Steel-Concrete Composite construction using rolled sections
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Introduction

Steel-concrete composite construction has long been recognised and used in the form of "traditional" composite beams in buildings and bridges. In this simple form of construction, the rolled steel section is connected to the concrete slab using mechanical shear connectors at the steel-concrete interface. Because of the resistance to longitudinal shear provided by these connectors, the steel and concrete are linked structurally. The reinforced concrete slab can therefore be used not only to provide a horizontal surface in the building, but also as a compression element in the composite section. The presence of the concrete increases both the resistance and the rigidity of the steel section, which forms the tension element in the composite section under bending (figure 1).

Steel columns were traditionally often encaised in concrete to increase their fire resistance. This type of section was used long before the adoption of true composite columns, for which the reinforced concrete encasing the steel section is assumed to support part of the vertical load (figure 2).

In the 1980s it was discovered (or rediscovered) that even a partial encasement in concrete (figure 3) provides a composite column with substantial fire resistance. The open form of steel H-sections facilitates filling with concrete between the flanges whilst the steel section is laid flat on the ground, prior to lifting into place. This eliminates the cost of formwork, and compensates for any overdesign that may be needed to achieve the highest levels of fire resistance. As a result of numerous research projects, reliable methods have been established for calculating the fire resistance of columns with precast concrete between the flanges.
The same technique of partial encasement first used for columns has been extended to cover beams in order to increase their fire resistance (figure 4). Although the lower steel flange gradually loses resistance as it is exposed to a fire, this loss is compensated by the presence of reinforcement located within the concrete encasement.

Other recent developments include improved design methods for composite beams, taking into consideration continuity at supports (allowing for cracking of the concrete in tension), and partial shear connection (which, by allowing some slip between the steel and concrete elements, can improve economy).

Composite construction therefore offers considerable possibilities faced to those offered by traditional steel construction, be it in terms of fire protection or otherwise to suit particular design criteria. Because of the way steel frames are constructed, it is also possible to combine both composite and non-composite members in a single project.

The fire resistance that can be achieved using composite construction has greatly contributed to its success, with the added advantage of being able to retain exposed steel surfaces that can be used for attachments. The excellent ability of composite structures to resist seismic loading is yet another advantage of this form of construction.
European standards

Basic design philosophy

Composite construction has seen rapid adoption in countries possessing the necessary standards and design guidance. Methods for evaluating fire resistance were proposed in the 1980s in the form of specific national authorisations. Subsequently, the appearance of the Eurocodes has led to a significant generalisation of design methods, not only for normal service conditions but also under fire.

The general philosophy adopted for the Eurocodes is to ponderate the loads and forces applied to a structure by using factors. The values of these load factors depend on the nature, and variation with time, of particular types of load. Each member within a structure, and the structure as a whole, must be checked for all potential combinations of loads. In addition, particularly for beams, the designer must verify that certain criteria are satisfied under the levels of loading expected during service. These criteria concern deflections, vibration, and cracking of the concrete, which are known as serviceability limit states.

Eurocode 4 Part 1.1 (ENV 1994-1-1) gives design methods for composite beams and composite columns under normal conditions. Part 1.2 (ENV 1994-1-2) gives methods for calculating the resistance of these elements under fire loading.

Eurocode 1 (ENV 1991) defines not only the loads to be considered during design, but also the safety factors to be considered under both normal conditions and fire. For an accidental fire condition the load factor is less than 1.0 for most imposed loads, because it is considered highly unlikely that an imposed load of maximum intensity would occur at the same time as a fire. These standards were completed in each country by a national application document for the Eurocode. Requirements for fire resistance also continue to be defined at a national level and, unfortunately, there is some disparity between different countries.

Quality of materials

Eurocode 4 permits the use of a wide range of steel and concrete grades for the materials combined in a composite member.

