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RECENT DEVELOPMENTS IN TECHNIQUES FOR PROTECTING STEEL FROM FIRE

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BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

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SYNOPSIS

The paper reviews progress in research and testing of fire engineering methods for the protection of steel structures against fire. Designers are now presented with options to the traditional dependence on regulatory requirements and standard fire tests. The available methods are outlined and their relative merits discussed.

Increasing knowledge of structural behaviour in fire conditions has led to a number of new practical solutions to steel protection which offer a more engineered approach to the total use of structural elements in maintaining stability of steel structures in fire and lead to a reduction in protection requirements. Use of unprotected steelwork in situations of low fire severity leads to further cost savings.

INTRODUCTION

New Zealand Standard Model Building Bylaw NZS 1900 Chapter 5 Fire Resisting Construction and Means of Egress (NZS 1900,5) contains requirements for fire resistance based on standard time periods ($\frac{1}{2}$, 1, $1\frac{1}{2}$ etc hours). This code requires proof that structural elements comply by means of satisfactory performance in a standard fire resistance test. No other solution is permitted even in the latest (1984) amendment to NZS 1900, 5. Notwithstanding, alternative solutions have been proposed for individual building projects, and some have been accepted by approving authorities. With little knowledge of the validity of these alternatives, and no formal mechanism available for considering them, approving authorities can be understandably reluctant to permit their use. This problem is not unique to New Zealand, nor to steel construction only.

Since NZS 1900,5 was first published in 1963, knowledge of the behaviour in fire of building materials and structures has progressed rapidly. There has been a massive amount of research, both by independent agencies and funded by industry, in many countries, resulting in a significant degree of confidence being placed in the accumulated knowledge of the factors that influence the behaviour of building fires, and of the response of building materials to those fires. Theory has shown that building fires depend on the nature of the contents (fire load), the availability of oxygen for combustion (ventilation), and the nature of the surroundings (thermal response). Further it was found that the properties of building materials, which are constant at ambient temperatures, are temperature dependent. Calculations based on this knowledge became possible with the advent of computers, and programs are available for conducting fire engineering calculations. Some of the reference texts contain tabulated information which reduce the calculation effort.

A bibliography on the protection of structural steel was published by Bastings (1984) and the topic was further covered by Bastings (1986). The present paper serves to add to and amplify those papers. The information has been supplemented by personal discussion between the author and a number of researchers and fire engineers in USA, Europe and Australia.

The aim of this paper is to provide designers and approving authorities with a guide to the various ways which may be used to ensure that structural steel does not collapse when exposed to a building fire. In order to avoid writing a treatise on fire engineering, no formulae or calculation methods are included. These can be found in the references, which are all available in New Zealand, and no doubt elsewhere, should a reader wish to become familiar with them.

The term designer used in this paper can have several meanings. It can apply to the architect, or to the structural engineer, and can include a relatively new name, that of the fire engineer. In many structures the traditional knowledge of the architect or the structural engineer may be adequate to arrive at fire protection specifications for the building. However fire protection and the provision of fire resistance is often left until the design is far down the track, whereas some knowledge of the methods described in this paper applied early in the design process could offer savings in fire protection costs and improved structural solutions. The fire engineer may be a separate individual, or the architect or structural engineer may acquire training to fit that role. Whichever, the designer will reap benefit for his client in understanding what methods may be available to him and being aware of their applications and limitations.

AVAILABLE DESIGN METHODS

A building designer has two choices of approach to the fire protection of the structure. He may opt to proceed along the traditional path of compliance with code or regulatory requirements or he may decide to adopt a fire engineering solution. Options under the fire engineering choice leave the designer with four possible methods. How these are interrelated is shown in Figure 1. The basic methodology is applicable to all structural materials, but this paper assumes the decision to use structural steel has already been made, and so Figure 1 and the subsequent text describes how the methods are applicable to the protection of steelwork. The references have also been chosen with an emphasis on steel design.

If it is decided that the regulatory approach is inappropriate for a particular building occupancy, the designer may consider embarking on a fire engineering design approach. In outline, this involves first, a knowledge of the characteristics of the development of a room fire; second, the ability to determine what temperatures will develop in the structure; and third, the ability to determine the response of the structure to those temperatures. The first choice is whether to use a real fire or the standard fire as the basis for the fire temperature conditions.

The real fire method predicts the heating rate and the maximum temperature of the gas in the fire room. The severity of the fire is first established from the fire load which is a measure of the amount of combustible material in the room. Fire load is often spoken of in terms of the equivalent quantity of wood (expressed in kg/m^2) but in the calculation methods referenced in this report the energy term megajoules per sq metre (MJ/m^2) is used. Information on fire loads is scarce for New Zealand buildings and reliance has to be placed on overseas sources e.g. Pettersson, Magnusson and Thor (1976), and Association of Cantonal Institutions for Fire Insurance (1973).

The thermal characteristics of the materials making up the walls, floor and ceiling of the fire room need to be taken into account. For instance, an insulated wall will conduct away heat slower than an uninsulated one, with the result that the fire will become hotter in an insulated room.

The third factor influencing real fire growth is the amount of ventilation received from the periphery of the room. This is provided by doors, which may be open, and by windows which can be expected to break due to thermal stresses early in the fire. In general, a high degree of ventilation will result in a shorter time for the fire to attain peak temperatures than if less ventilation is available.

These three influences are all taken into account in the calculation methods set out by Petterson, Magnusson and Thor (1976).

Once it has been agreed on the real fire situation as a basis, the gas/temperature condition in the fire room can be calculated. From this the designer may calculate an equivalent time period in the standard fire condition, and use this period instead of the code or regulation requirement for fire resistance in applying results of the standard test. The resulting Method 2 has the appeal that it can be applied where no information is available on the thermal characteristics of the fire protection materials under consideration, a situation likely to apply currently in New Zealand.

Method 3 bases calculations on the time temperature conditions of the standard fire. This has the important advantage of providing an alternative to carrying out expensive fire tests. It is an approach approving authorities can accept because they were able to relate results to the fire resistance time periods required by their codes or regulations.

The final option is to follow the full fire engineering design of Method 4. This takes the real fire gas/temperature condition in the fire room, calculates the resulting temperature conditions in the structural members, and predicts the time to collapse under loaded conditions.

Any building is designed for a specific occupancy and from a fire safety point of view changes in occupancy are not desirable. In practice sufficient safety margins are built into Method 1 that occupancy changes can be considered. The basis for the calculations of real fire gas temperatures, critical steel temperatures and collapse loads suggests all these reduce the inherent safety margins of Method 1. A study of each method as structured in Figure 1 will show the effect of these reductions. Method 1 has the most margin, followed in order by Methods 3, 2 and 4. This indicates that full design methods may only be appropriate for structures such as grandstands, passenger terminals, or special industries where the occupancy is not expected to change during the life of the structure. A further application of the real fire approach in use today is where an existing building, already designed by the regulatory method, is being assessed for a change of occupancy which would require increased fire resistance under the code, then an analysis of a real fire situation may well show the existing provisions to be adequate.

There are two further obstacles in applying methods 2, 3 or 4. The first is that NZS 1900,5 makes no formal provision for accepting any alternative to Method 1. Some overseas codes do have provisions covering the acceptability of calculation methods, and any revision to NZS 1900,5 should include this. Secondly there will be difficulties of understanding and acceptance by Approving Authorities. Use of fire engineering will involve the building official in a more complex situation than in Method 1 and it will involve him in a new learning process. He may have to seek advice from an independent authority on the validity of a design proposal, and this is in fact happening in the United Kingdom.

METHOD 1 - REGULATORY REQUIREMENTS AND STANDARD TEST RESULTS

This method will be to some degree familiar to all those involved with buildings. It has formed the basis for structural fire protection in New Zealand for over 40 years and no doubt much longer in other countries. Nevertheless many are not aware of the basis of this method nor its limitations, and these should be understood because, no matter how fast the other methods become accepted and used, this method is likely to remain the predominant one for many years to come.

The standard fire resistance test is used in many countries, and the national differences in its procedures are comparatively small. The version generally used in N.Z. is ISO 834: 1975 Fire resistance tests - Elements of building construction. Similar national standards include BS 476 Part 8, ASTM E119 and AS 1530 Part 4. The test is a laboratory simulation of a flaming fire, standardised to provide a common basis for comparison (see Figure 2).

