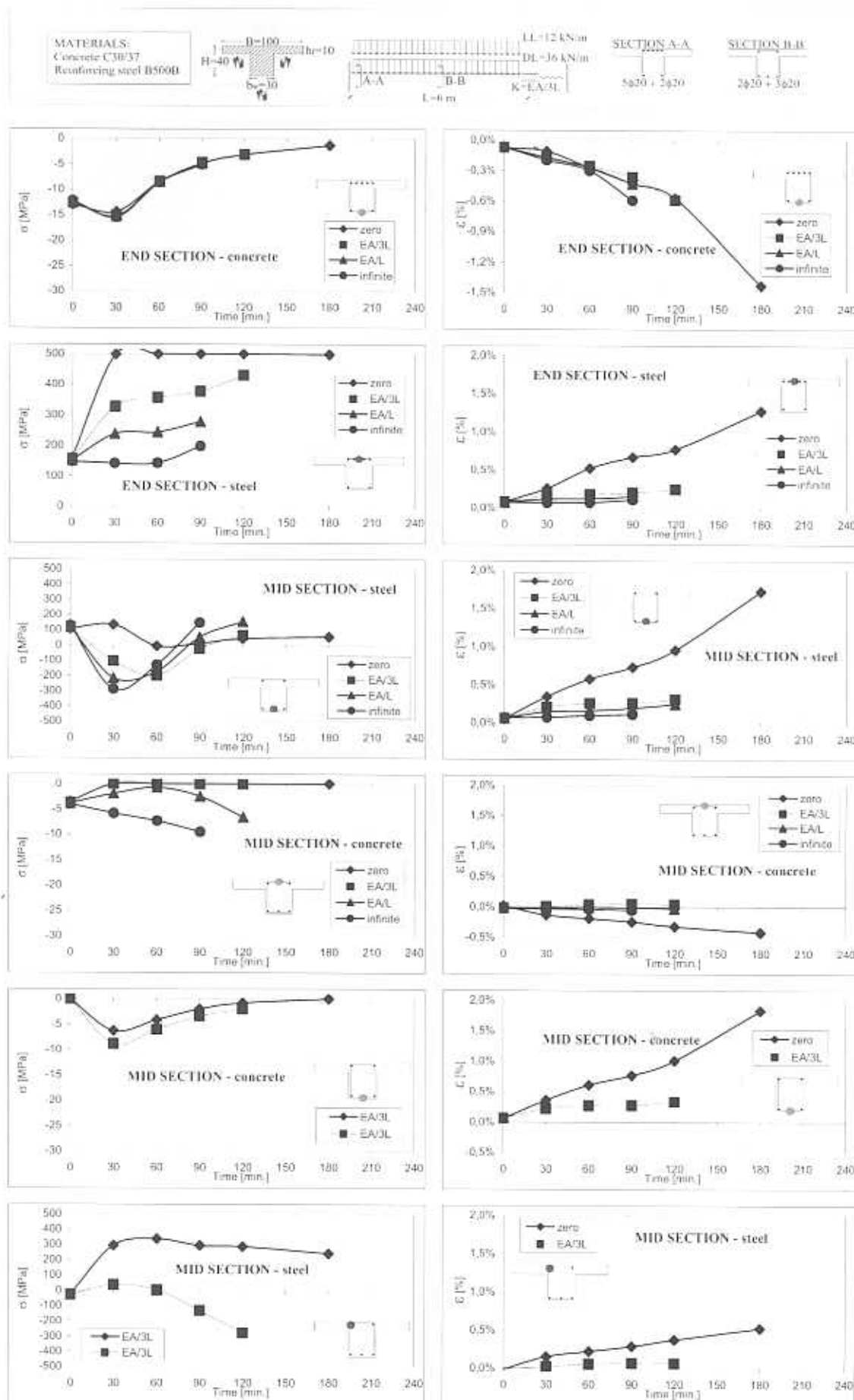
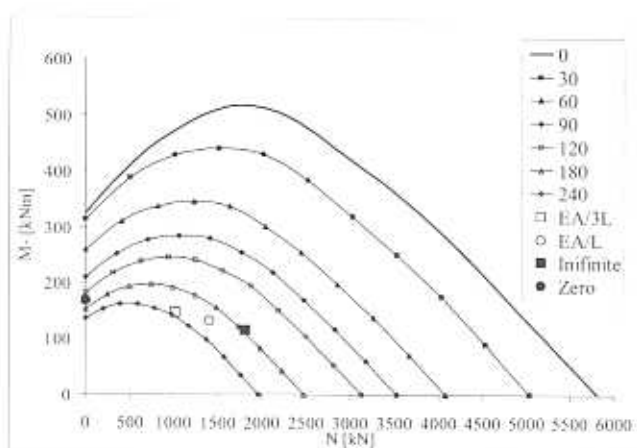
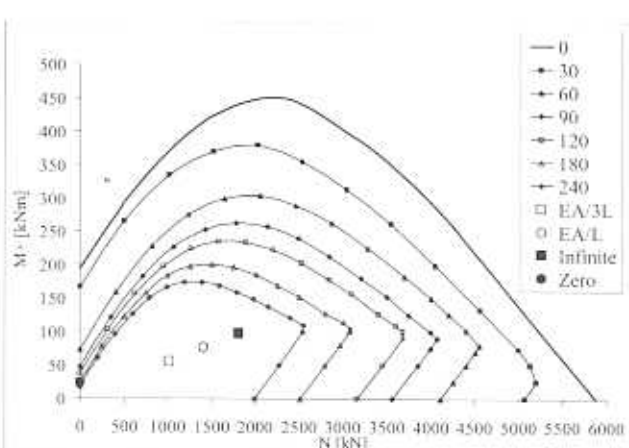


Fig. A4-30: Effect of the axial restraint stiffness on the stress and strain time-histories for a 6m T-beam

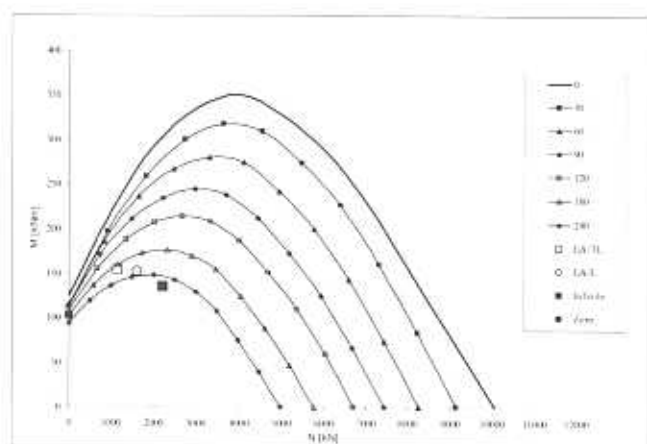


(a) Support section – Negative moment

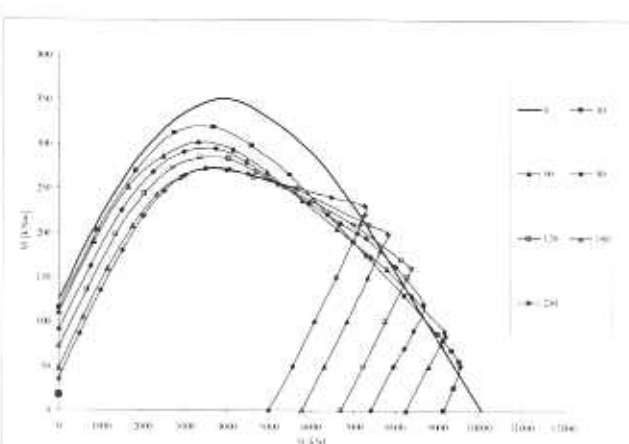


(b) Mid-span section – Positive moment

Fig. A4-31: Bending moment – axial force interaction curves; 6m span rectangular beam

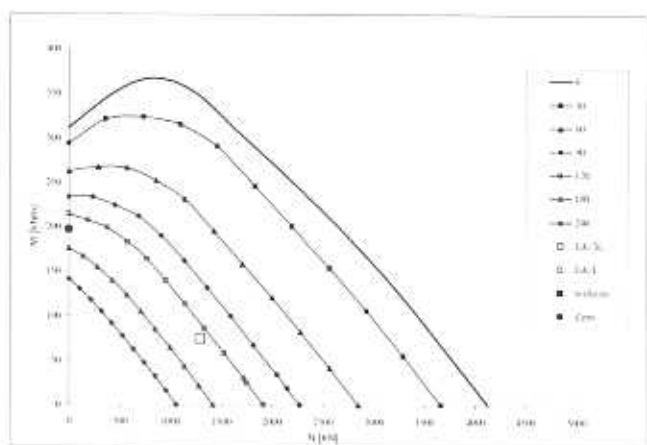


(a) Support section – Negative moment

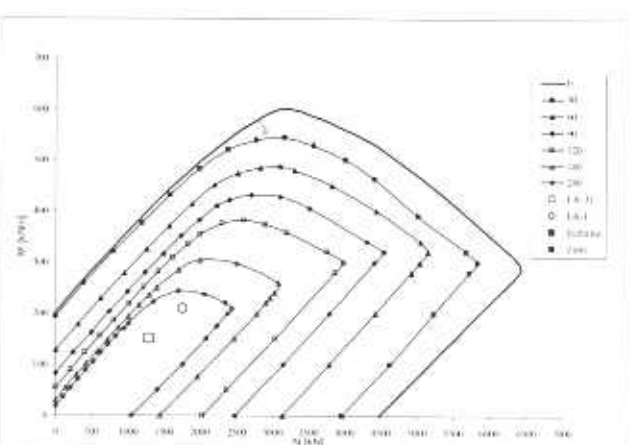


(b) Mid-span section – Positive moment

Fig. A4-32: Bending moment – axial force interaction curves; 6m span one-way slab



(a) Support section – Negative moment



(b) Mid-span section – Positive moment

Fig. A4-33: Bending moment – axial force interaction curves; 6m span T-beam

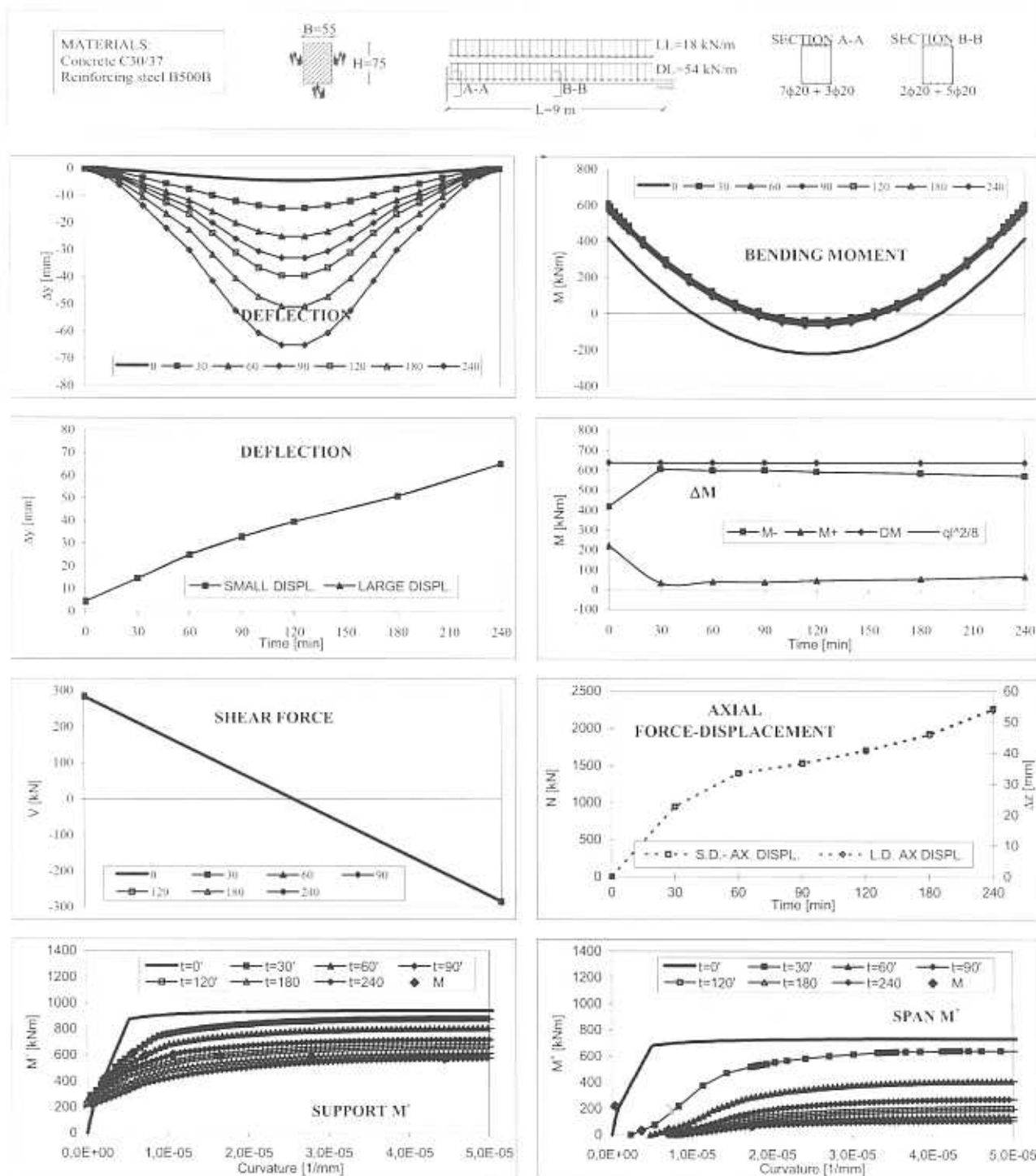
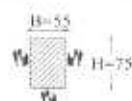


Fig. A4-34: Behaviour of a 9m span rectangular beam with axial restraint of stiffness $k=0$