The traditional range of steel grades (S235, S275 and S355) is supplemented with higher strength grades S420 and S460. Steels of these higher grades are achieved using the QST process (HISTAR sections), and are particularly useful for members subjected to substantial loads. On the other hand HISTAR steel grades allow a finishing without any preheating nor post-heating during welding.

Concrete should be either grade C20 till C50, with normal or lightweight aggregate. Any commonly available reinforcement may be used, S500 being the most common grade.
Fire resistance: ENV 1994-1-2

Composite sections, with either total or partial concrete encasement, possess significant fire resistance. However, it is not possible to assess the fire resistance of a composite member simply by considering temperatures in the steel (as is the case for bare steel sections, which experience a more-or-less uniform temperature across the section).

The presence of concrete increases the mass and thermal inertia of a member. The variation of temperatures within the body of the member at a given time under fire loading is significantly non-uniform, in both the steel and concrete components. This leads to substantial temperature gradients. The presence of areas near the core of the section that are relatively cold ensures that the member can remain stable for some time under fire loading.

Part 1.2 of Eurocode 4 gives several methods for calculating the fire resistance of a composite member:

- use of tables that are essentially based on the performance achieved in tests
- calculation of the ultimate resistance using a simplified method based on test data
- numerical modelling using software that has been sufficiently validated using test results, such as CEFICOSS, which is used by Arcelor Sections Commercial.

Both the accuracy of the method, and the scope of its application, increase passing from the first to the third of the methods listed above. The great benefit of software such as CEFICOSS is that the analysis of complete structures, be they flexible or rigid, is a realistic proposition. Fully encased beams and columns are generally assessed using tables, which are extremely simple to use for these applications. Simple design methods based on test results are generally used for partially encased sections.
Publications giving methods for the verification of the fire resistance of other composite sections, and for more complex load situations, include the following:

Composite beams

Beam and slab

Composite beams can be configured in several ways based on rolled steel sections, as shown in Figure 5. The simplest and most common form is as shown in Figure 5a. It is generally used for spans between 6 and 16 m, but can be used to span over 20 m. When necessary, this type of beam can be protected against fire using an intumescent coating, sprayed fire protection, or even boxed in using fireproof boards.

The conception of this type of composite beam is substantially linked to the form of reinforced concrete slab that is adopted. The slab is generally cast in-situ using profiled, galvanised metal decking as permanent formwork, or sometimes using thin concrete precast slabs as the formwork. Although the resistance of the composite beam is relatively independent of the manner of forming the slab, the beam deflection under the dead weight of the concrete is significantly affected by the construction sequence. In order to eliminate, or at least reduce, dead load deflections it is possible to:

- prop the beam during casting of the slab;
- after hardening of the concrete and removal of the props the dead load of the concrete plus steel is supported by the composite beam section. Propping is essential when a system as shown in Figure 5b, using stub girders, is adopted.
- precamber the steel section during fabrication, by an amount calculated to compensate for deflections during concreting of the slab. The precamber may be applied to the steel section either when cold, using a press, or by controlled local application of heat.
- provide some continuity of the beam at the end supports.

Car park, Helmond (NL)
When propping is adopted the loads in the props may be quite large. The designer/builder should therefore think carefully before using props in a multi-storey building, and must consider the rigidity and strength of any lower levels that are used to support the props. The use of propping becomes less economical when there are significant inter-storey heights.

Unless special measures are taken to control deflections during concreting, the accuracy that can be practically achieved using precambering is of the order of several centimetres. However, this should still allow accurate positioning of the formwork, and the correspondence of holes in adjacent frame members to be lined up so that connections can be made. It is necessary to avoid any harmful or uncontrolled rotation of the secondary beam connections due to the movement of a precambered primary beam during concreting.

It is clearly necessary to verify that the lateral torsional buckling resistance of the steel beam is sufficient to support the loads applied during concreting, and provide lateral restraint when necessary. Correctly anchored profiled metal decking often provides sufficient restraint.