There have been many critics of the standard fire test. In its early development, the gas time-temperature regime of this test was thought to simulate a real fire situation, but subsequent knowledge has revealed that real fire conditions can vary widely from it. Furthermore it has other deficiencies which include: only one specimen is tested (a few countries require two); the tested specimen may not represent the real building construction conditions; and differing design details and type of fuel used in test furnaces may produce different failure results on the same test assembly.

In addition to the disadvantages inherent in the test procedures the regulatory fire resistance periods advancing in $\frac{1}{2}$ and 1 hour steps are arbitrary in comparison with increases in severity of real fires. This system is designed for simplicity of operation with codes and regulations and is recognised as being conservative. Fire incident history in countries where this regulatory method applies tends to indicate it errs on the safe side. However, considerations of cost have focused attention on whether protection of steel is necessary or how much less protection would still be adequate, and those considerations have motivated much interest in other methods of determining steel protection.

Method 1 solutions are conservative in that the practical solutions keep the steel "cool", i.e. they ensure that temperatures in the fire exposed steel do not exceed values that are found to be critical in steel behaviour. In terms of the fully loaded steel member which is required to be tested this temperature for common structural steels is 550°C. It will be seen later that there are circumstances in which this temperature can be safely exceeded, with consequent cost savings by reducing, or even eliminating protection.

METHOD 2 - EQUIVALENT STANDARD FIRE AND FIRE TEST RESULTS

Method 2 identifies the time on the standard fire time/temperature curve corresponding to the predicted maximum gas temperature in the real fire (see Figure 2). The background to this method was first explained by Law (1972). It was later summarised by Cooke (1975), and Petterson, Magnusson and Thor (1976) provide detailed calculations. It enables the use of identifiable building characteristics (fire load and ventilation data) to determine an equivalent time exposure in the standard fire test. These calculations omit the influence of the thermal characteristics of the fire surroundings, and while this appears to be a deficiency, in the very detailed explanation in the European Convention for Constructional Steelwork (1974) it is shown (pp 11-22 to 11-42) to be insignificant for structural steel members. In the UK, Method 2 has reached a "status" of formal recognition in that it is included (clause 8.2.4) in the draft Code of Practice BS 85/12865 DC. Its limitations, along with those of Methods 3 and 4 will be discussed later.

Once the equivalent time exposure is determined, this time is then used in substitution for the regulatory time prescribed by the code or regulation, and the procedure then follows that of Method 1.

METHOD 3 - STANDARD FIRE AND CRITICAL STEEL TEMPERATURE

In this method use is made of the time/temperature characteristics of the standard fire as input to determine the time to failure. It is a shortcut to the early steps of Method 4 because it avoids the choices on fire loads and ventilation necessary to carry out a real fire calculation.

Once the standard fire characteristics are adopted as input, then the procedures of Method 3 follow similar principles to Method 4, but they include simplifications. The design method takes account of location and thermal response, but assumes constant thermal properties of protective materials and takes them to be the average for the temperature range considered. It also assumes that the mechanical properties of steel at elevated temperatures are independant of time, and that there is a uniform distribution of temperature throughout the steel members. The end result of these calculations is to prove that the calculated time to reach critical steel temperatures (i.e. failure) is equal to or greater than the prescribed fire period called for by the codes or regulations.

Details of the design method can be found in the ECCS Recommendations (1983). Pettersson & Witteveen (1979) provided a detailed background to these recommendations as they were drafted in 1979. In that paper the authors pointed out that calculated fire resistance results by this method generally turn out to be lower than obtained in a standard test, and proposed correction factors to allow for non-uniform temperature distribution in the steel specimens, and to acknowledge that the actual yield strength of the steel will exceed the nominal value used in the calculations. These corrections have been included in the final (1983) Recommendations.

METHOD 4 - REAL FIRE AND COLLAPSE LOAD

Once again, as in Method 2, the starting point is the real fire gas/temperature heating conditions. Here this information is applied directly to a knowledge of the beams and columns that go to make up the structure, with a view to determining how hot the steel becomes.

The first step in this method is the determination of the temperature at the surface of the steel, which will be lower than that of the surrounding hot gases. There is little that is new in this if one has an understanding of heat transfer physics. It does, however, require data on the heat transfer properties of fire protection materials at elevated temperatures, and such data is not readily available for New Zealand materials.

The location of the steel member in relation to the rest of the structure has an important bearing on the temperature it attains in a fire. An internal column, surrounded on all four sides to the fire, will be subject to the maximum temperatures obtained, unmodified, from these calculations. Where columns are external to, and separate from, the building envelope, the effect of a fire inside will be markedly reduced. A beam supporting a concrete floor, or a column partially buried in a wall, will suffer less exposure, as will a beam protected by a suspended ceiling or located high enough above a fire so as to be above the flames. Ways of taking account of these differences in location will be discussed later.

The way in which the steel member responds to the high temperature at its surface will depend on the Thermal Response Factor, the ratio of surface area (for collecting the heat) to the cross section of steel (for absorbing the heat), which will be discussed in more detail later. The use of this ratio has been extensively developed in recent years in the scientific understanding and application of protective coating technology.

The final step is to use the temperature conditions in the steel to determine their effects on the ability of the steel to continue to carry load. This involves a knowledge of the mechanical properties of the steel, and the load conditions. The mechanical property that is of primary interest is the yield strength, which is temperature related. Figure 4 shows a typical relationship with temperature of the yield strength of common structural steel grades, and shows the importance of understanding the relationship of design load level to load capacity, because increased fire resistance can be obtained if lower than full design stresses are used.

For a more detailed presentation of the full fire engineering design (Method 4) readers should study Pettersson, Magnusson and Thor (1976), which has a flow chart on p 144 useful for fully understanding the method. The end result in these calculations is to prove that the load capacity of the structure subjected to the real fire conditions is equal to or, preferably, greater than the applied load.

PROTECTION SYSTEMS

Manufacturers offer a wide range of choices of proprietary fire protection systems. These have been subjected to the standard fire test as the accepted means of gaining approval for use under methods 1 and 2. Figure 3 sets out a breakdown of the conventional solutions available to meet regulatory fire resistance requirements. All these solutions can be seen as keeping the steel cool by placement of a separating and insulating layer between the steel and the hot fire gases. It is convenient to subdivide these solutions into the two broad categories of encasements and profile coatings, and these can both be further subdivided, as shown in Figure 3.

Encasements

Board Casings (see Figure 5)

In the larger industrialised countries, there are many systems offered in this category. For example, in the U.K. there are 23 systems approved for use (ASFPCM and Constrado 1983). These systems include boards containing gypsum plaster, mineral fibre, calcium silicate, or Portland cement, and also ceramic fibre blanket and preformed insulated sheet steel. Standards Association of New Zealand, Fire Properties of Building Materials and Elements of Structure (MP9:1980) does not list any such systems suitable for beams or columns as approved for use in New Zealand, although it is understood a limited number have been approved in recent years and will be referenced in a new edition under preparation.

With board casings it is necessary to prove by test that the board has the right properties to insulate the steel, and in addition that it is so fixed and supported, that it will remain in place long enough to carry out that function. This means that the manufacturer of each proprietary system has to establish methods of fixing to the steelwork suitable for his material, and to show by fire resistance tests that these are adequate. The technical data for each system must include adequate specifications for fixing which must be followed to ensure compliance.

Retardant Plasters (see Figure 6)

A method of encasement which has been popular in the USA and UK was to wrap a metal mesh around columns or beams and coat with hand applied plasters. The plaster base was usually gypsum and contained vermiculite or Perlite. This method was particularly attractive where adjacent ceilings or walls were to be similarly plastered because the provision of fire protection was no longer a separate building process. This method of protection has been used in New Zealand in the past, but seems to have little popularity today.

Concrete or Masonry (see Figure 7)

The encasement of steel with cast in place concrete is probably one of the oldest systems used anywhere (Figure 6). Most countries include recommendations in their building codes or regulations for amounts of concrete cover to the steel, which if complied with are deemed to satisfy the fire rating requirements. Difficulties of casting column casings have been a disadvantage of this method, and there are examples in New Zealand of how this has been overcome by using precast concrete channel casings. Other disadvantages, such as excessive weight which increases the seismic design problems, and loss of floor space from increased column size, have tended in recent years to put this method of protection amongst the least favoured. However, in situations such as industrial occupancies, where resistance to damage is important this method continues to have application. Encasement of steel beams with concrete would seem to be a practical possibility, with the advent of concrete pumping techniques.