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



SECTION A-A
7 ϕ 20 + 3 ϕ 20
SECTION B-B
2 ϕ 20 + 5 ϕ 20

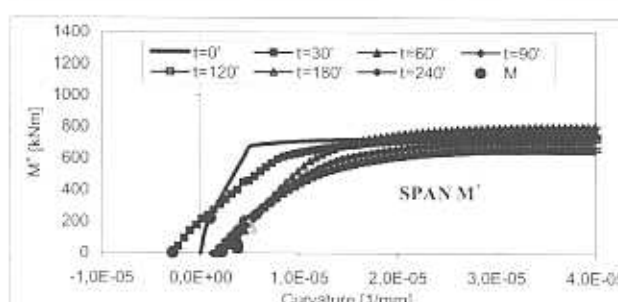
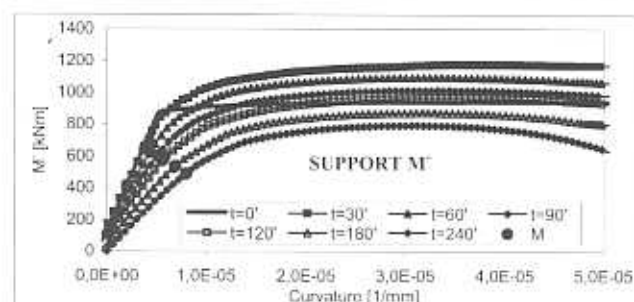
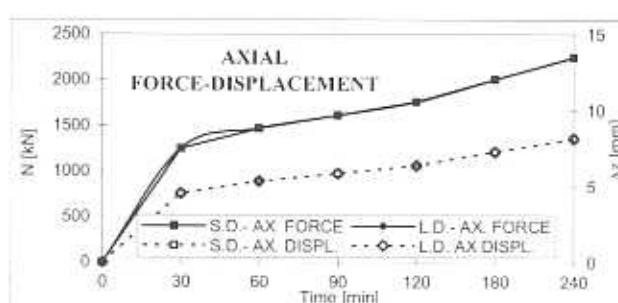
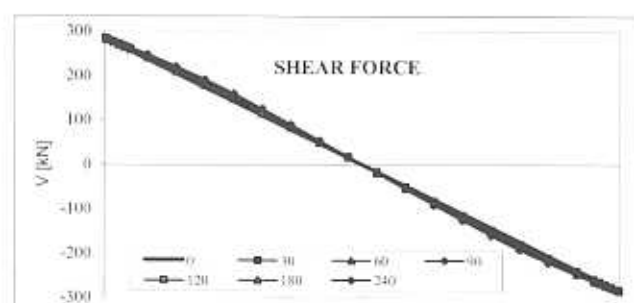
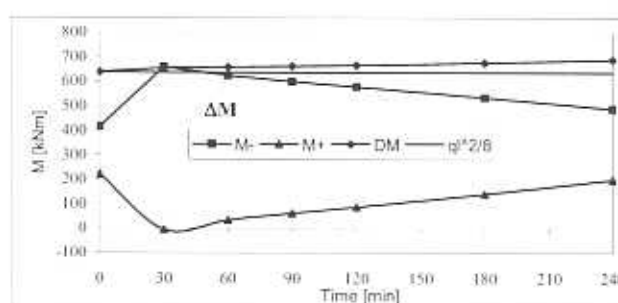
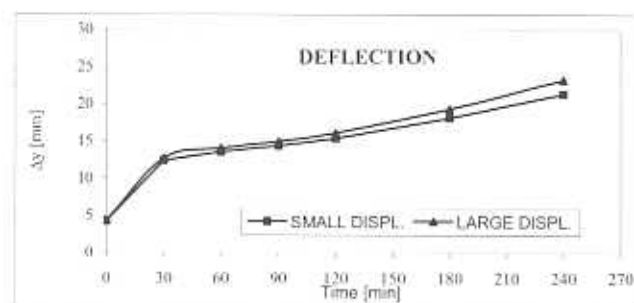
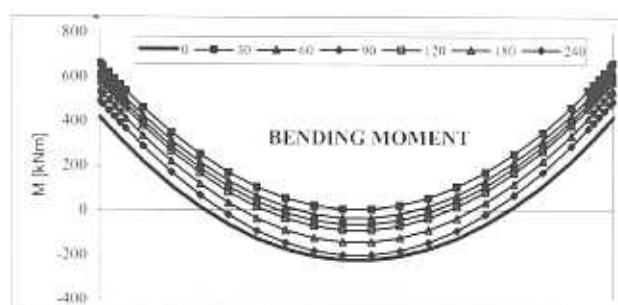
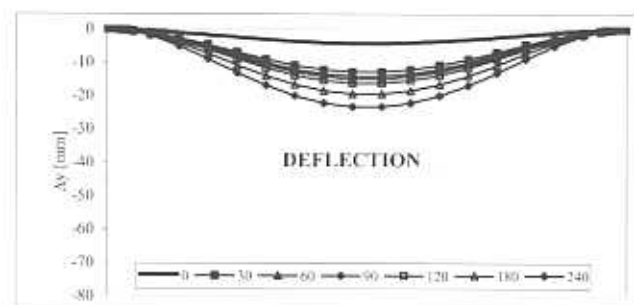


Fig. A4-35: Behaviour of a 9m span rectangular beam with axial restraint of stiffness $k = EA/3L$.

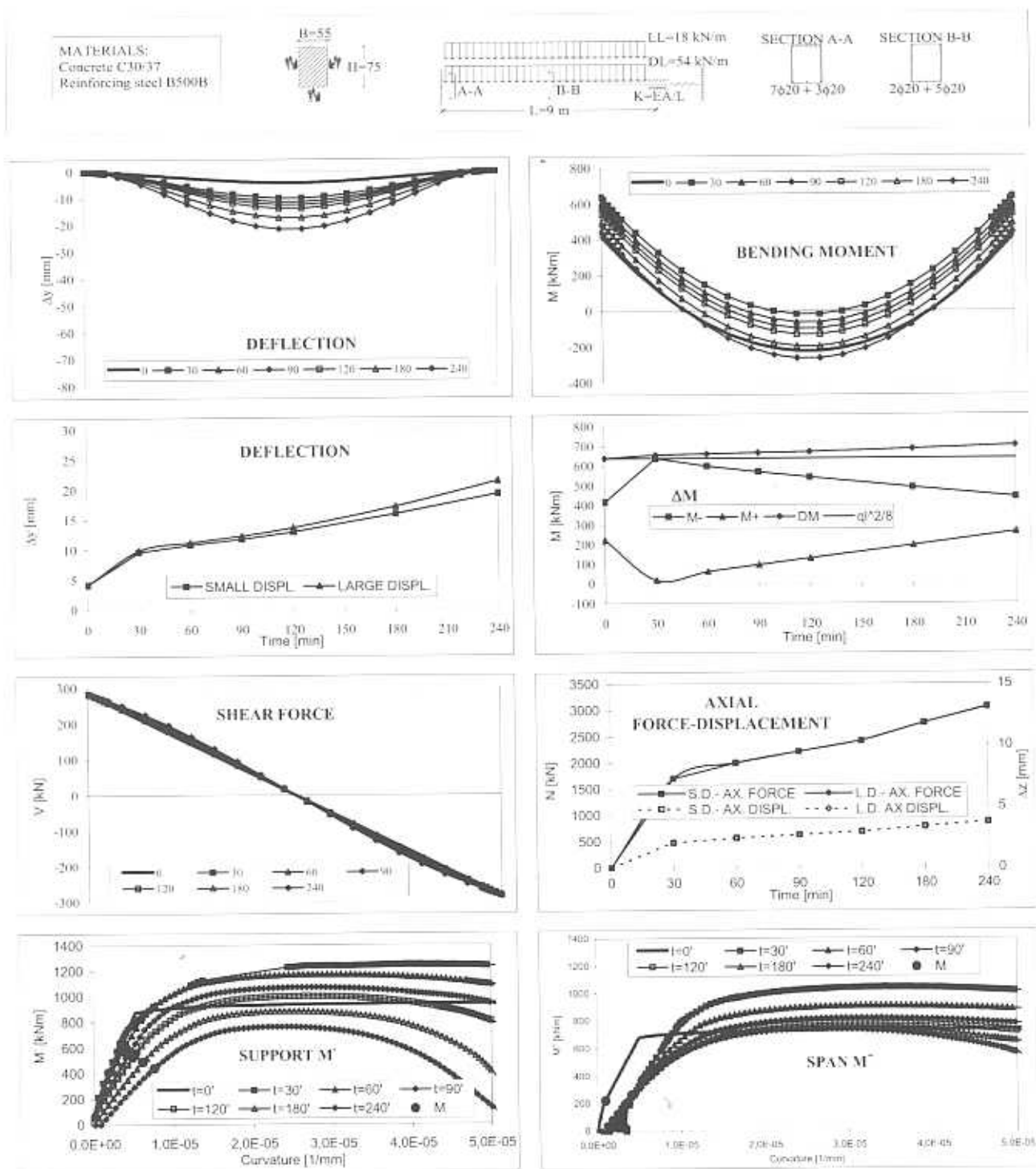
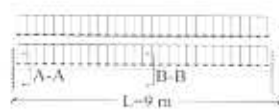


Fig. A4-36: Behaviour of a 9m span rectangular beam with axial restraint of stiffness $k=EA/L$

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



LL=18 kN/m
DL=54 kN/m

SECTION A-A
7 ϕ 20 + 3 ϕ 20

SECTION B-B
2 ϕ 20 + 5 ϕ 20

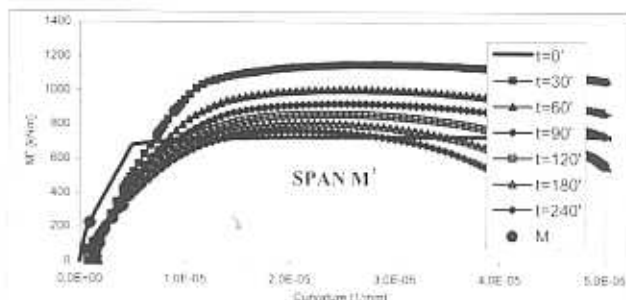
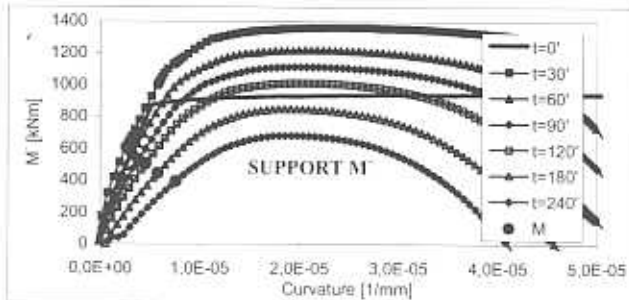
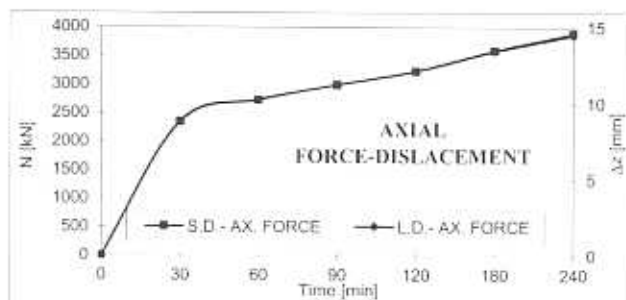
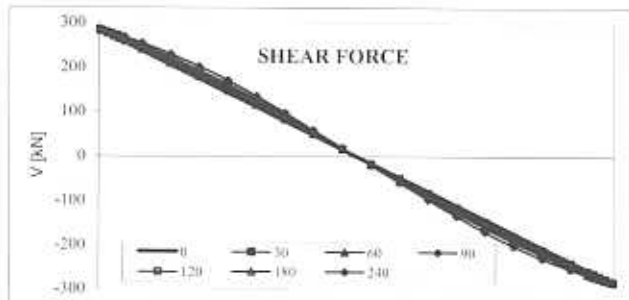
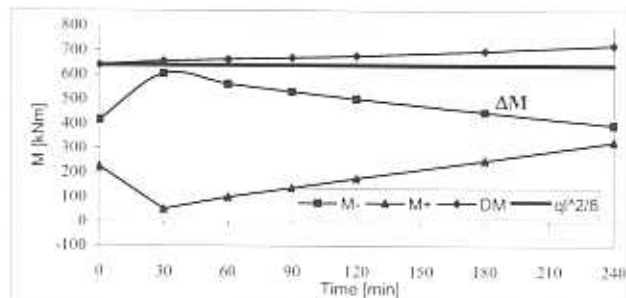
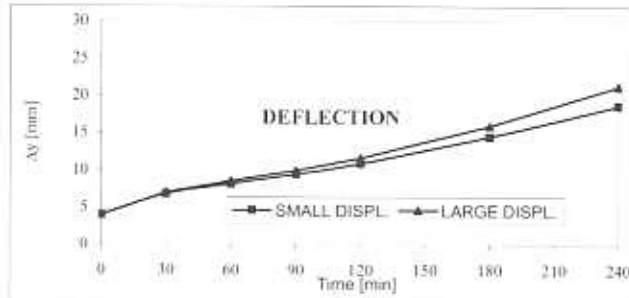
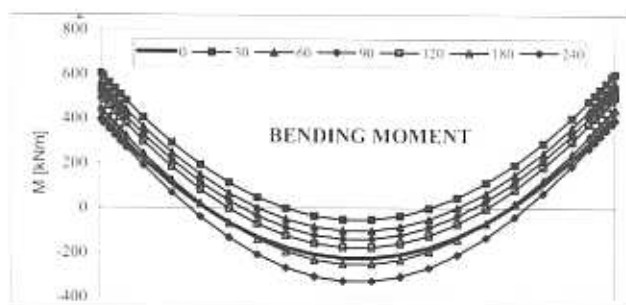
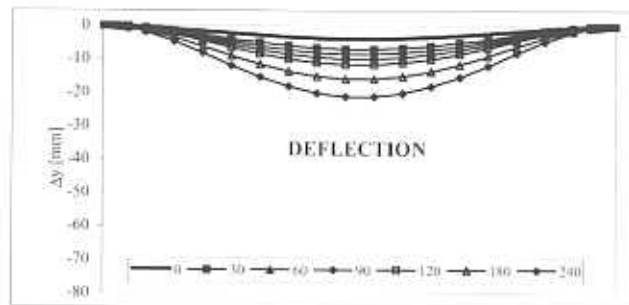
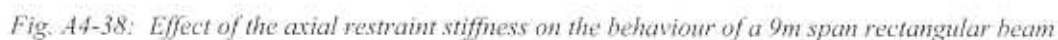