Propping of the decking or precast slabs is needed when they cannot support the weight of wet concrete and the other construction loads (for example the weight of the operatives) imposed during concreting. This is often the case for spans in excess of 2.5 to 3.0 m. It should also be remembered that the weight of any additional concrete placed due to deformation of the steel beam and metal decking during concreting (an effect known as ponding) may not always be negligible.

One implication of the various points discussed above is that the designer should carefully consider how the beams and slabs will be constructed, and should clearly state the assumptions made during the design on the appropriate contract documentation.
Shear connection in composite beams

The mechanical shear connection between the slab and steel beam is essential for achieving structural interaction between the two components under bending. The most common form of connection comprises welded headed shear studs (Figure 6a), which are attached to the steel beam using a special welding ‘gun’. Uniform spacing is desirable to facilitate the correct positioning of the studs, and so that their positioning can be checked visually. Several other types of connector exist as an alternative to welded studs, including angles fixed using shot-fired pins (Figure 6b). Although these offer a reduced resistance, they avoid
the need for welding and may therefore be appropriate in certain circumstances. Various other types of connector may be used, as shown in Figure 6.

The types of connector shown in Figures 6a and 6b are relatively flexible, whereas the other types shown in the Figure are rigid. The difference is significant, because rigid connectors do not allow redistribution of the longitudinal shear force amongst themselves. The ability of the more flexible connectors, which are known as “ductile”, to redistribute the shear allows the use of partial shear connection for beams in buildings.

When possible, shear studs are welded to the steel beams in the fabrication shop. This can be done when the decking is not continuous over the beams, or when precast slabs are used. It should be noted that it is not necessary to protect either the studs or any surfaces of the steel beam in contact with the concrete against paint, given that the design method takes no account of bond between the concrete and steel.

For the thicknesses of decking (and galvanising) generally used it is possible to weld the studs to the beams on site using what is known as “through-deck welding”. Certain precautions should be taken with regard to the conditions of contact between the various components; excess humidity, unclean surfaces, or the presence of paint (which can be avoided by applying masking tape to the beam before painting) can all affect the integrity of the weld. Despite these restrictions, through-deck welding of the studs on site, using appropriate welding equipment, is widespread in practice.

On site, as in the fabrication shop, a simple bending check applied to some of the welded studs allows rapid assessment of the weld quality.

Occasionally, in order to avoid site welding of the studs, the steel decking is delivered to site with circular holes cut through it at the shop-welded stud positions. Clearly this requires the production of very precise drawings, or other appropriate information, and a number of corrections on site are inevitable.
Design of composite beams

Resistance at the ultimate limit state

According to Eurocode 4 the resistance of a composite beam should be verified at the ultimate limit state for any cross section that could be critical. This is true whether the beam is simply supported or continuous over several supports. Other than for certain relatively complex cases associated with continuity and moment redistribution (which are also covered by the standard), in general this verification amounts to no more than a simple comparison of the plastic resistance moment and the applied moment at one or two critical sections.

For the common case of a beam that is simply supported at its extremities and subjected to uniformly distributed loading, it is sufficient to ensure that the ponderated applied moment $M_{sd}$ is less than the ultimate resistance moment $M_{pl,Rd}$. This resistance is calculated according to the traditional rectangular stress block method, as shown in Figure 7. No account is taken of the concrete within the depth of the decking profile, or within the depth of the dry joint when precast concrete slabs are used as permanent formwork.

Vertical shear forces are assumed to be resisted uniquely by the web of the steel section, the ultimate shear resistance of which must be greater than the ponderated applied shear. It is necessary to consider interaction between bending and vertical shear above the supports of continuous beams, or beneath concentrated loads, when the applied shear is greater than 50% of the web capacity.

![Figure 7](image.png)

European parliament, Luxembourg (L)
Serviceability limit states

To ensure adequate behaviour in service it is necessary to verify the beam deflections, the cracking of the concrete at the supports, and the natural frequency of the beam. The designer should also verify that the stresses induced in the section under service loading do not cause any local plastification, which would invalidate any deflections calculated using elastic theory.