Encasement of columns by concrete or ceramic masonry has the attractions of a good quality finish, and of damage resistance, but these are offset by the same disadvantages that apply to concrete, i.e. too heavy and takes up too much floor space, and in addition the high cost of skilled labour for placement. Masonry encasements seem impractical for beams.

Profile Coatings

Sprayed Fibre (see Figure 8)

Protection coatings consisting of a cementitious material, usually fibre-reinforced, have been common for many years. They have been very popular in North America where the presence of many systems has made costs very competitive, and many applications exist in New Zealand buildings. Since sprayed coatings are rough and uneven, they are not attractive for locations exposed to view but where appearance is not a problem they provide an efficient protection system, with advantages of being lightweight, and space saving. They are often relatively soft and therefore susceptible to physical damage, hence they need encasement where there is potential for damage. Concerns have often been expressed about damage by subtrades following spraying, and it will be of interest to note that recent research (as yet unpublished) by the British Steel Corporation has shown that significant amounts of sprayed materials may be removed without affecting the overall fire resistance of protected steel members.

The principal fibre traditionally used in sprayed systems has been asbestos but this is now prohibited in countries where concerns have arisen about the health hazards of asebstos fibres. This is a big loss to the building industry since asbestos in all its forms has for many years been a useful fire protection material. Most spray-on systems manufacturers have developed mineral fibre substitutes for asbestos which have been tested and approved.

Intumescent Coatings (see Figure 9)

This is a relatively new method of protection by application of thin coatings (typically 0.5 to 3 mm thick) containing chemicals that interact at 150 - 200°C (intumesce) to form a stable foam layer which can be up to 75 mm thick and insulates the steel from fire temperatures. Such coatings can be applied by the normal painting techniques of brushing, rolling or spraying, but they should not be confused with intumescent paints available for retarding ignition and inhibiting flame spread on combustible linings.

Intumescent coatings appear to offer great potential for steelwork fire protection. There are currently seven proprietary systems shown by ASFPCM 1983 to be approved in the United Kingdom, where an air of confidence is evident in their use. They offer the advantages of lightness and space-saving. Some makes are supplied with a wipe-clean sealer top coat which is tough enough to protect the underlying fire protection coating from mechanical damage. The sealer coat of one such system is offered in any of the BS 4800 colour range to provide a decorative finish. Of the seven systems available in the UK, 2 offer ½ hour protection only, 3 offer ½ and 1 hour, and the other 2 offer 1½ to 2 hours. Coating thicknesses range from 0.6 to 5 mm in the ½ and 1 hour range, and up to 12.5 mm for 2 hour protection. One of the UK systems is now available in Australia and New Zealand and provides ½ and 1 hour protection. Protection times are not necessarily related to coating thickness, and at the present time it should not be assumed that an intumescent coating would survive for any significant time beyond that already shown to be satisfactory by test.

DETAILED CONSIDERATIONS NECESSARY FOR THE DESIGN OF FIRE PROTECTION OF STEELWORK

Thermal Response Factor

It has been known for many years that the greater the mass of a steel section and the smaller the perimeter, the better the inherent heat sink. From this knowledge has been developed the concept of the Thermal Response Factor or the ratio of the heated perimeter Hp to the cross-sectional area A of steel. A section with a large Hp/A (large surface area to collect heat and small mass of steel to absorb it) will take a shorter time to reach a critical temperature than will one with a small Hp/A. Therefore sections with small Hp/A values need less thickness of applied protection than those with large values. In calculating the Thermal Response Factor the perimeter Hp is that which requires protection. With four-sided encasement systems Hp is the sum of the sides of the enclosing rectangle, but with profile coatings Hp is the length of the total profile. In either case, where a concrete floor is cast on a steel beam, the perimeter is reduced by the width of the top flange to permit three-sided protection.

In the U.K. the Thermal Response Factor concept has been systematically developed for a large range of protection systems, information on which is published in ASFPCM 1983. This includes tables of Hp/A values for both three- and four-sided protection for all the steel sections available in the U.K. Data for proprietary systems of protection have been obtained from fire tests and the results incorporated in equations which relate fire resistance to thickness of protection and the Hp/A factor. An equation is specific to one protection system but from it tabulated recommendations of thickness v. Hp/A are provided for each regulatory fire resistance period (Figure 10).

Similar concepts are in use in other countries although the terminology differs. The ECCS Recommendations (1983) use surface area (F) to volume (V) ratio with the unit for F/V being m^{-1}. In Australia, Bennetts, Proe and Firkins (1984) use exposed surface area (ESA) to mass (M) ratio ESA/M, with units for ESA/M being $mm^2 . kg^{-1}$. In USA, the International Conference of Building Officials Uniform Building Code 1985 Standard 43-9 uses weight per unit length (W) to heated perimeter (D) with units for W/D of $lbs. ft^{-1} . ins^{-1}$. Thus, while the end result of calculations in these countries will be the same, the difference in terminology makes for difficulties in transferring numerical information. In New Zealand the SANZ Fire Ratings Committee has decided to follow the U.K. method.

Critical Temperature and Load Factors.

At some elevated temperature, called the critical temperature, steel will yield if the imposed load exceeds the loadbearing capacity at that temperature - as is illustrated in Figure 4. Most structural codes call for load factors which dictate that stress levels in structural members at full design loads will be about 0.6 of yield strength. It will be seen from Figure 4 that the corresponding temperature is 550°C. In the U.K. 550°C has been adopted as the critical temperature for the standard fire tests and assessment procedures applied to proprietary protection systems, and while this seems the only possible way to deal with it, the fixed value for the critical temperature is very restrictive.

To overcome this, for fire engineering calculations, BS 85/12865 DC has adopted a ratio of applied load at the fire limit state to the ambient load capacity, and Table 4 of that document recommends a range of values of this ratio from 0.7 to 0.2, with limiting temperatures ranging from 470 to 790°C. This provides the designer with greater flexibility to choose a limiting temperature appropriate to the level of loading in the structural members, rather than be limited to 550°C. Advantages accruing from this can be a reduction in protection needs, and therefore cost, with the limit being proof that no protection may be necessary.

Restraint and Continuity

Rotational end restraint is undoubtedly effective in reducing stress conditions in a beam and therefore can be of value in reducing protection needs. From unpublished information aimed at quantifying the advantages of beams fixed to columns (to provide rotational restraint) or from continuous beams in multistorey buildings, British Steel Corporation claims that the fire resistance of small section sizes, ie with high Hp/A values, can be raised to well in excess of 30 minutes with failure temperatures in excess of 800°C. A user's design guide is in preparation and it is expected this will assist in taking advantage of this information.

Composite Action

It is well known that under ambient temperature conditions a concrete floor slab rendered composite with a steel beam supporting it provides a considerable increase in the section modulus over the steel beam alone. This situation is recognised in determining the Thermal Response Factor, where three-sided protection is appropriate, but it hardly takes advantage of the composite action. Kirby (1985) reports briefly on a test by British Steel Corporation on a composite beam/slab section which showed the fire resistance could be raised from 23 to 35 minutes. No information was given by Kirby on how to calculate this advantage of composite action and so further investigations are needed.

Degrees of Protection

Full Protection

Every designer is familiar with protection systems which fully protect steel members, and these have been discussed above. These solutions ensure that none of the steel is exposed to the fire.

No Protection

Heavy Sections can achieve worthwhile fire resistance when unprotected. Currently, information in Kirby (1985) suggests this may have an upper limit of only ½ hour which restricts application of this solution to buildings required by

Methods 1 and 2, or established by Methods 3 or 4, to those needing only that level of fire resistance. Nevertheless this is a worthwhile approach since often it will be found that the cost of increasing the mass of a steel member is less than applying an expensive protection system to a light steel member.

Partial Protection

In between the extremes of full or no protection, there are construction details where part of the steel section is built into a floor or wall. It will be evident the unexposed portion of the steel section does not attain very high temperatures in a fire and can provide stability even when the strength of the exposed portion is degraded by heat. British Steel Corporation has conducted investigations which have shown by tests that a variety of sections can achieve considerably longer periods of fire resistance when only partially exposed. Figure 11 shows some of these details. No design guide appears to be available on this yet.