Fig. A4-37: Behaviour of a 9m span rectangular beam with axial restraint of stiffness $k = \infty$



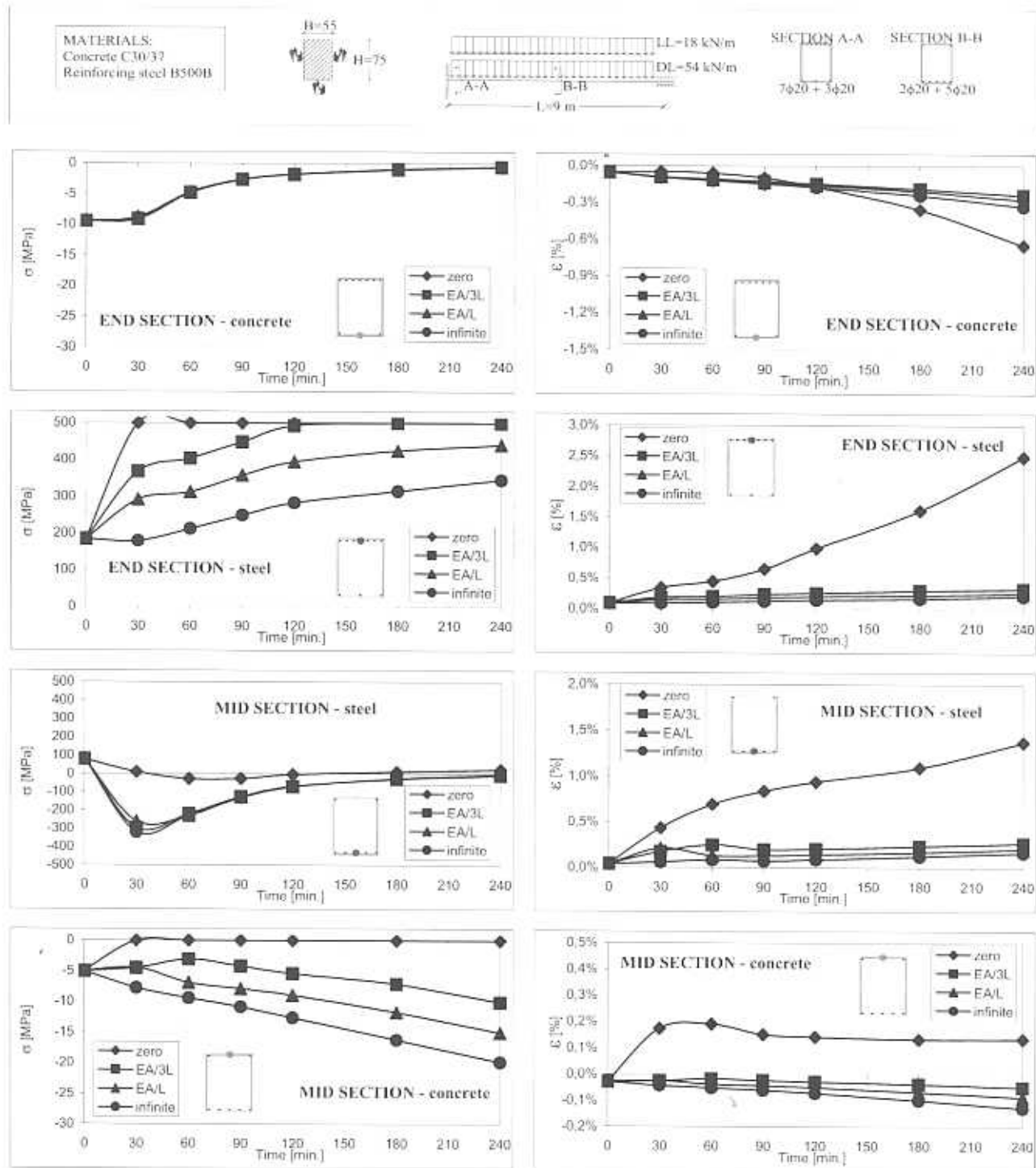


Fig. A4-39: Effect of the axial restraint stiffness on the stress and strain time-histories for a 9m rectangular beam

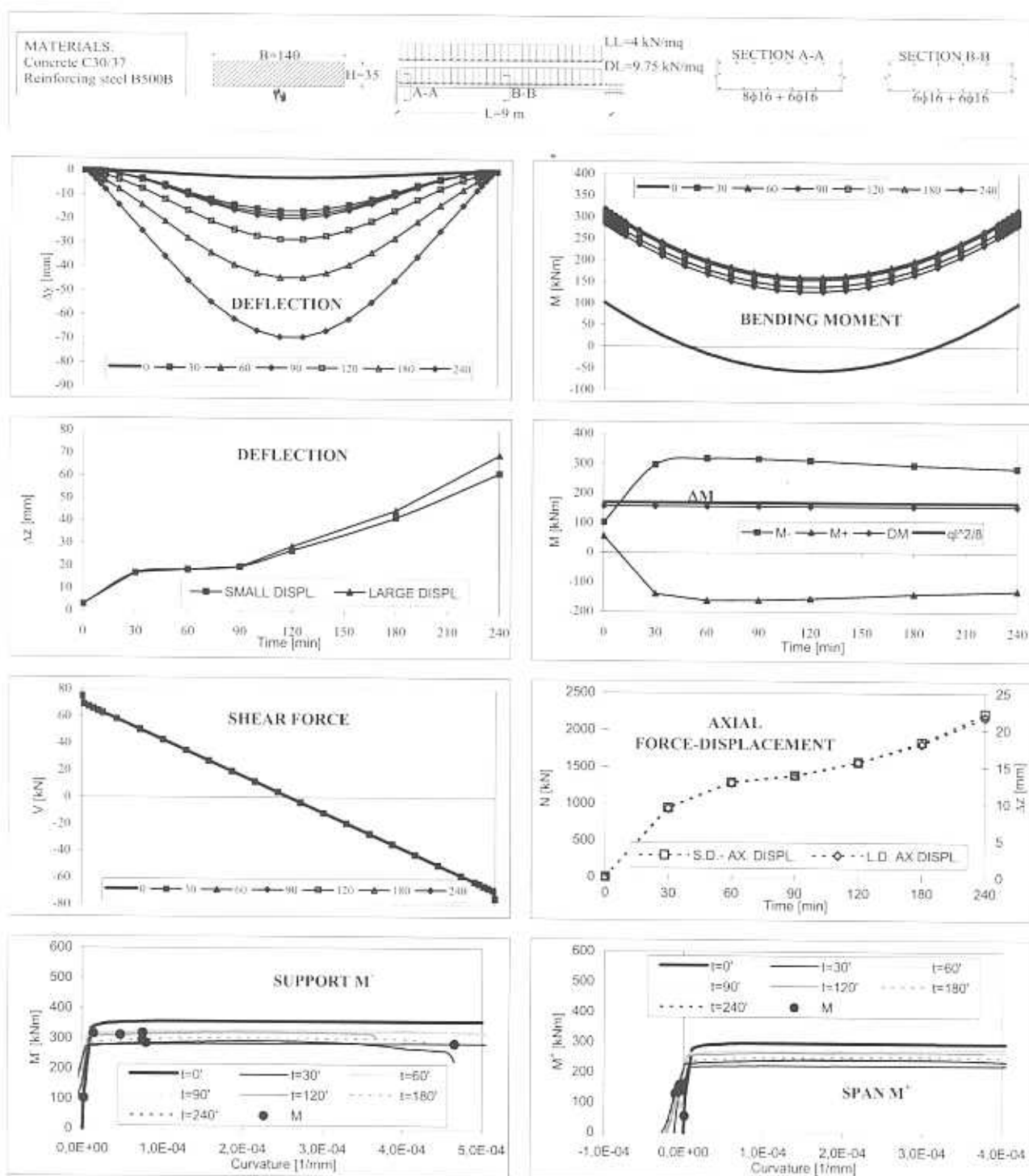


Fig. A4-40: Behaviour of a 9m span one-way slab with axial restraint of stiffness $k=0$

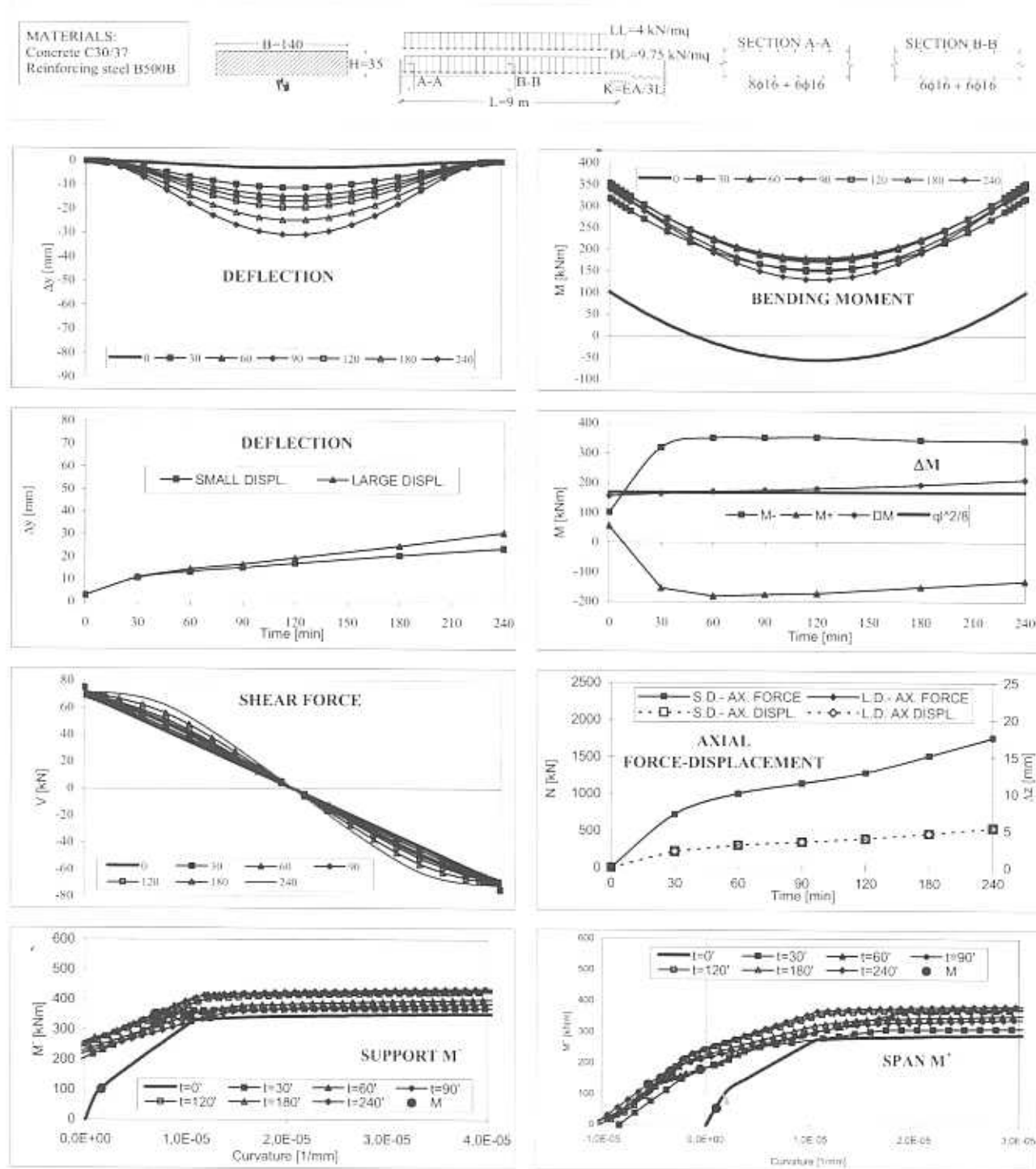
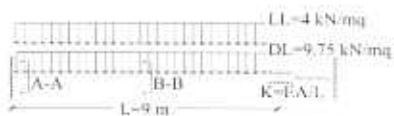


Fig. A4-41: Behaviour of a 9m span one-way slab with axial restraint of stiffness $k= EA/3L$.