The magnitude of the deflections depends on the construction sequence. Dead loads may be supported by either the composite section, or the more flexible bare steel section, depending on whether or not the beams and slabs are propped during construction. The magnitude of any precamber to be applied during fabrication will depend on the calculated dead load deflections. The rigidity of a composite member may be calculated according to classic elastic principles; the effective section of the slab is transformed into an equivalent steel section using an appropriate modular ratio for the two materials. The designer must take into account creep of the concrete under long term loading (self weight etc), shrinkage of the concrete, and possibly the influence of partial shear connection.

Control of crack widths is necessary where the concrete will be subject to tension, for example at the internal supports of a continuous beam. This dictates the adoption of a certain minimum area of longitudinal reinforcement in the slab. In no case should the percentage of reinforcement drop below either 0.4% or 0.2%, depending on whether or not the slab is propped during construction.

For most cases when the slab will be subject to normal “people traffic” design standards recommend that the rigidity of the floor is such that its natural frequency is greater than 3 Hz. This check is relatively simple, using a formula which considers the span, the mass, and the rigidity (EI) of the section.

Shear connection

Shear connectors and transverse reinforcement placed in the slab above the beam transfer the longitudinal shear force between the steel and concrete. Any adhesion between the steel and concrete is not taken into consideration.

The headed shear studs normally used are ductile, which means that they have sufficient deformation capacity to enable the adoption of partial shear connection. The term “partial shear connection” refers to situations in which the resistance of the composite beam is governed by the strength of the shear connection. In other words, it is possible to reduce the number of shear connectors (within certain limits) when full shear connection would lead to an excess in beam capacity, as it is often the case.
Partially encased composite beams

The fire resistance of a traditional composite beam can be improved considerably by infilling the areas between the steel flanges with reinforced concrete (Figure 8). This process is, however, only possible for beam depths greater than 180 to 200 mm, which allow the inclusion of appropriate reinforcement (with sufficient cover) in the concrete. Clearly, the weight of the structure increases due to the additional concrete, which must be allowed for in the design. However, this additional weight is generally compensated by the increased rigidity of the beam, and so does not normally result in an increase in the size of steel section required, when the beam is wide enough to accept the concrete.

Concrete filling takes place on the ground before erection of the beam. The steel beam is laid on well aligned, rigid supports, which are sufficiently closely spaced to avoid deformation of the steel section under the weight of the concrete. Prefabricated reinforcement cages are dropped into the voids between the flanges, positioned, and held in place to ensure that adequate concrete cover is achieved. If possible the concrete is poured directly from the mixer truck into the prepared beam, which can be turned over after only a very short period to allow concreting of the opposing chamber.

The process of concreting on the ground requires delivery of the finished steel members approximately one week before they are due for erection. It also requires an area that can be serviced by a crane; this area may be either on site or perhaps in a nearby workshop or similar depot.

The main longitudinal reinforcing bars, which are placed in the concrete to enhance the fire resistance of the composite section, are complemented by other secondary bars. In particular, stirrups are needed to avoid spalling of the concrete in a fire and a resulting premature heating of the core of the section at one precise location.

The concrete infilling between the flanges must be mechanically anchored to the web of the steel section so that thermal stresses do
not cause any break and fall off of the latter. Several solutions are proposed in Eurocode 4; headed studs can be welded to the web, or reinforcing bars that penetrate the web may be added, or stirrups may be welded to the web (as discussed later).

In theory the steel surfaces in contact with the concrete are not painted, with the possible exception of a 3 cm return towards the interior of the flanges. It should be noted however that the presence of paint on the web and studs has no determinant influence on the behaviour of the beam because, as already said, any natural adhesion between the steel and concrete is not considered in the design method.
Design of partially encased beams

Design for normal load conditions

Partially encased beams are often designed for normal load conditions as traditional composite beams. The reinforced concrete between the flanges is taken into account as a dead load, but is completely neglected when determining the resistance of the section, and even when calculating deflections.