Combined Steel/Concrete Sections

Hollow Sections

Where structural hollow sections (SHS) are filled with concrete (Figure 12) the fire resistance can be increased. Whereas an unprotected hollow section has little fire resistance, due to rapid heating up of the thin steel walls, the presence of concrete will absorb heat from the steel. Even so, if the strength of the column is based on the steel section only the fire resistance is unlikely to be more than ½ hour. However, the combined effect of the heat absorbing capacity of the concrete together with the composite action of shedding load from steel to concrete has been shown to achieve fire resistance ratings as high as 2 hours. At the limit,the entire load is carried by the concrete but the steel still remains able to contain the concrete from spalling, thereby delaying sudden failure.

Where columns carry bending during a fire, plain concrete cores can only withstand small bending moments once the external steel has yielded, because of the small tensile strength of the concrete. This can be overcome by reinforcement within the concrete, and both steel fibres and conventional reinforcing have been shown to be effective. British Steel Corporation Tubes Division (1984) has published a design manual for SHS concrete filled columns which contains detailed design procedures relating load capacity to fire resistance times. Reservations have been expressed about some aspects of this manual, and perhaps the recommendations it contains on fire resistance need to be treated with some caution.

UB and UC Sections

A method of combining concrete with steel in the form of UB and UC sections, has been developed in Europe. Technical impetus has come from ARBED Recheches of Luxembourg which is a steel industry research and marketing organisation. ARBED has had computing work done at the University of Leige, Belgium, and has made use of testing work done at several European fire testing stations. This form of construction (Figure 13) also combines the heat sink effect of the concrete with composite steel/concrete interaction, and again, as in hollow construction, the concrete has to be reinforced to achieve useful load capacity in fire situations. Fire resistance ratings of ½ to 2 hours are possible. At present, tables are available (in German) of fire resistance v. load for columns for use with European steel sections, and work is progressing on tables including US sections, and for beams. Computer programs are being offered by ARBED and it may be possible in the near future to make use of these.

Both the forms of combined construction are attractive in that the protected member is no larger than the steel section, and the good finish of rolled steel can be exploited with paint finishes. The exposed concrete faces in the ARBED system can be given a variety of treatments, including exposing the aggregate. Both systems appear to be expensive and it remains to be seen whether they will have much application except in special structures where their features can be exploited.

British Steel Corporation has experimented with column sections where the space within the flanges were blocked in with lightweight concrete masonry (Figure 14). Tests have shown it is possible to achieve $\frac{1}{2}$ hour FRR on selected fully loaded sections. It appears this result is achieved by a reduction of the heated perimeter combined with the provision of a heat sink adjacent to the steel. These results cannot be applied in New Zealand unless tests are done on local masonry.

Water Cooling

Around the world there are now a number of buildings whose main structural members are hollow and contain water. In the event of a fire the heat passes through the steel and is absorbed by the water. Members are interconnected and the system is provided with a header tank so that a circulation is set up to continuously cool the heated members. Such a system seems to have the potential of coping with a fire of any duration so long as water circulation is maintained. An engineering design method has been published by Bond (1974). It seems doubtful if watercooling will be used extensively because of the cost of providing safeguards and maintaining them to ensure readiness for action throughout the life of the building.

External Steel

Structural members may be unprotected when located outside the building envelope. Where columns are located between windows so as to be shielded from fire in the interior by fire resisting construction, and in such a way as to avoid flame impingement, then the steel may not attain critical temperatures. A similar effect can be gained by siting the columns sufficiently far from the facade that radiation from the fire and the flames is dissipated. Designs have also been proposed for shielding spandrel beams from flames emerging from windows so that the beams need no protection. Law and O'Brien (1981) have published a design guide with calculations for determining whether external steel needs protection.

DESIGN CODES

A number of design codes now cover some applications of fire engineering design.

In 1985 British Standards Institute issued BS 85/12865 DC and this draft is intended to become Part 8 of BS 5950 The Use of Structural Steel in Buildings, and will be a Code of Practice for the Fire Protection of Structural Steelwork. In this draft, all the four methods covered in this paper are envisaged, and much useful tabulated data is included for use in carrying out fire engineering design calculations. It is short on guidance on Method 4, since it states (Section 8) that various calculation methods may be used, but then goes on only to provide guidance on the equivalent time procedure of Method 2. Nevertheless, the publication of this document is an important step in making it possible to use fire engineering design methods.

A committee of the Standards Institute of Australia has been working on a similar document, to become Section 12 of AS 1250. At this time of writing it is not known what progress has been made.

In the USA, the International Conference of Building Officials Uniform *Building Code* 1985 (Sect. 4302) permits ratings to be established by calculations which are published in UBC Standard No 43-9. These are limited to columns only, and represent an elementary approach to calculating the thickness of applied protections for satisfying regulatory fire resistance requirements without the need to conduct standard fire tests.

In New Zealand, similar code provisions are neither available, nor under discussion. Clause 5.3.2 of NZS 1900.5 does provide a means whereby fire resistance requirements may be deduced, but it provides no rational basis for doing so.

CONCLUSIONS

There are four methods which may be used in determining the degree of fire protection needed for structural steel. These are summarised in Figure 1.

Method 1 - Regulatory requirements and standard test results

This method has a long history of use in many countries. It is a "rule-book" approach to stating the requirements for structural fire protection, and goes hand-in-hand with a "cookbook" of recipes for complying with the rules. Its application is basically simple, since the approval authority has only to require proof that the construction meets the requirements. In New Zealand the "rule-book" is NZS 1900, 5, and the "cookbook" is the MP9 Series. The resulting protection has many built-in margins of safety.

Method 2 - Equivalent standard fire and standard test results

To overcome the deficiencies identified in the regulatory time steps system Method 2 replaces this with the temperature conditions of a theoretical real fire, corrected to an equivalent time period in the standard fire condition in order to make use of the "cookbook" of results of fire tests. It has the advantage that in most situations the calculations will call for shorter time periods than the codes or regulations, with consequent savings in protection costs.

Method 3 - Standard fire and critical steel temperature

This method is attractive in that it shortens and simplifies the design process by using the standard fire time/temperature relationship which may make it more acceptable to the building official. The resulting protection needs will be, in most cases, more costly than those arrived at by use of Method 4.

Method 4 - Real fire and collapse load

Using the accumulated scientific knowledge of fire behaviour and the response of building materials to fire temperatures is the ultimate in the application of fire engineering and has potential for special structures. One advantage is the saving in cost of using exposed steelwork, or if that is not achievable, then a reduction in protection needs. Another is a reduction in the dependence on costly fire tests.

The future

The development of fire engineering has been boosted by the interest taken by the iron and steel industries in several parts of the world. Applications of fire engineering to steel buildings are not yet common but there are examples overseas where a fire engineering approach has made possible some spectactular structures. It seems that insofar as New Zealand is concerned, the historical approach, described here as Method 1, will be with us for many years yet. There will be value in pursuing Method 2, and in fact this was the basis on which recent consideration was given to changes in the fire resistance requirements of NZS 1900,5 under Amendment 16. Methods 3 and 4 are likely to have more limited application, at least until we are able to gain information on the thermal properties of the steel used in New Zealand, and of the protective processes which are available here. The building industry could well apply itself to determining and publishing this information.

Some building codes, such as NZS 1900,5, and regulations do not have adequate mechanisms to permit acceptance of Methods 2, 3 or 4 and changes are needed to achieve this. Many building officials are unfamiliar with fire engineering calculations, and they will need guidance on how they may be applied.