MATERIALS-
Concrete C30/37
Reinforcing steel B500B



SECTION A-A

8 ϕ 16 + 6 ϕ 16

SECTION B-B

6 ϕ 16 + 6 ϕ 16

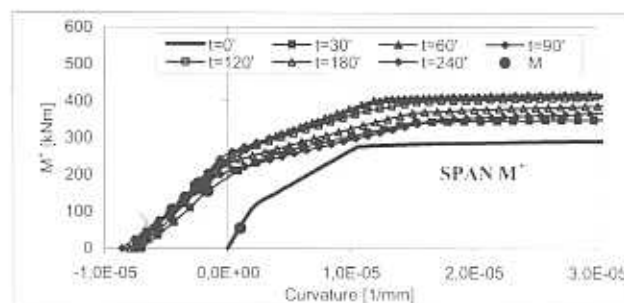
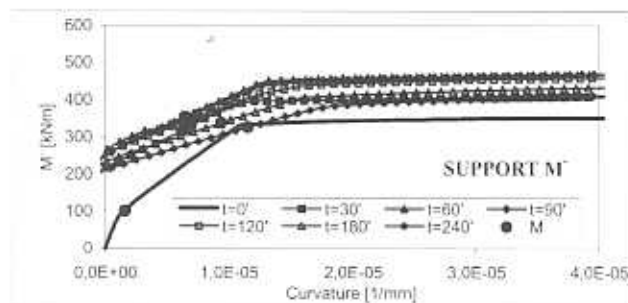
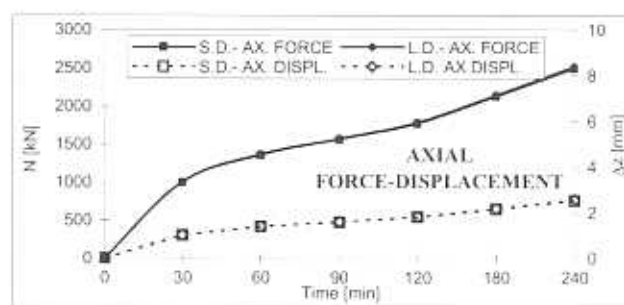
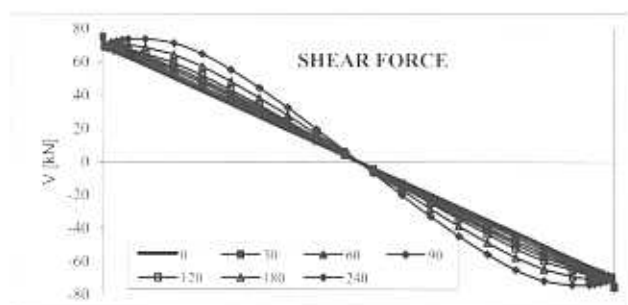
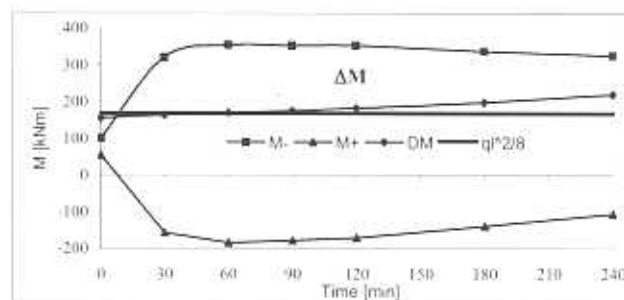
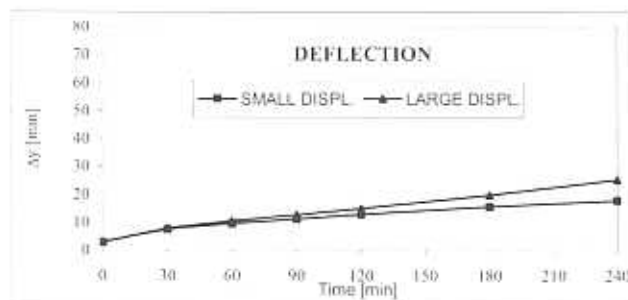
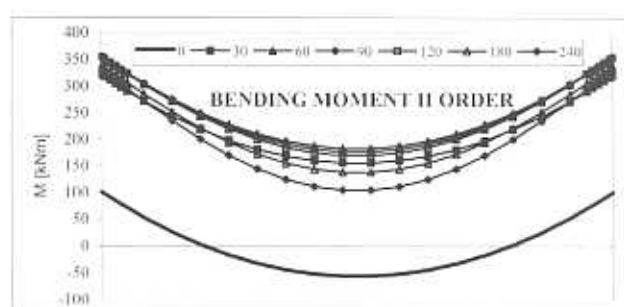
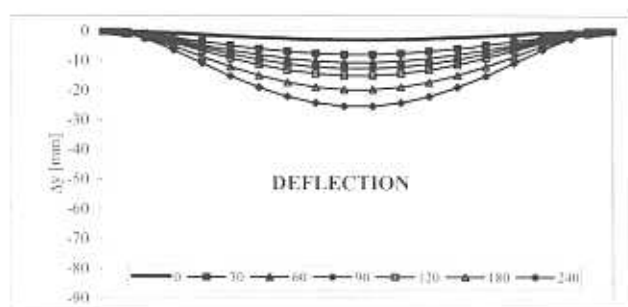


Fig. A4-42: Behaviour of a 9m span one-way slab with axial restraint of stiffness $k = EA/L$

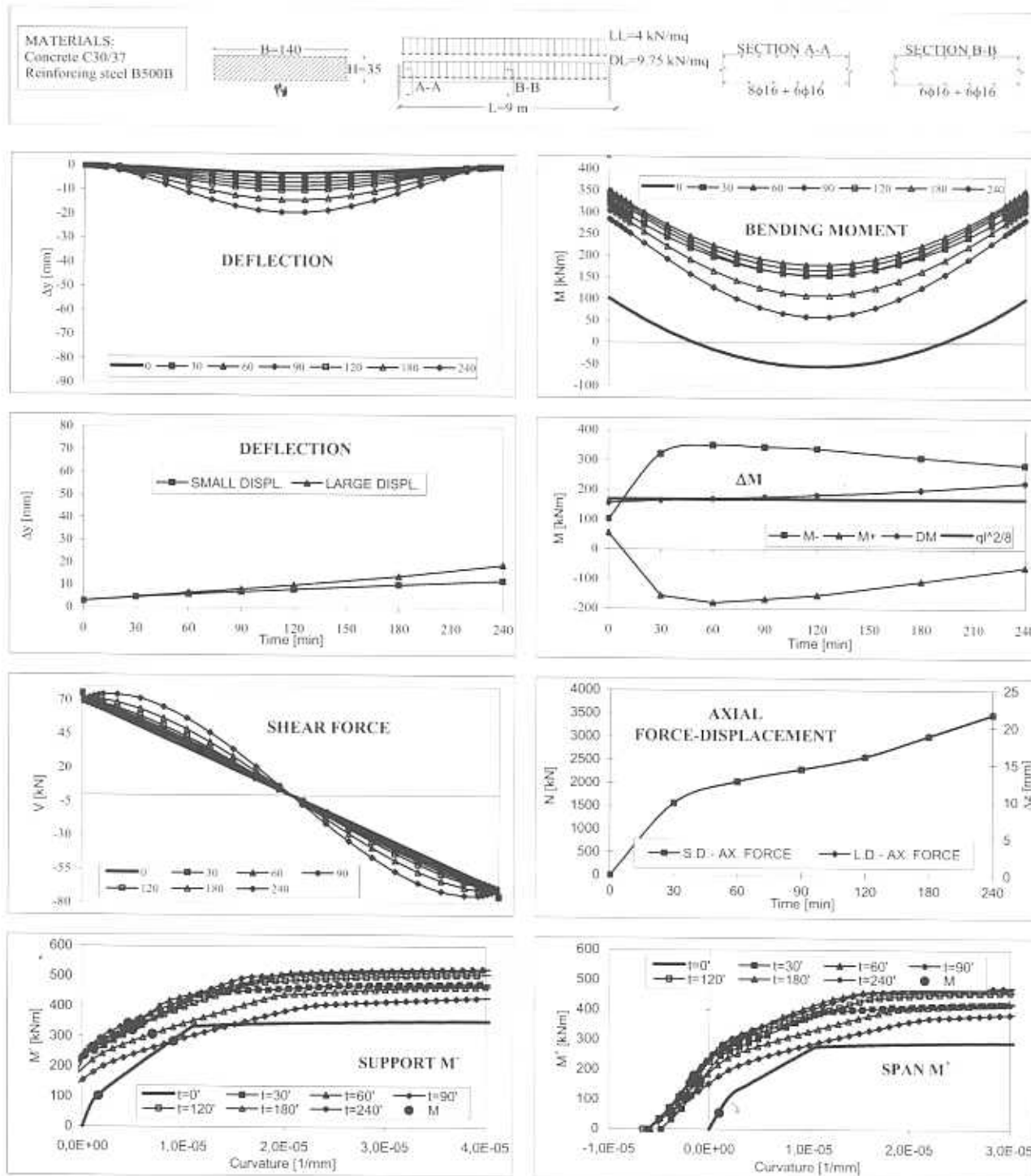


Fig. A4-43: Behaviour of a 9m span one-way slab with axial restraint of stiffness $k = \infty$

MATERIALS:
Concrete C 30/37
Reinforcing steel B500B

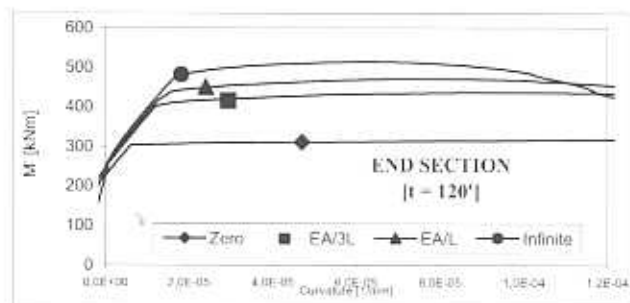
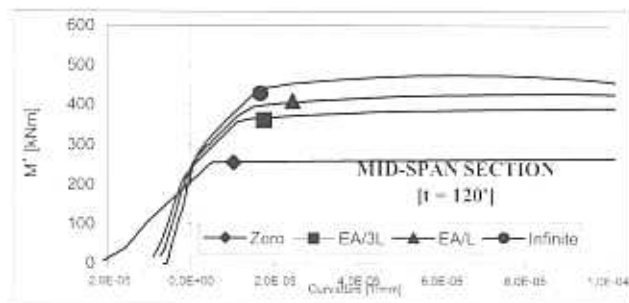
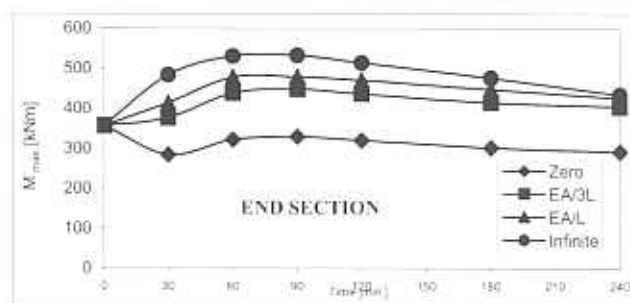
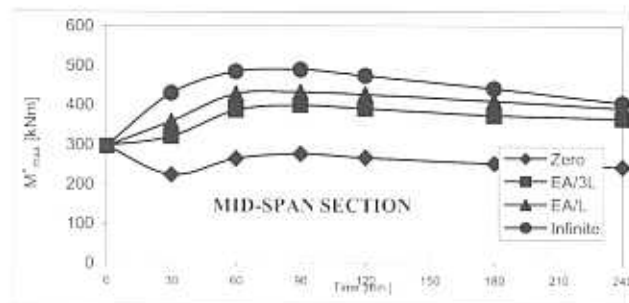
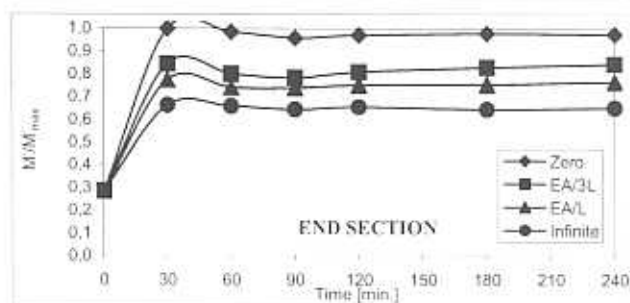
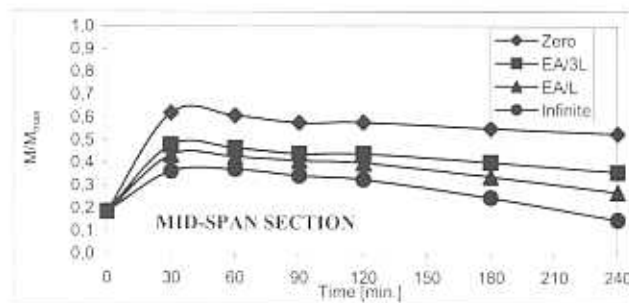
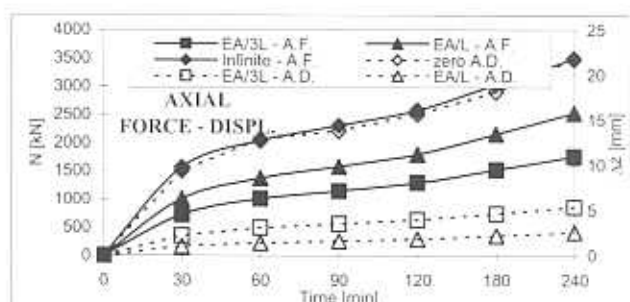
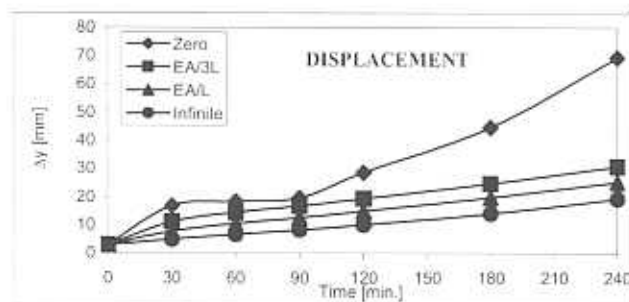
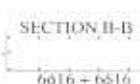
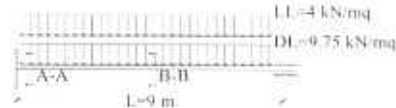
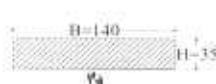


Fig. A4-44: Effect of the axial restraint stiffness on the behaviour of a 9m span one-way slab