Although such simplified assumptions are clearly conservative, the basic version of Eurocode 4 gives no alternative rules specifically for partially encased beams. The section of the reinforcing bars needed is determined by fire resistance requirements rather than normal load conditions.

In reality, the increase in rigidity of the section due to the presence of the concrete and reinforcement may be considerable. Starting at several percent for the smallest practical beams, the increase in rigidity may exceed 20% for the largest beams in their final condition.

Unfortunately, an accurate calculation of the rigidity for use in deflection calculations is rather laborious. It is necessary to carry out several elastic analyses to cover the various stages of construction and the load application sequence. The evolution of the section that is acting structurally, and of the concrete properties in function of the time, must all be considered.

If the presence of the reinforced concrete infill has not been taken into account for when determining the second moment of area (I), the designer should be aware that the actual deflections will be less than those predicted. This will be true in both the final state and intermediate states during construction, and can have a significant influence on the magnitude of any precamber (when specified). The increased rigidity will also be significant at any other stage when it is necessary to predict the deflections, for example when determining the capacity for adjustment needed at interfaces with prefabricated elements such as staircases or cladding panels.

Eurocode 4 (ENV 1994-1-1) Annex G

Tests have shown that the presence of concrete between the steel flanges not only increases the rigidity of a beam, but also its ultimate bending moment resistance and its vertical shear capacity.

Annex G of Eurocode 4 proposes supplementary rules which take into account the concrete between the flanges under service conditions. The rules are applicable whether or not there is a participating slab.

The annex proposes a simplified method for calculating the second moment of area of the beam (I), ignoring any concrete in tension.

Normal, relatively weak concrete (C20) is generally used to infill between the flanges.
Verification of the fire resistance for partially encased beams

Resistance to an ISO standard fire

Eurocode 4 Part 1-2 proposes two methods for determining the resistance of a partially encased composite beam subject to a standard ISO fire. The first of these, the “tabular” method, requires some resistance calculations in conjunction with interpolation of tabulated values. This method is very conservative, and predicts very high values for the areas of reinforcement required. Ideally, it should not be used in preference to the second, “simple calculation”, method.

It is possible to measure the progressive heating through a section during a fire test. Zones of different temperature can be defined for each material, in which the loss of resistance due to the elevated temperature can be evaluated.

The simple calculation method for predicting fire resistance considers the ultimate moment resistance of the section, which is calculated by dividing the section into different zones. The material properties for each zone are modified using reduction factors, which depend on the average temperature in the zone. These temperatures are determined by considering the section to be exposed to an ISO fire for the required fire resistance period.

The method is equally applicable for both positive moments (Figure 9) and negative moments at supports (Figure 10). Unfortunately, even though simple, hand calculations using this method still take some time. However, the method has been programmed, and software is available on request from the Technical Assistance department at Arcelor Sections Commercial.
Fire resistance is assured if the moment resistance calculated for the time required (with material strengths reduced to reflect the zone temperatures at that time) is greater than the moment applied by the combination of loads appropriate for the accidental fire condition.

Eurocode 4 Part 1.2 allows redistribution of the moments in a beam under certain conditions, even if the beam has been assumed to be simply supported under normal service loading. In order to comply with reinforced concrete design standards it is always necessary to have at least a minimum level of continuity reinforcement (anti-crack reinforcement). This reinforcement will remain cold during a fire, and limit the rotation capacity of the composite beam. In order to benefit from a redistribution of moments it is necessary to ensure that the gap at the ends of the beam satisfies a defined limit (10 to 15 mm according to the situation, which may well be achieved anyway).

In practice some moment redistribution is not needed in the majority of cases for simple beams. A minimum of two 12 to 20 mm bars (see Clause 5.3.2 of ENV 1994-1-1) placed at the bottom of the infill concrete is generally sufficient to achieve 90 or 120 minutes fire resistance for floor beams.
Composite columns

The types of composite column illustrated in Figure 11 are the most common, being of either square or rectangular cross-section. They are compared below. Sections that are completely encased in concrete may also contain two steel members placed side by side, with sufficient gap between these members to allow correct filling with concrete.