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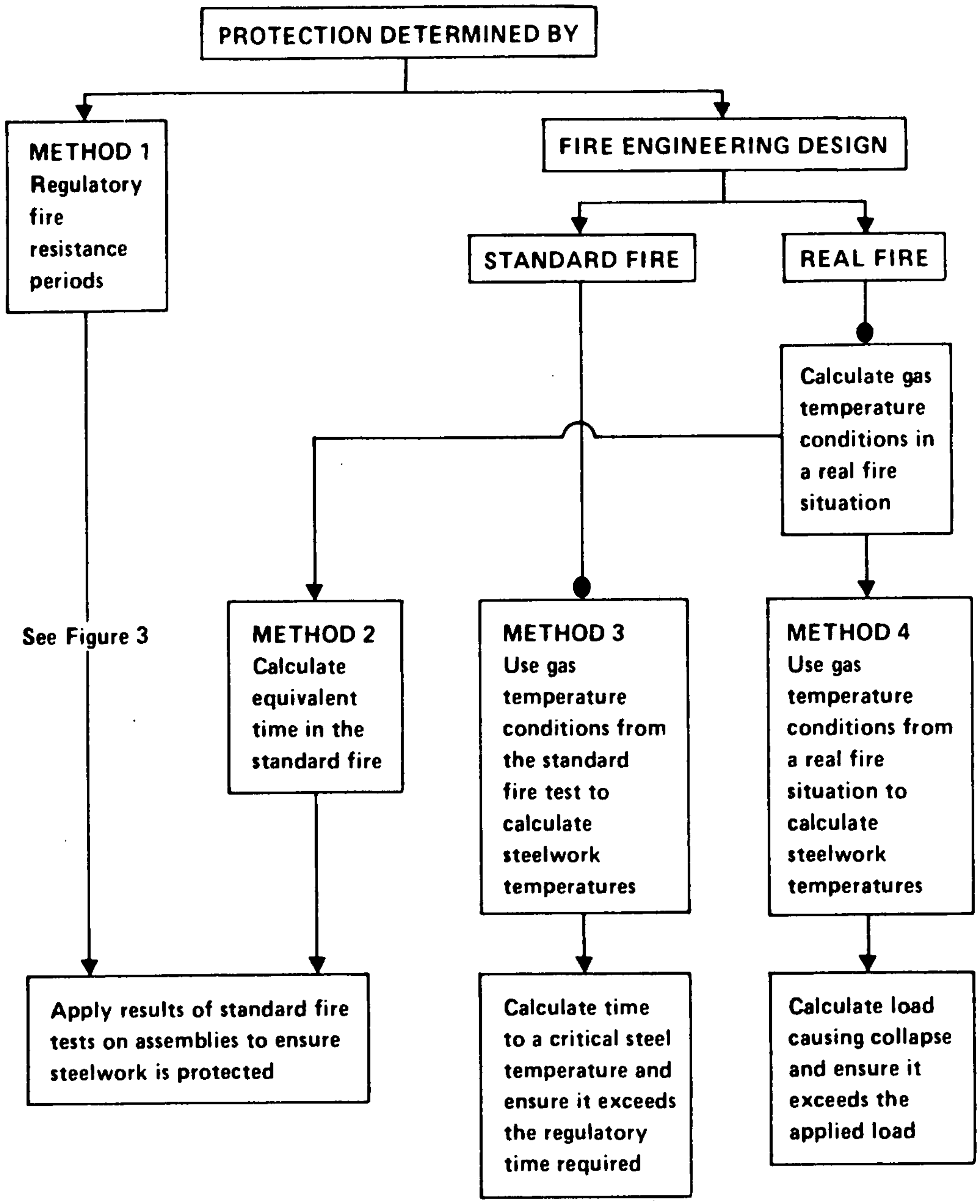


Figure 1 Alternatives available for determining fire protection of steelwork

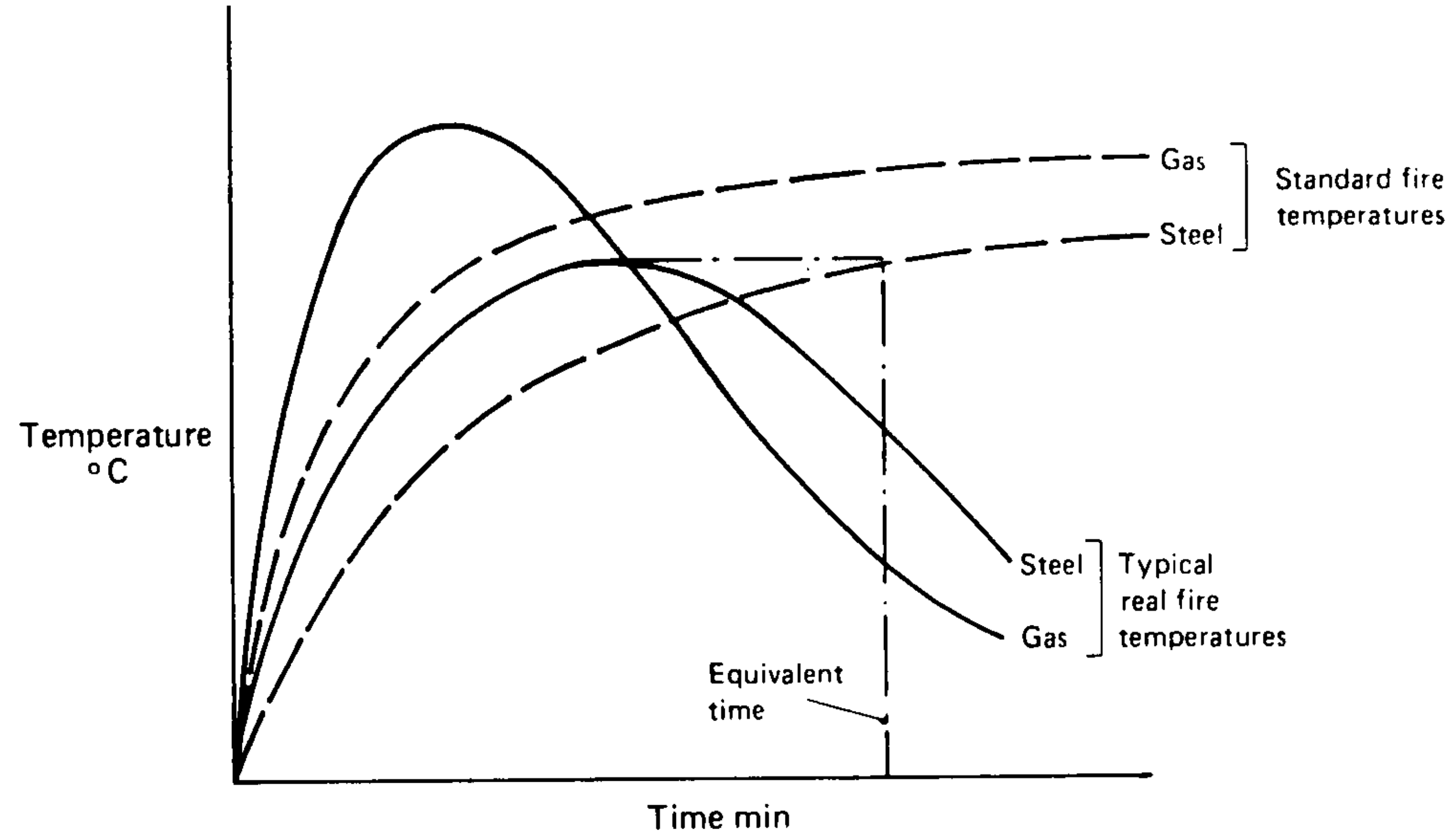


Figure 2 Standard and Real Fire Temperatures, and the Equivalent Time of Method 2.