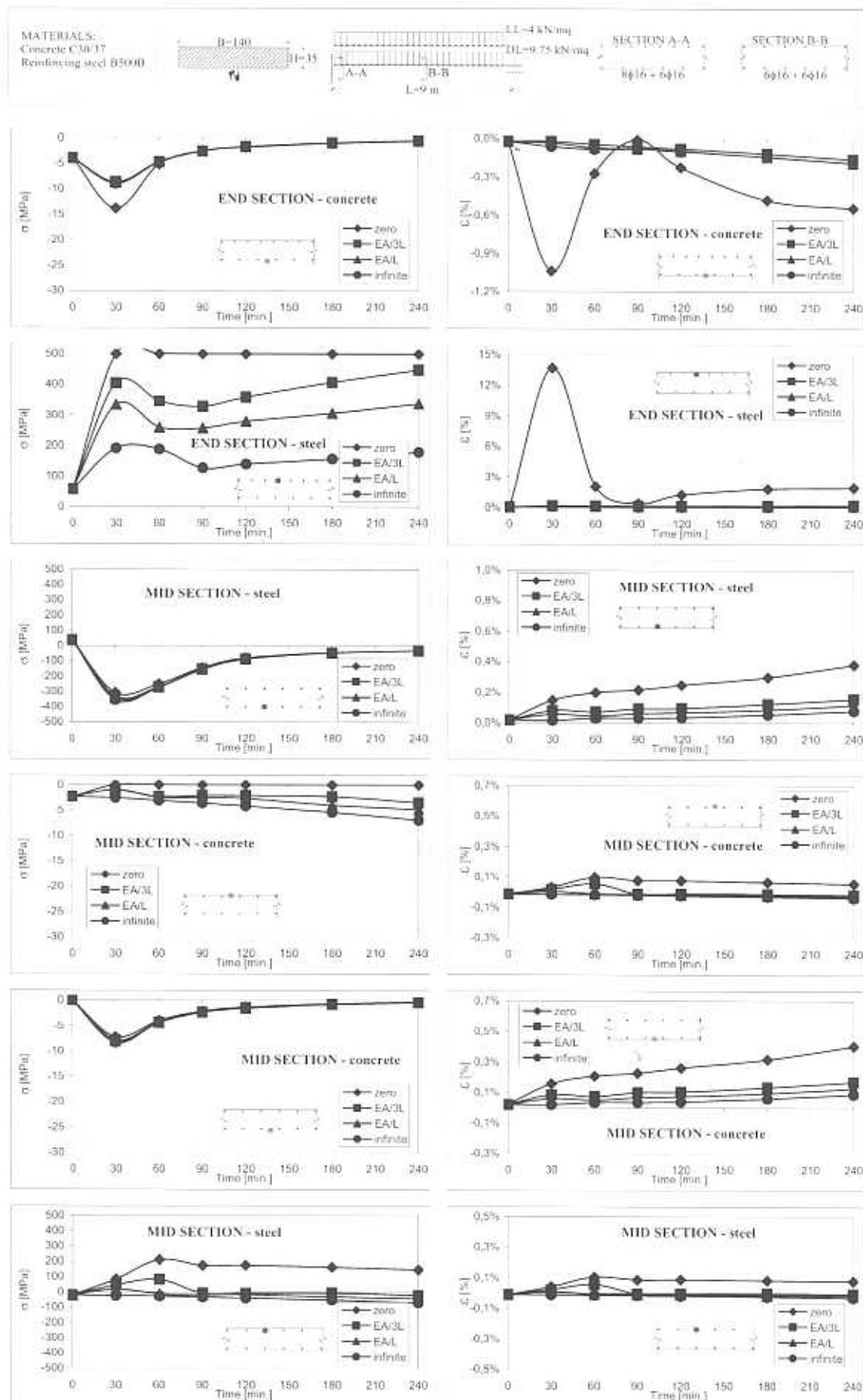


Fig. A4-45: Effect of the axial restraint stiffness on the stress and strain time-histories for a 9m one-way slab

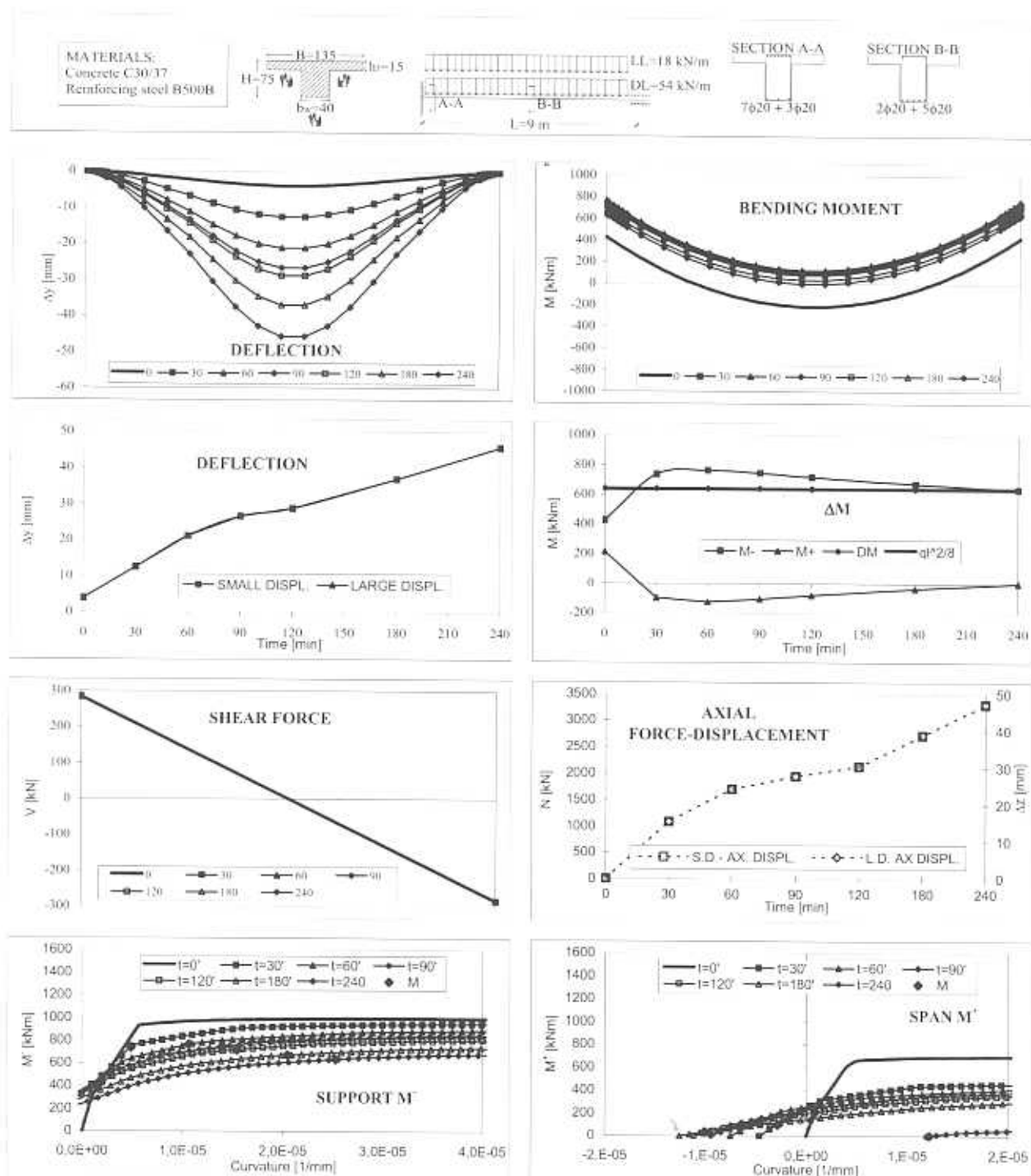


Fig. A4-46: Behaviour of a 9m span T-beam with axial restraint of stiffness $k=0$

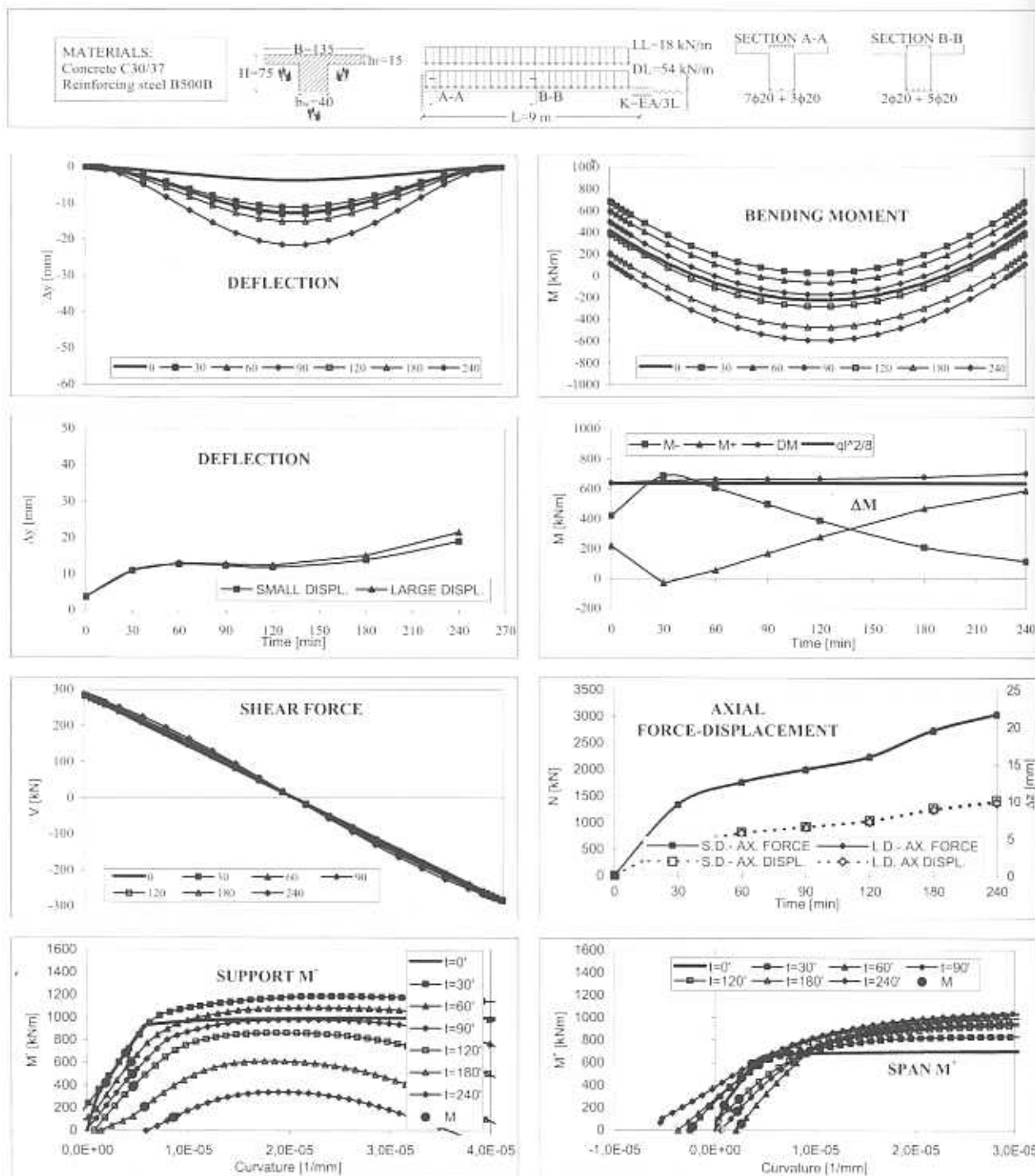


Fig. A4-47: Behaviour of a 9m span T-beam with axial restraint of stiffness $k=EA/3L$

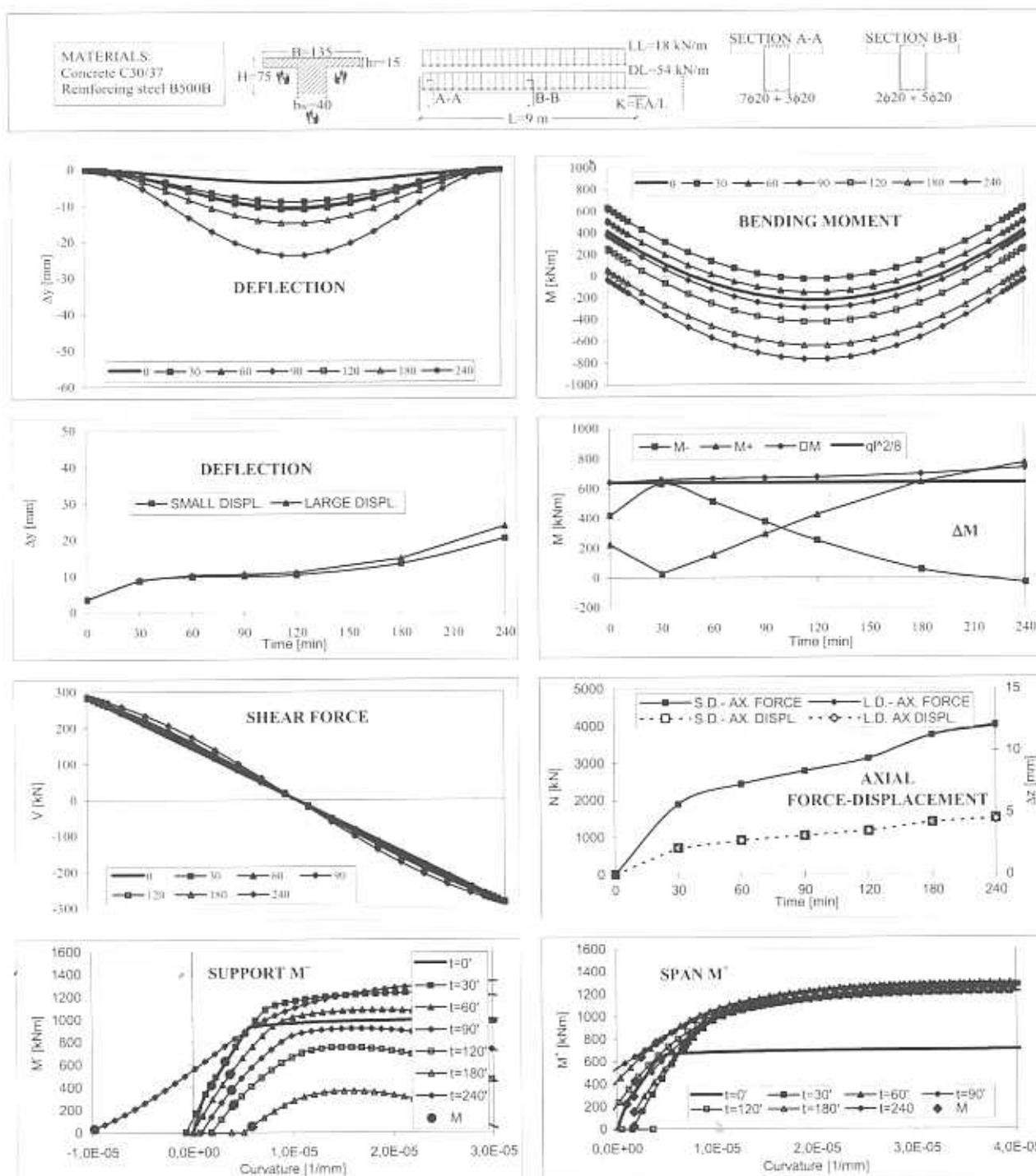


Fig. A4-48: Behaviour of a 9m span T-beam with axial restraint of stiffness $k=EA/L$