Circular sections are also used, primarily to meet architectural requirements. They may be formed either using traditional formwork (Figure 12), or by placing the steel member inside a metallic tube (Figure 13). The former type is effectively a variation on the more common completely encased rectangular section, with the same advantages and disadvantages as described below.

![Composite columns](image1)

**Figure 11**
Common forms of composite columns

- Composite column filled with concrete
- Composite column encased in concrete

**Figure 12**
Circular composite H column encased in concrete

**Figure 13**
H column encased in concrete inside a metallic tube

Bank Bruxelles Lambert, Brussels (B)

Office building Winthertur, Barcelona (E)
So-called cruciform cross-columns (Figure 14) comprise two steel sections, sometimes identical, one of which is cut into two Ts. The Ts are welded to the web of the other steel girder. This type of column is used when the buckling length is substantial in both axes. The steel members used for this type of composite section are generally considerably deeper than they are wide, with a depth greater than 400 mm, or even sometimes 500 mm. Concreting on the ground prior to erection is possible, but requires four operations and a fairly complex procedure to fix the reinforcement.

Other types of section that combine two steel members may also be used (Figure 15). The main steel girder is reinforced in each the area between the flanges by one or more smaller steel sections. The latter are typically H sections, or thick flanged T sections, which are welded to the web of the main member. The provision of this quantity of steel within the body of the concrete clearly leads to a composite column with excellent fire resistance capabilities.

It is worth noting that the list of composite column section types described above is not exhaustive, and other types can certainly be imagined.
Design of composite columns

Eurocode 4 proposes a method for the design of composite columns at the ultimate limit state. The apparent complexity of this method is in fact relatively superficial, and it can be easily programmed. The method may be used for any of the typical types of section described above when loading is primarily axial. Additional bending moments may be present.

Axial compression

The designer must verify that the axial load in service, increased by using the appropriate load factors, is less than the resistance of the composite member. The buckling resistance of the member is a function of the plastic compression load, suitably reduced using a coefficient that reflects the slenderness of the member (Figure 16).
Axial compression and uniaxial bending

When the axial load is accompanied by moments about one axis it is necessary to determine the N-M interaction curve for the section bent about that axis (Figure 17). The designer must then verify that at the ultimate limit state the ponderated moment does not exceed the moment resistance limit, which generally increases as the level of axial load decreases (shaded part of the diagram). The interaction curve can be determined by calculating numerous successive points, considering the movement of the plastic neutral axis across the section. Alternatively, the curve can be determined relatively easily by establishing several critical points using the procedures given in Eurocode 4.

Sony Center Potsdamerplatz, Berlin (D)
Axial compression and biaxial bending

In the case of biaxial bending an N-M interaction curve must be determined for bending about both axes. Corresponding points on the curves in the y-y and z-z planes are joined by a straight line which defines, along with the axes, a surface inside which the factored moments about the two axes must remain (Figure 18).

The reduction coefficient to allow for buckling is applied for the axis that is considered to be critical, which in theory means that each axis must be checked successively.

Figure 18
CALCULATION OF BIAXIAL COMPRESSION AND BENDING

a. Plane in which a failure is supposed possible in taking into account the buckling
b. Plane without taking into account the buckling
c. Interaction diagram showing the bending resistance
Shear connection in composite columns

Region of load introduction

Loads are only rarely introduced into the column via a header plate which distributes them correctly between the steel and concrete components. Normally, the floor beams are attached directly to the steel component of the column, so that part of the load must be transferred into the other component (the reinforced concrete). This necessitates provision of adequate transfer studs in a transfer zone as described below (Figure 19).
Longitudinal shear resistance

Shear transfer is achieved either by frictional or adherence stresses between the contact surfaces, or by mechanical shear connection which prevents any significant slip between the steel and concrete.