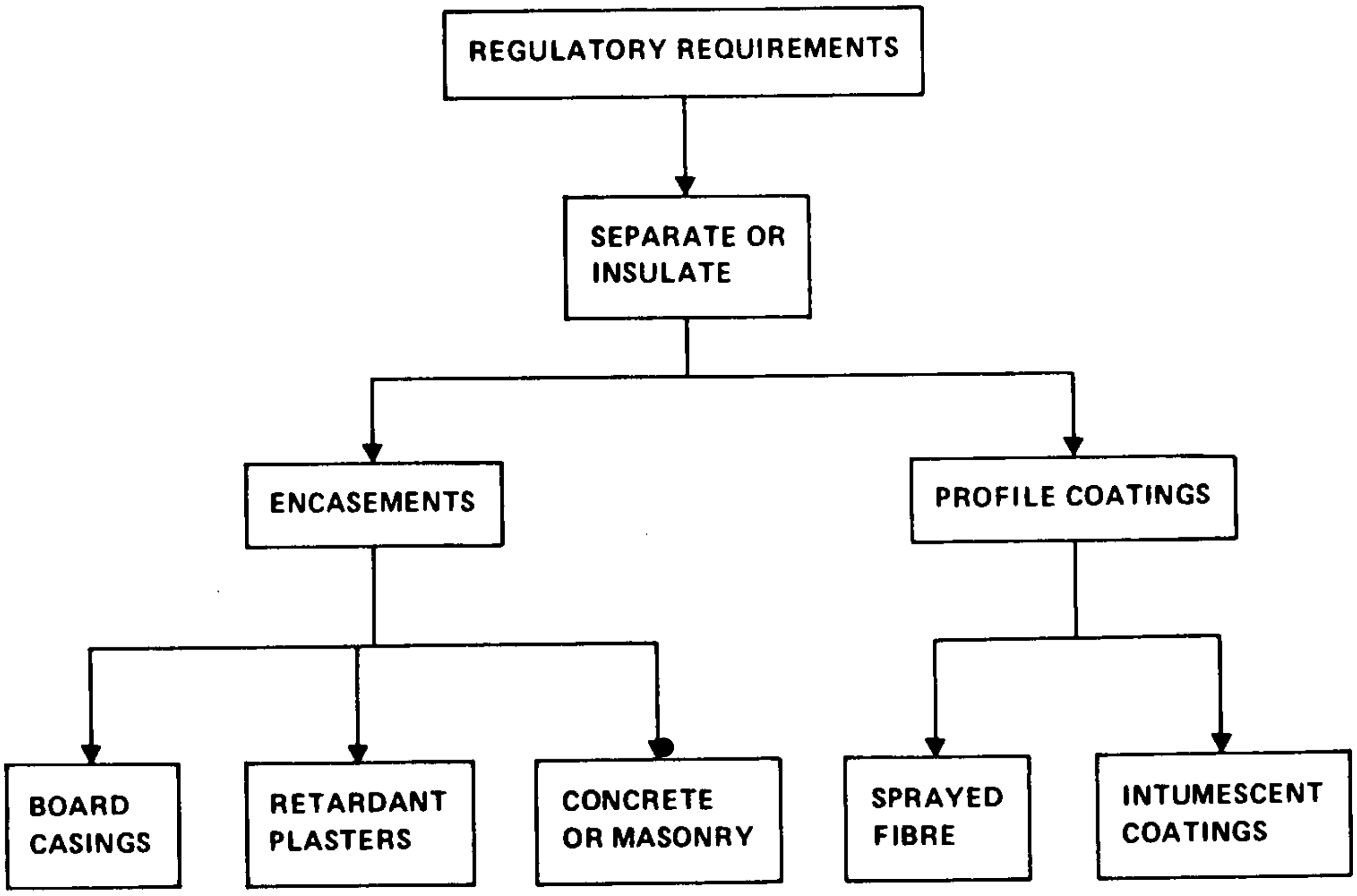


Figure 3 Options for meeting regulatory requirements for steel protection

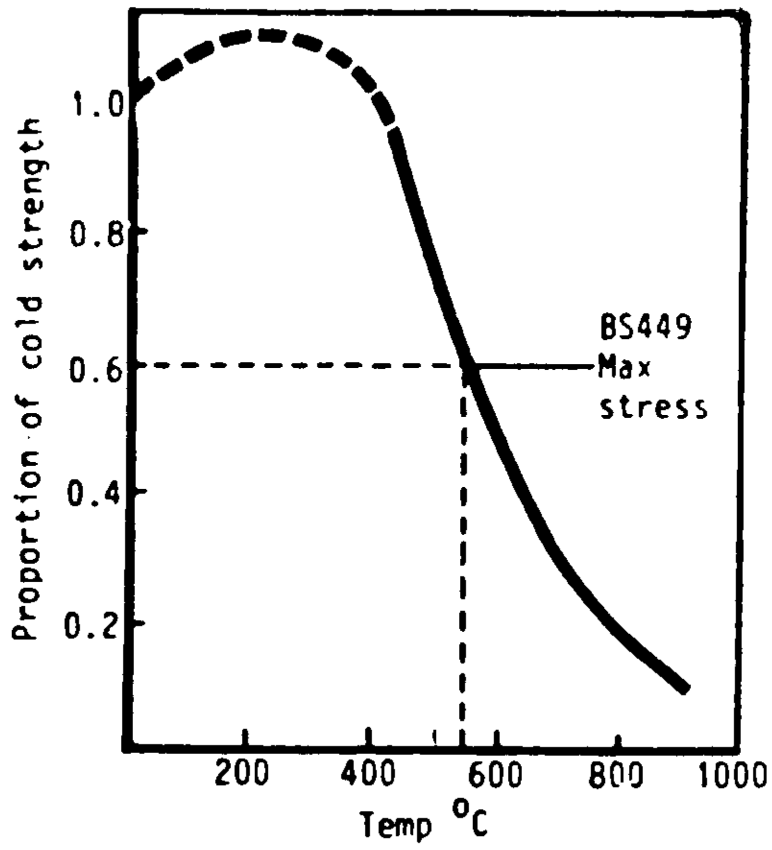


Figure 4 Variation of yield