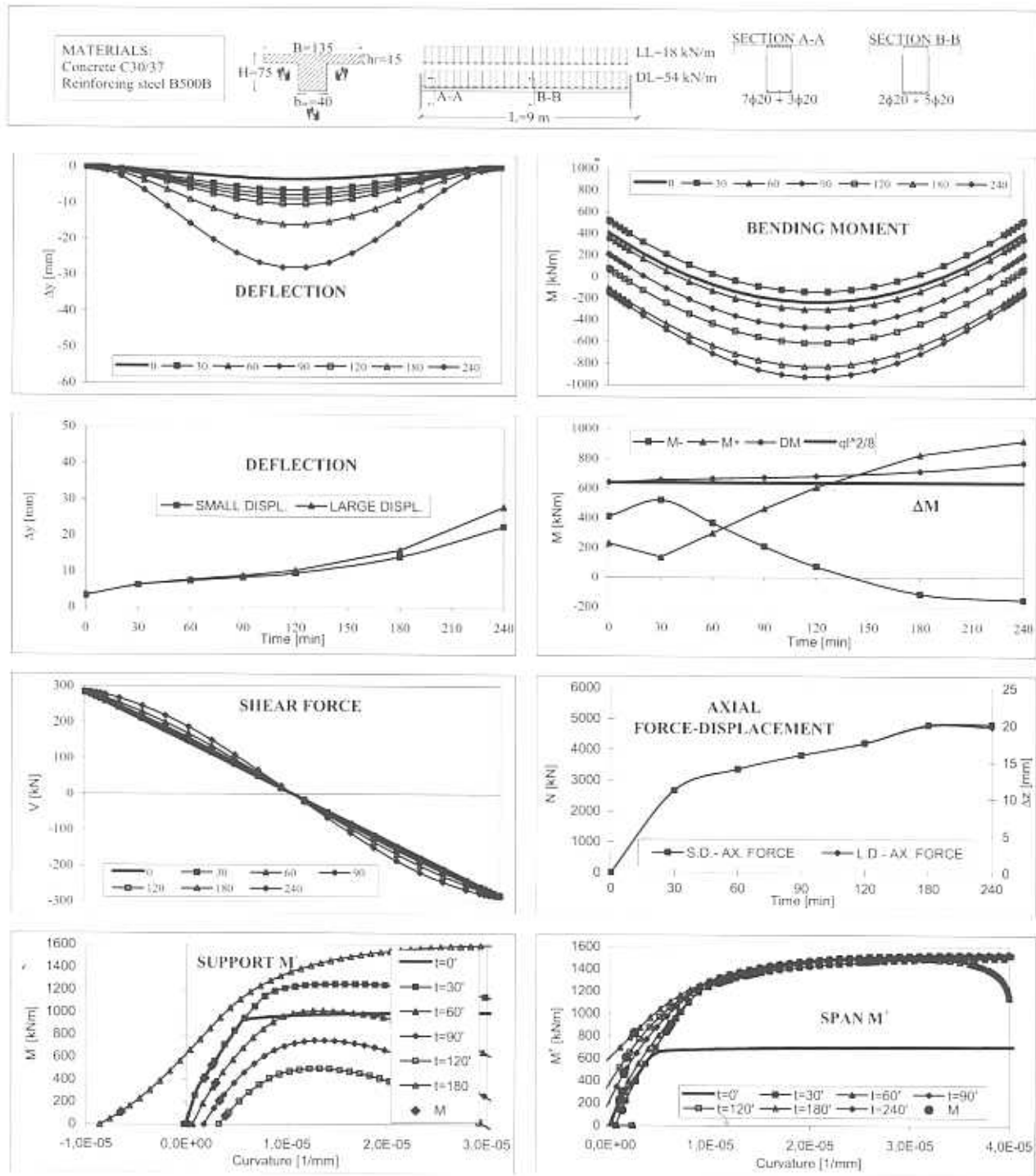
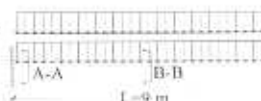


Fig. A4-49: Behaviour of a 9m span T-beam with axial restraint of stiffness $k = \infty$

MATERIALS:
Concrete C30/37
Reinforcing steel B500B



LL=18 kN/m
DL=54 kN/m

SECTION A-A
7φ20 + 3φ20

SECTION B-B
2φ20 + 5φ20

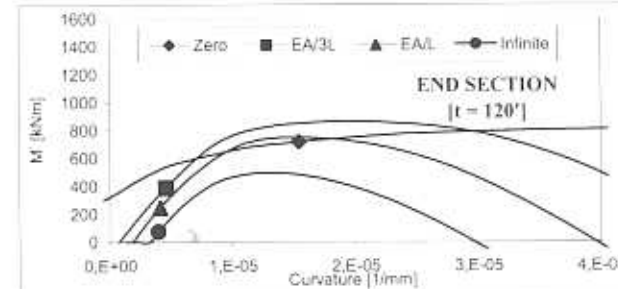
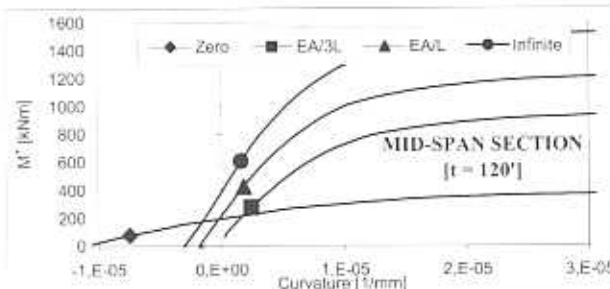
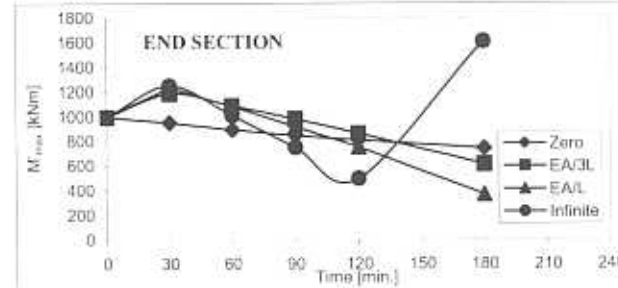
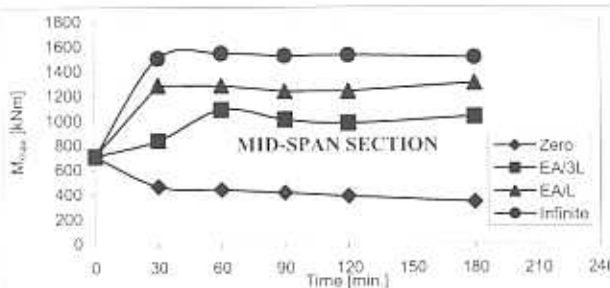
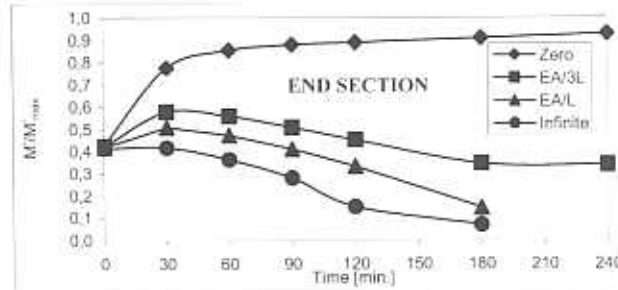
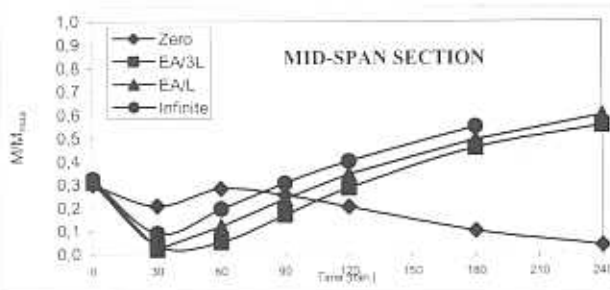
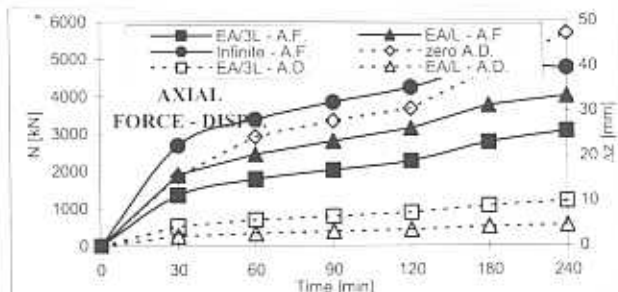
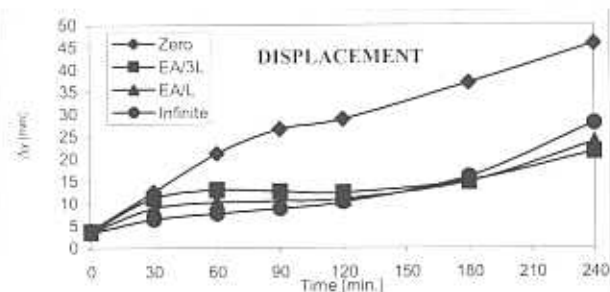


Fig. A4-50: Effect of the axial restraint stiffness on the behaviour of a 9m span T-beam

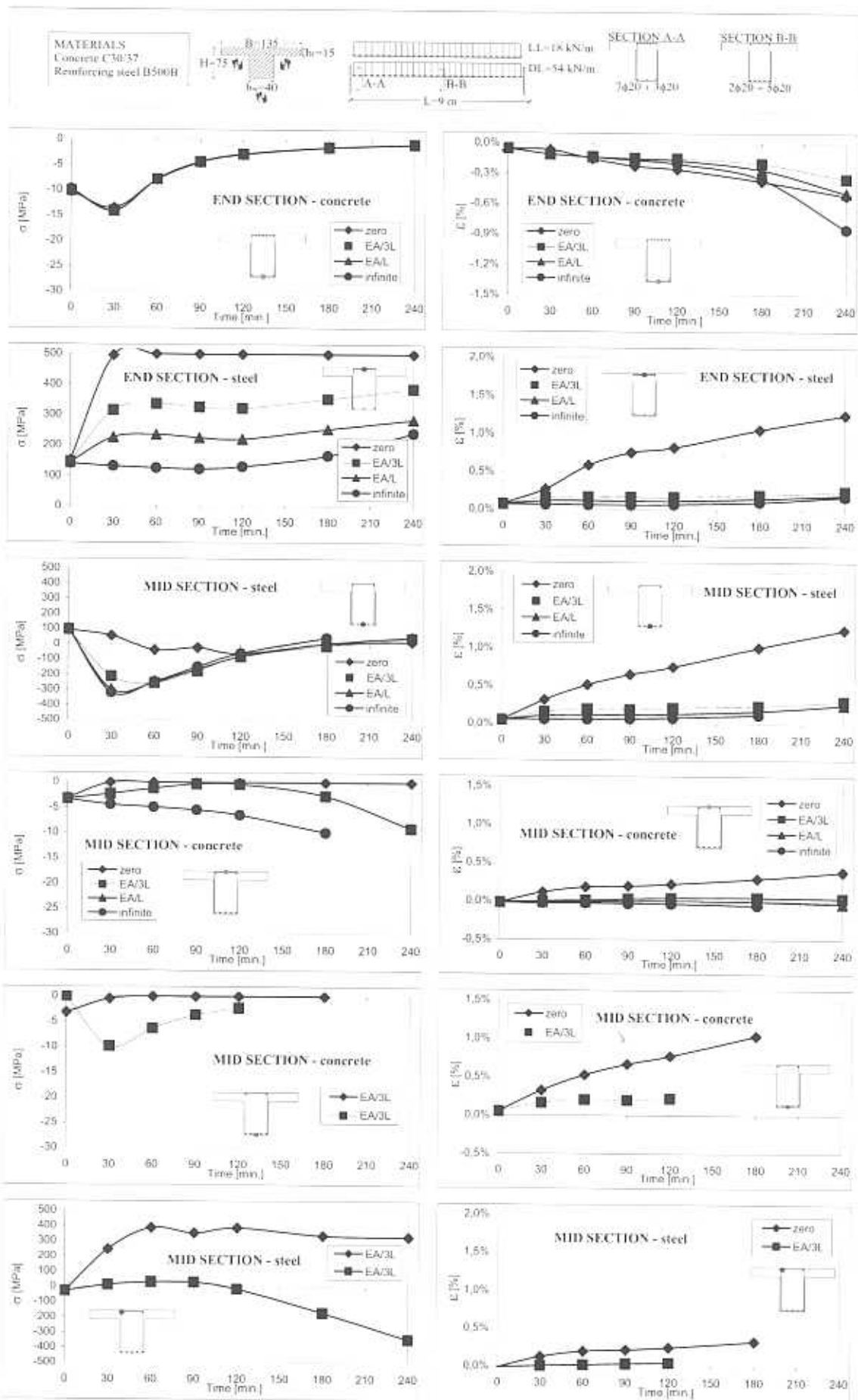
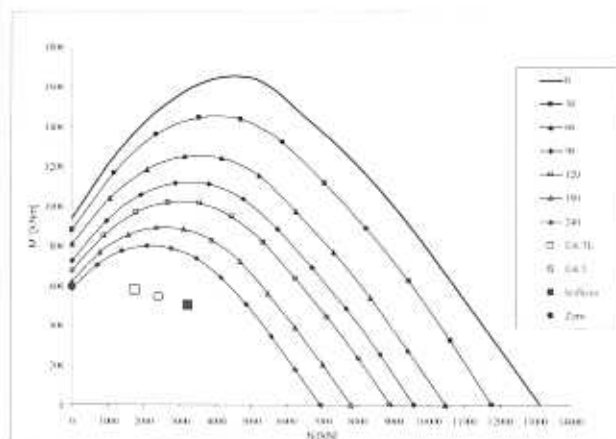
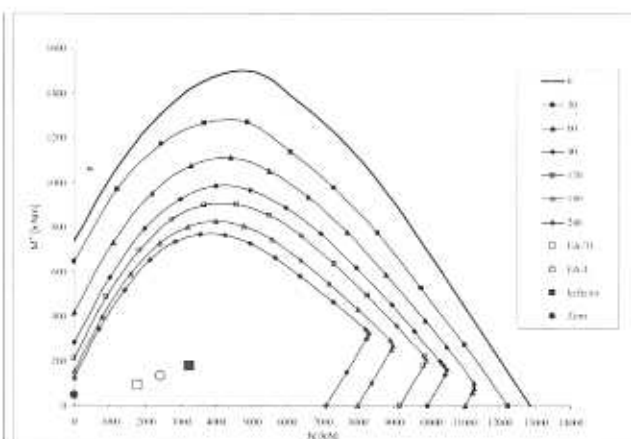