strength of structural steel with temperature

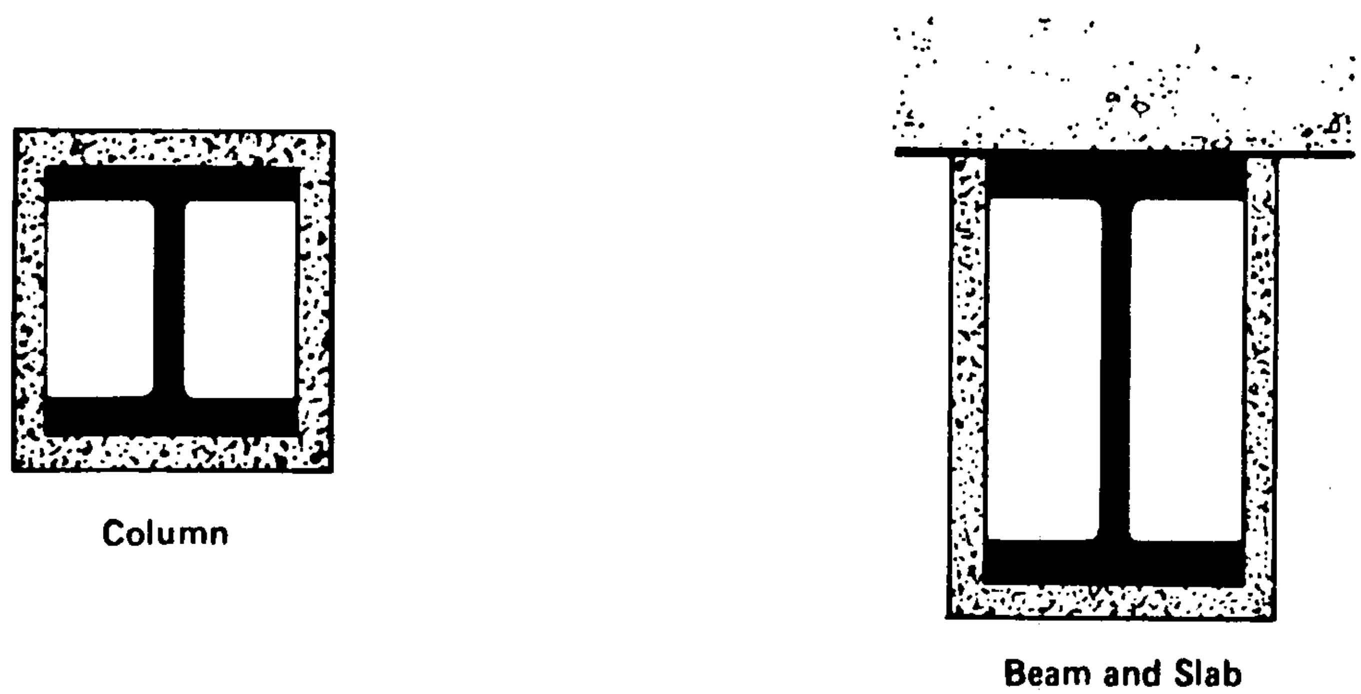


Figure 5 Board Casings

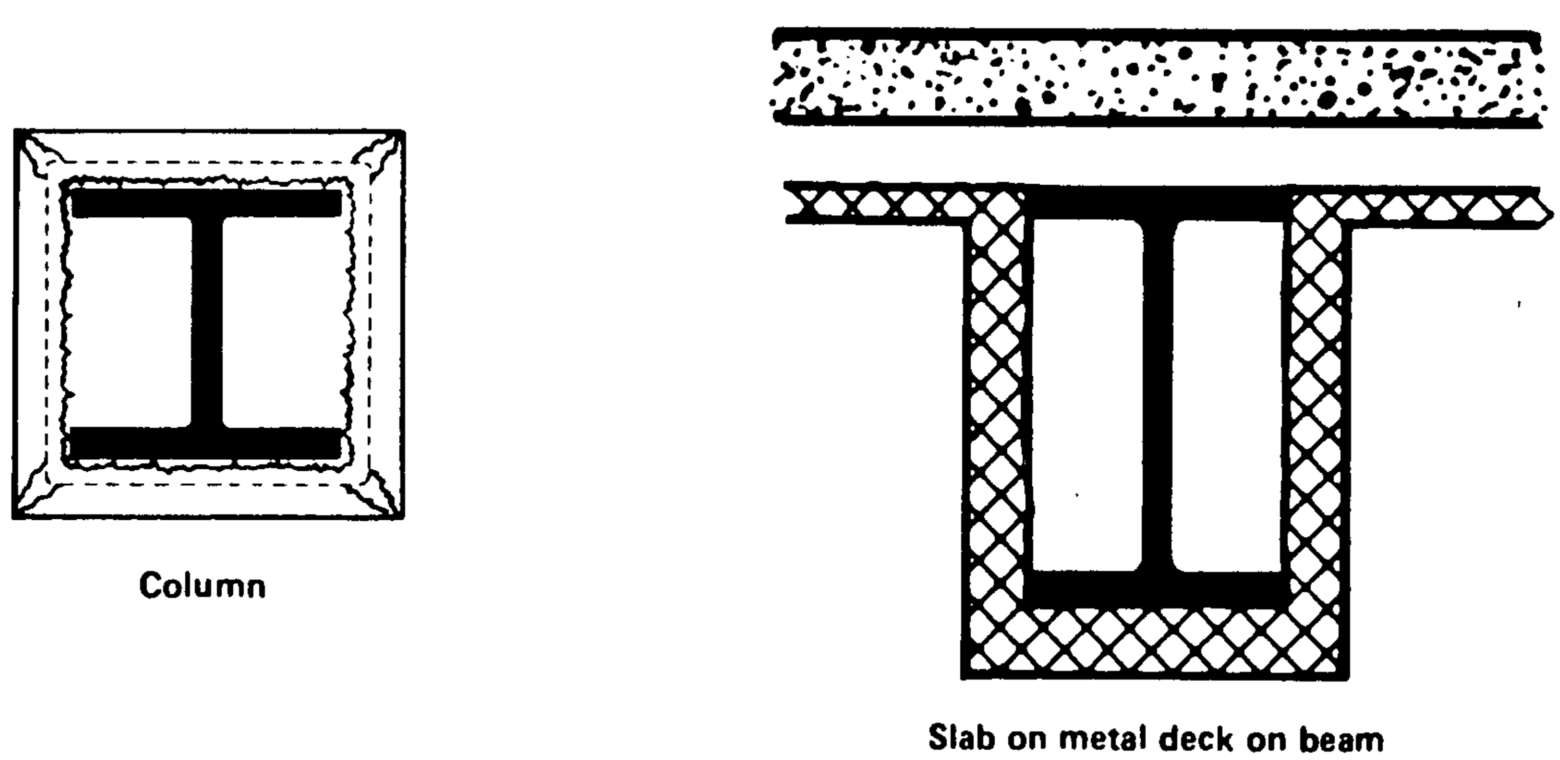
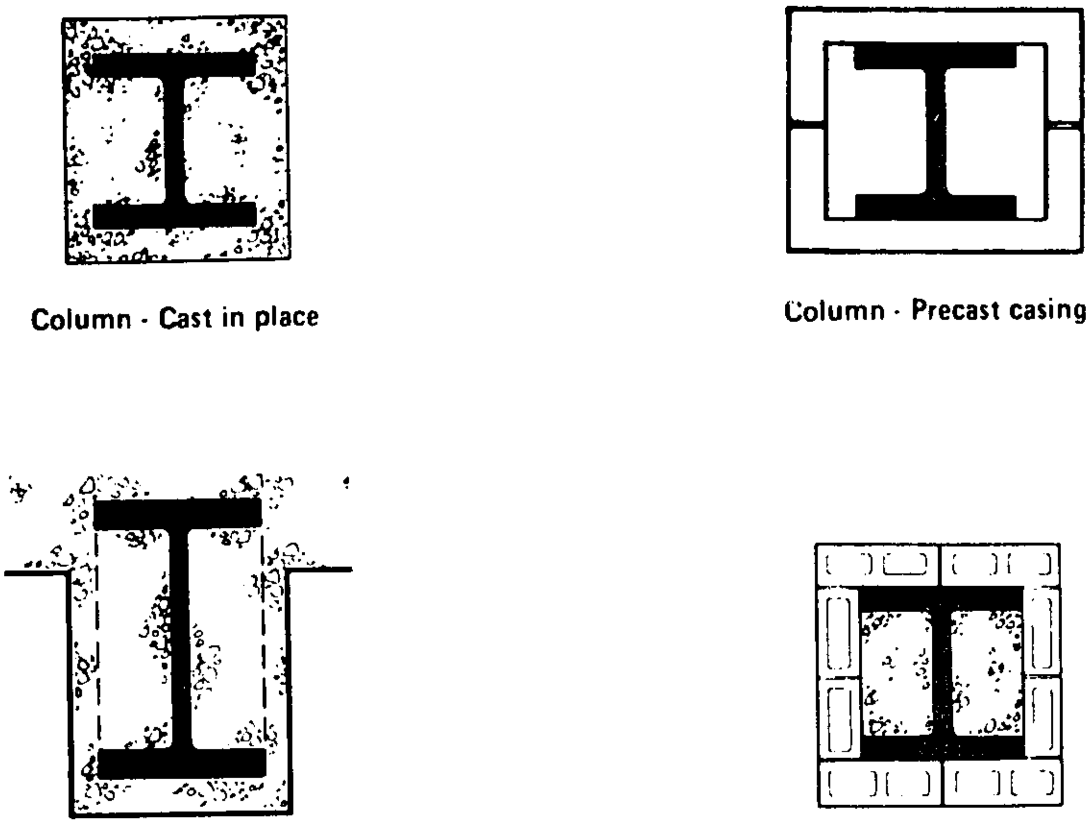


Figure 6 Retardant plasters



Column - Cast in place

Column - Precast casing

Beam - Cast in place

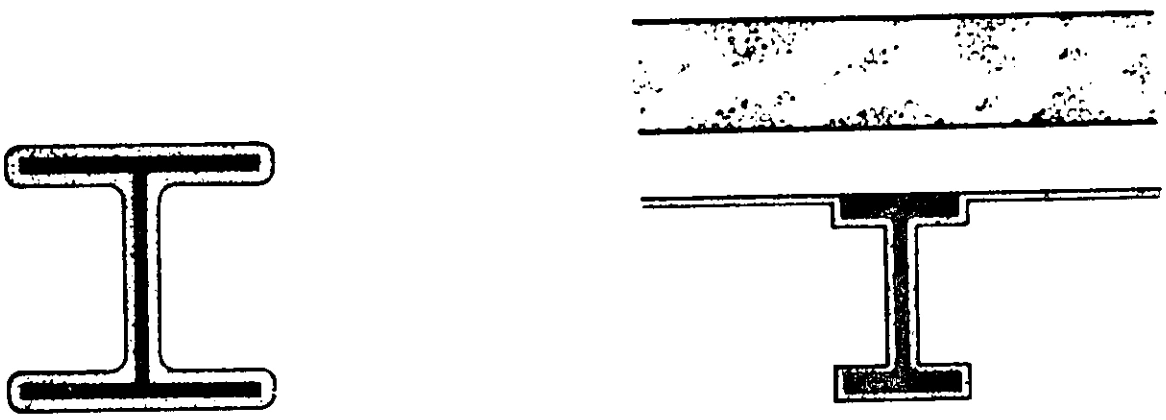
Column - Masonry

Figure 7 Concrete or masonry encasement

Column

Beam and slab

Figure 8 Sprayed fibre coating



Column

Slab on metal deck on beam

Figure 9 Intumescent coatings

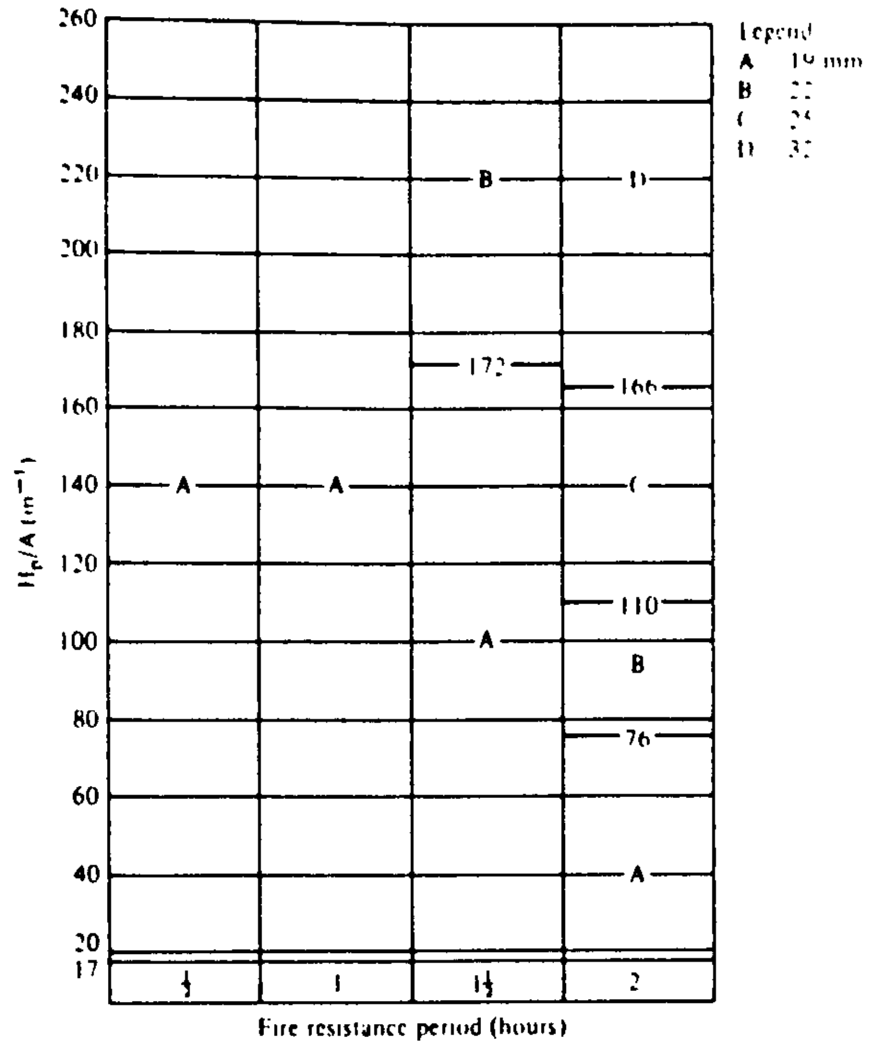
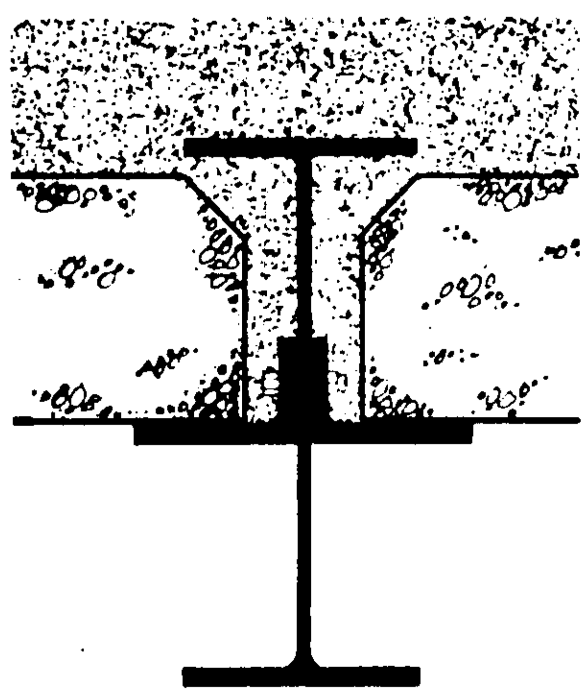
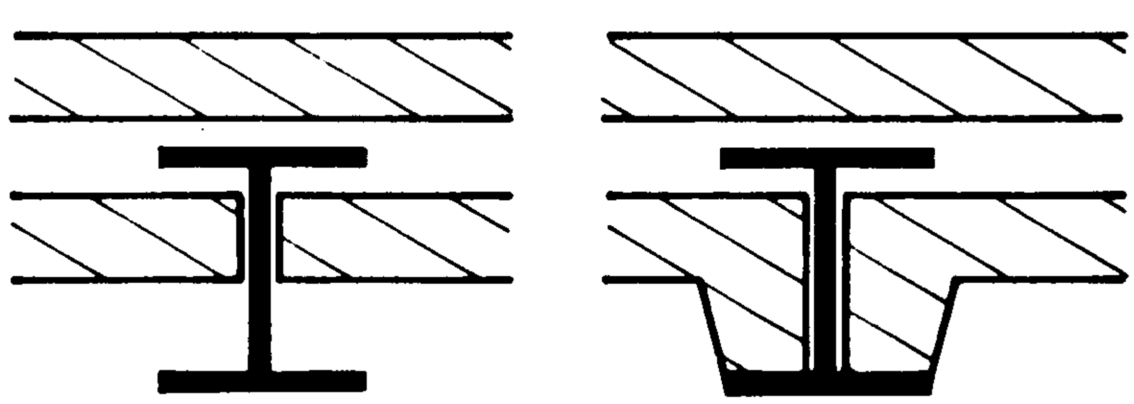


Figure 10 Typical bar chart showing relationship between coating thickness and thermal response factor



Beam with shelf angles to support precast flooring

UC used as a beam with precast flooring



Columns partially encased in a wall

Figure 11 U.K. Construction details which extend fire resistance

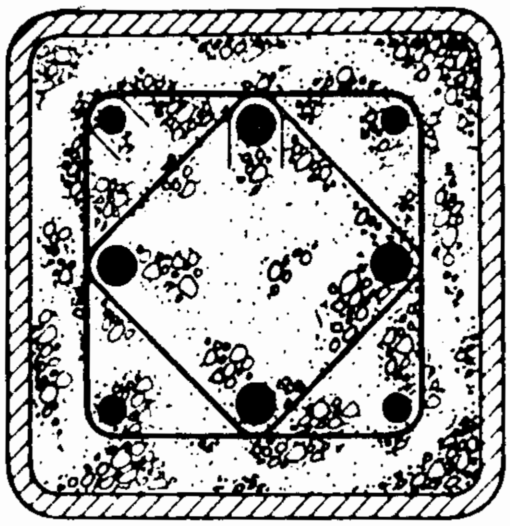


Figure 12 Concrete filled hollow columns

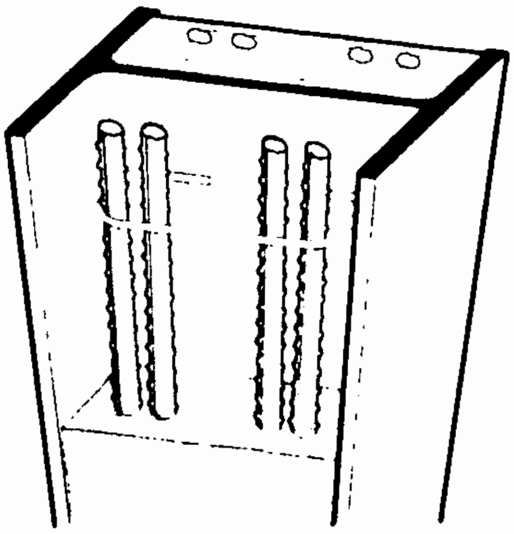


Figure 13 Concrete filled universal columns

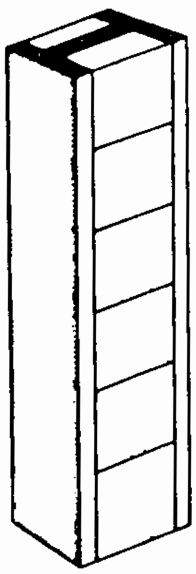


Figure 14 Lightweight masonry infil to universal columns

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