Fig. A4-51: Effect of the axial restraint stiffness on the stress and strain time-histories for a 9m T-beam

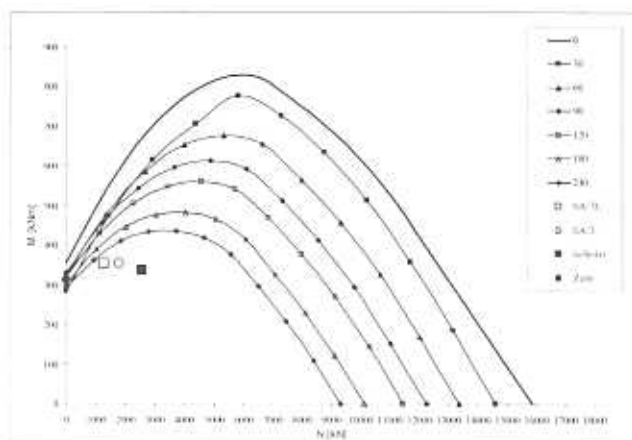


(a) Support section – Negative moment

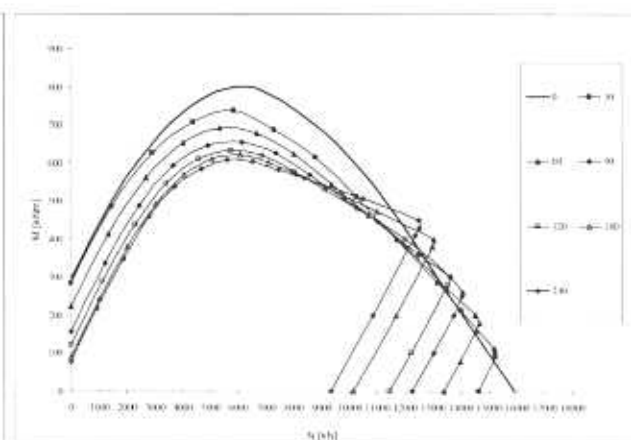


(b) Mid-span section – Positive moment

Fig. A4-52: Bending moment – axial force interaction curves; 9m span rectangular beam

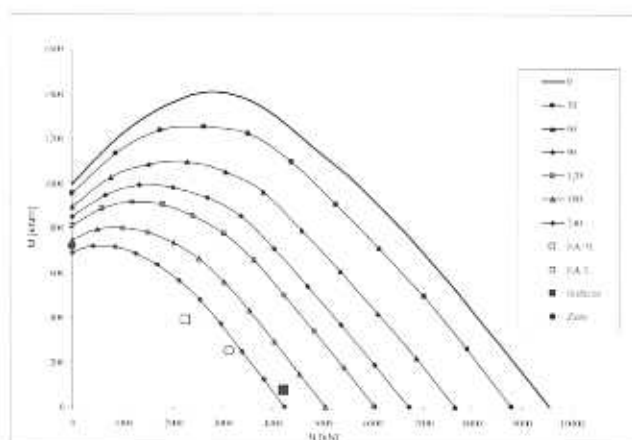


(a) Support section – Negative moment

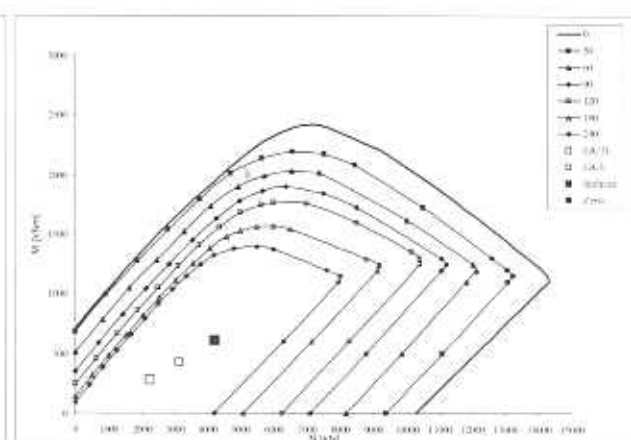


(b) Mid-span section – Positive moment

Fig. A4-53: Bending moment – axial force interaction curves; 9m span one-way slab



(a) Support section – Negative moment



(b) Mid-span section – Positive moment

Fig. A4-54: Bending moment – axial force interaction curves; 9m span T-beam

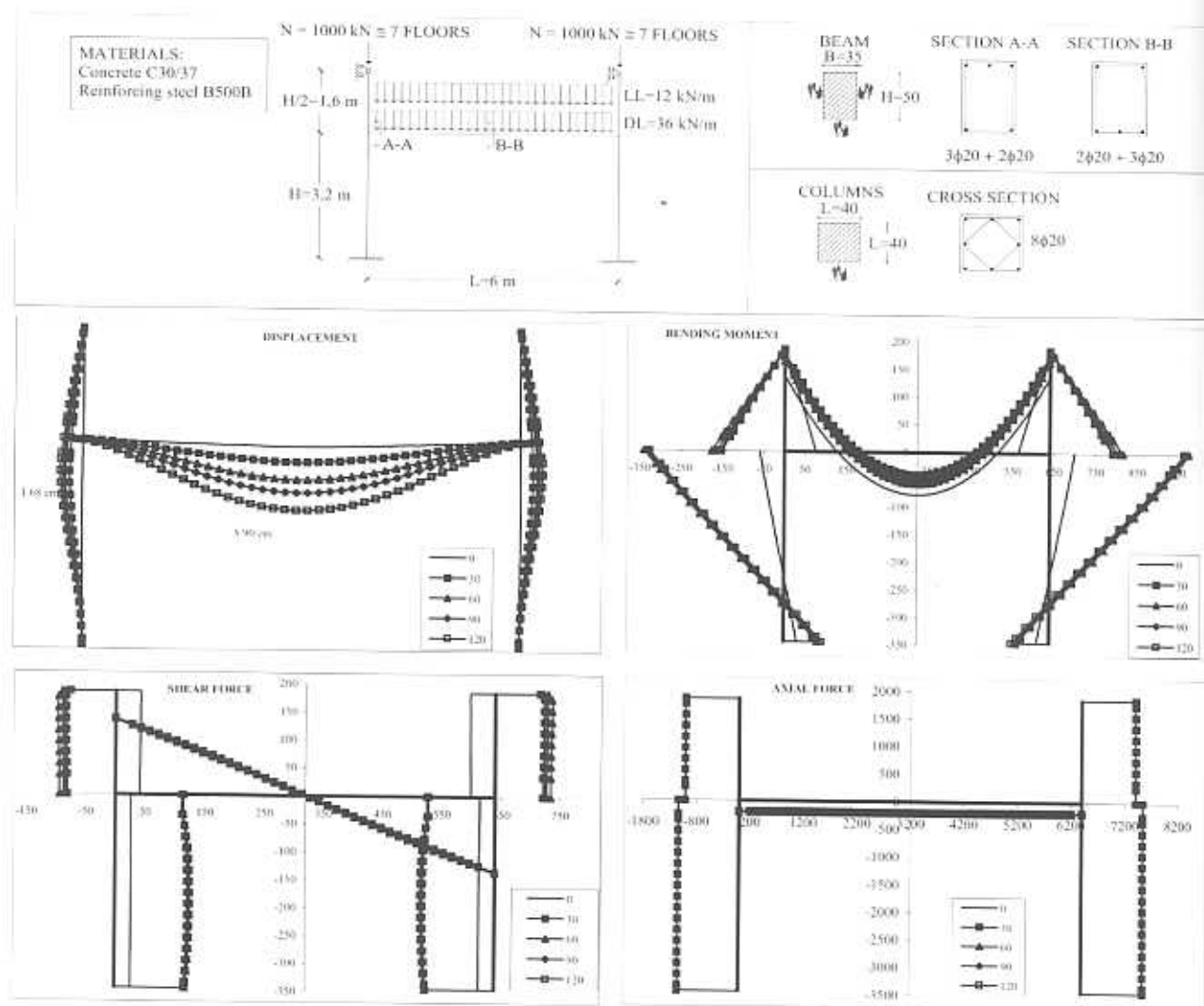


Fig. A4-55: R/C frame with columns exposed to the fire on one side and with a 6m-span beam (rectangular section)

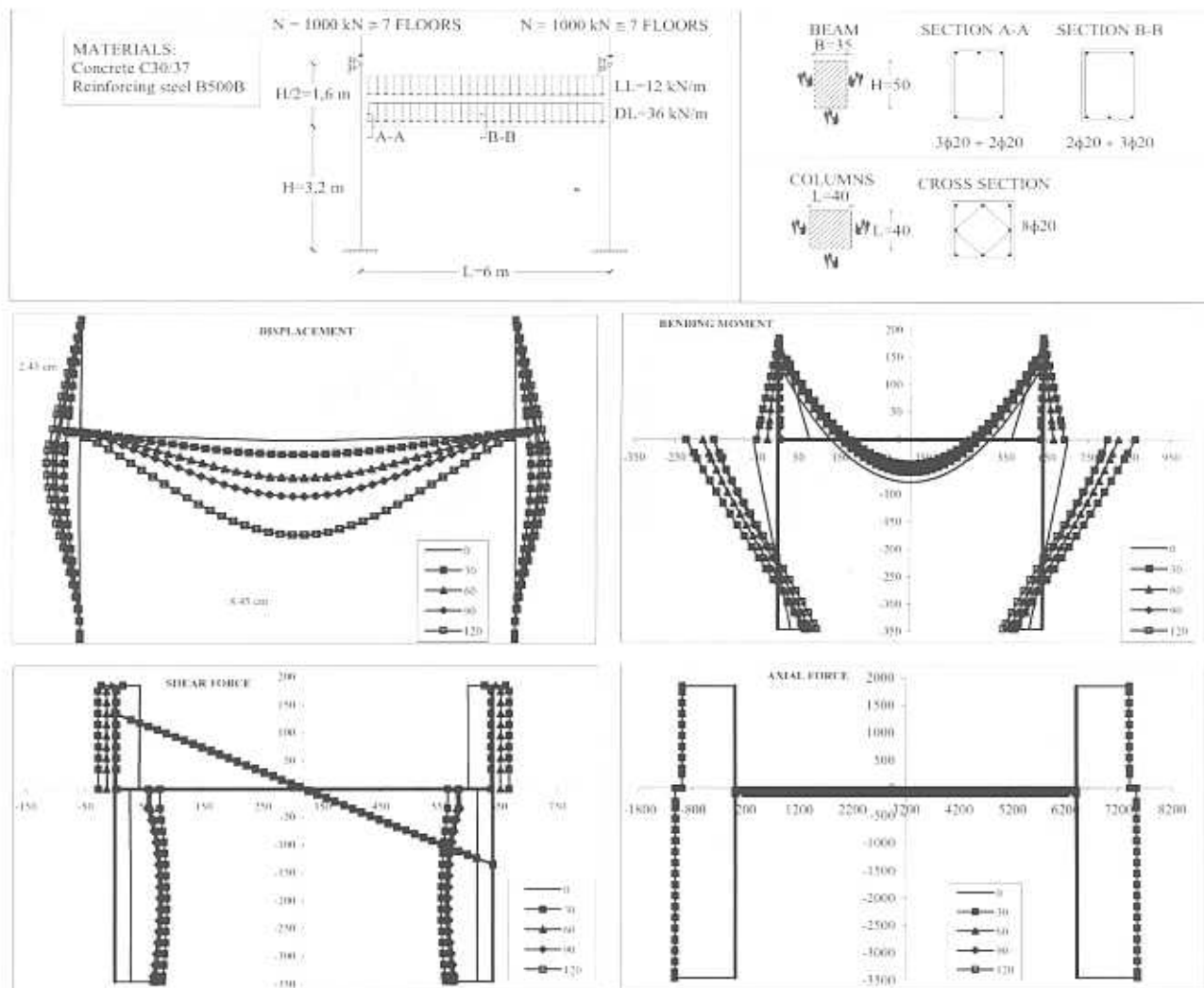


Fig. A4-56: R/C frame with columns exposed to the fire on three sides and with a 6m-span beam (rectangular section)

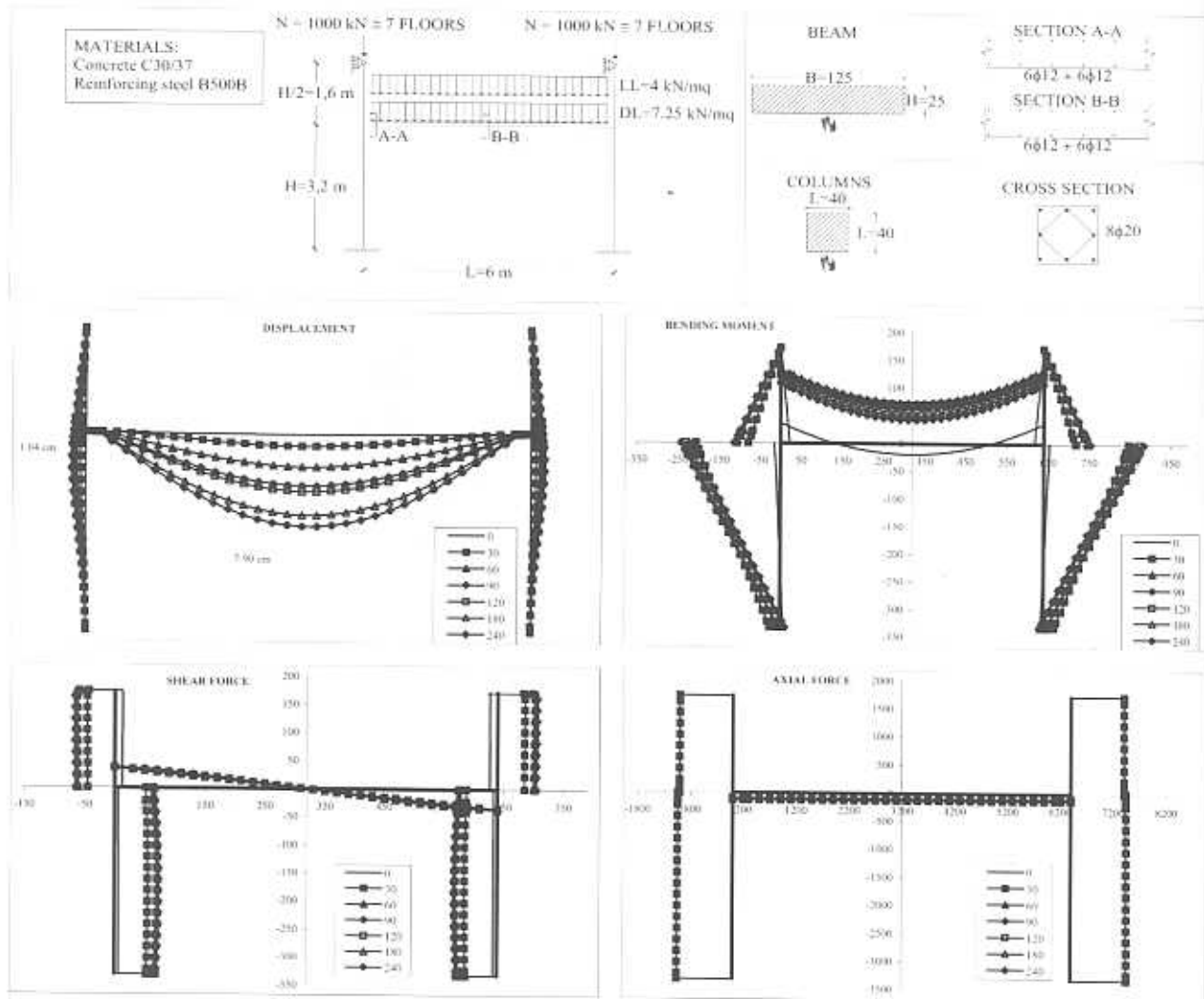


Fig. A4-57: R/C frame with columns exposed to the fire on one side and with a 6m-span beam (one-way slab)

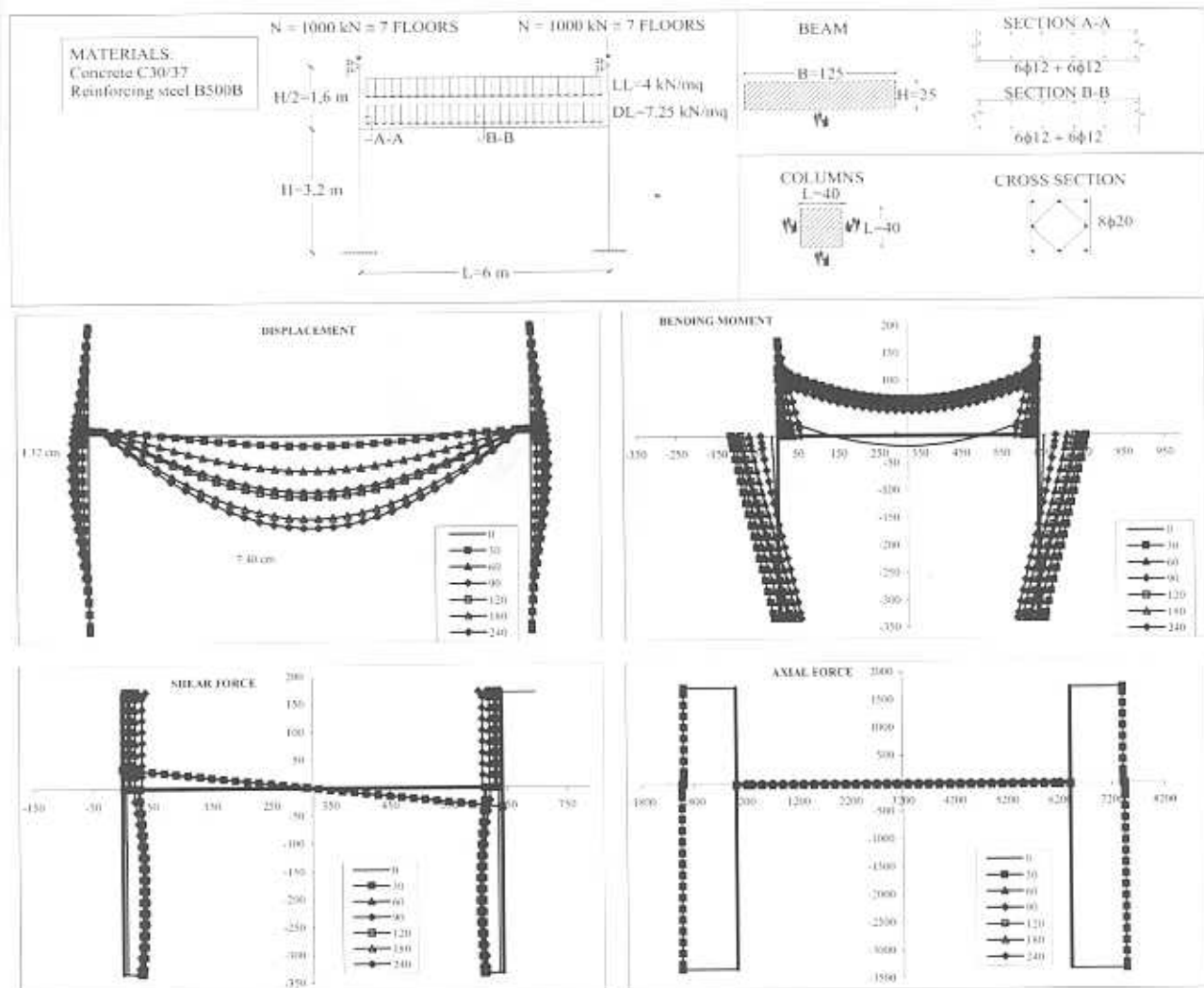


Fig. A4-58: R/C frame with columns exposed to the fire on three sides and with a 6m-span beam (one-way slab)

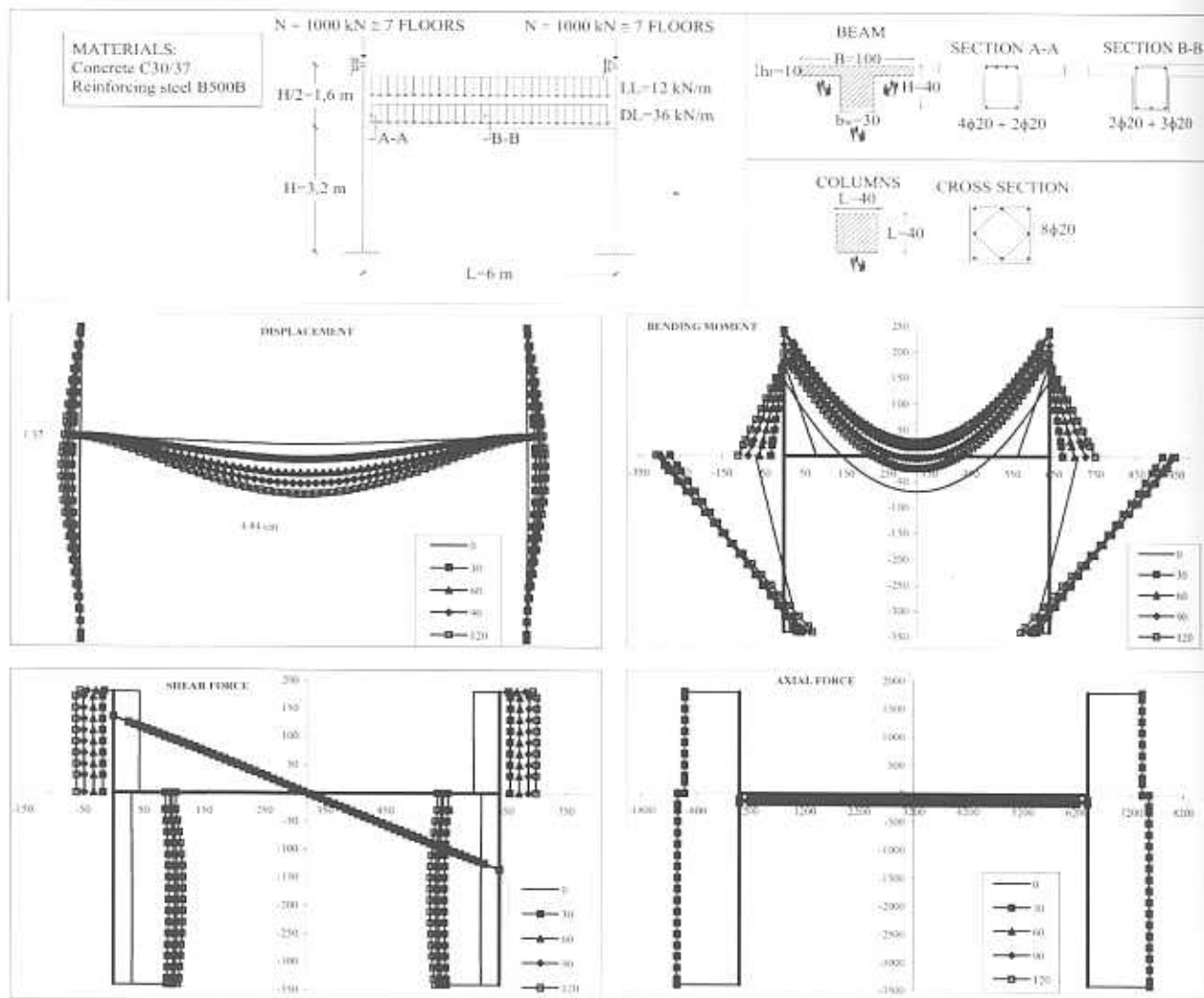


Fig. A4-59: R/C frame with columns exposed to the fire on one side and with a 6m-span beam (T-section)

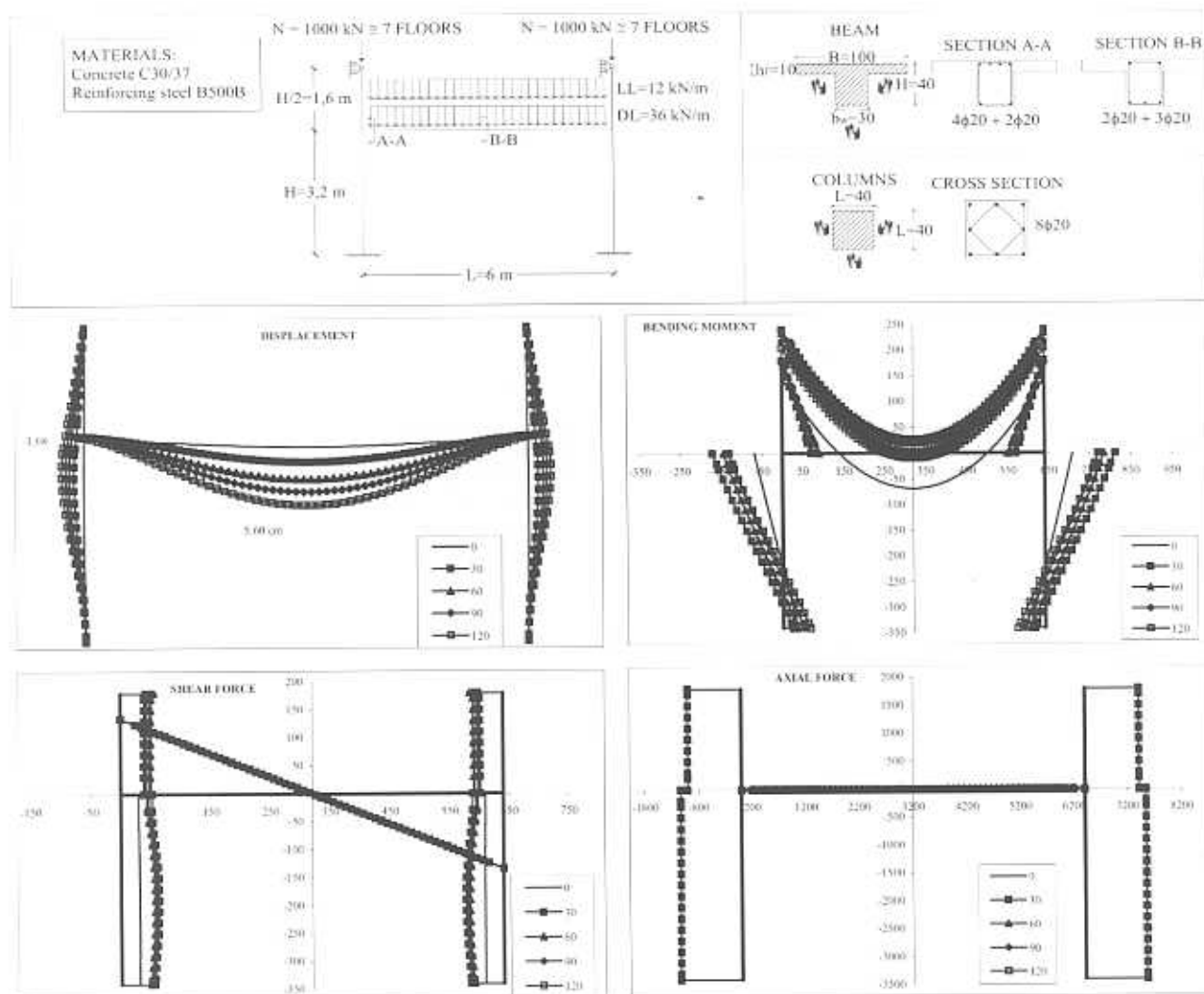


Fig. A4-60: R/C frame with columns exposed to the fire on three sides and with a 6m-span beam (T-section)

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Fire design of concrete structures – structural behaviour and assessment

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