When relying on friction and adherence it is clearly necessary to avoid any painting of the steel surfaces. These types of shear transfer may be sufficient when the steel component is fully encased.

Partially encased sections, with concrete only present between the steel flanges, must in all cases adopt a certain number of mechanical shear connectors fixed to the web of the steel member. Without such connection the concrete may spall away from the web under the action of thermal stresses during a fire, and indeed fall off the steel member. The mechanical shear connection required may be achieved in a number of ways, either using headed studs welded to the web, or welding the stirrups to the web, or perhaps using bars passing through holes in the web and linking the stirrups on either side (Figure 20).
Fire resistance of composite columns

Fully encased sections

When the steel member is completely encased in concrete a composite column possesses a very high resistance to fire. Eurocode 4 Part 1.2 proposes a simple table (Figure 21) which allows the fire resistance of a column to be verified for most practical cases, without the need for calculations. If necessary, the documents referenced earlier provide any additional information that may be required to undertake a more accurate verification.

Partially encased sections

The fire resistance of partially encased sections is less than that of fully encased sections, because some of the steel surfaces remain exposed to the fire.

Figure 22 shows the general behaviour of a structural element subjected to a standard ISO fire. Starting with the failure load at room temperature, fire tests under progressively diminishing loads demonstrate ever increasing fire resistance capabilities. A curve can be developed for any structural element; the rate at which resistance falls in a fire is greatest when the materials used are the most sensitive to fire.

For a level of load corresponding to the combination of actions under the fire condition, this resistance curve can be used to predict...
the fire resistance period for the element in question. The "natural" fire resistance period for partially encased composite columns is generally in excess of 60 minutes.

When the natural fire resistance of a composite column is less than that required it is necessary to choose another section which has a higher resistance curve. The curve must demonstrate that the required fire resistance (90 or 120 minutes) can be achieved under the proposed level of loading in fire.

The procedure described above generally leads to overdesign of the column under normal service conditions. However, the excess cost remains moderate for a fire resistance of 90 minutes (R90) provided a logical choice of a section with thin flanges is made. The required overdesign is preferably achieved by adopting one or more of the following measures:
- increase the design grade of the steel
- increase the grade of concrete
- increase the amount of reinforcement

If the last of these measures is adopted the maximum percentage of reinforcement can exceed the maximum value of 4% specified in the relevant standards, which is appropriate for normal service conditions. Up to 6% may be adopted in order to achieve the necessary fire resistance.

For most current applications in buildings increasing the material strengths or the amount of reinforcement is not sufficient to achieve two hours fire resistance (R120), and it is also necessary to increase the size of the steel member.

Verification of the fire resistance of partially encased columns

Even when the simplified tables proposed in ENV 1994-1-2 are used, some calculation is necessary. It should also be noted that the simplified design method proposed in Annex F is more commonly used, having been widely disseminated in the form of software.

As for partially encased beams, this method considers contours of temperature within the body of a section after 30, 60, 90 and 120 minutes of exposure to an ISO standard fire (Figure 23). The section is divided into zones (Figure 24) in which the mechanical properties of the different constituent materials vary as a function of the average temperature in the zone. The collapse load is then calculated using a process which is essentially the same as that used to calculate the collapse load under normal service conditions, but considering reduced material strengths.

Figure 23: isotherms in a composite column subjected to an ISO fire of 90 minutes

Figure 24: reduced section of a composite column for the calculation of the fire resistance
The fire condition may govern design for this type of column, particularly when fire resistance requirements exceed 1 hour. The measures described above to “overdesign” the section can be adopted. A reduced buckling length (Figure 25) can generally be adopted provided the floor slabs ensure compartmentalisation of the fire.

**Cruciform sections and sections with reinforced steel members**

There is no simplified direct method of design for these types of section. The designer must use numerical simulation (CEFICOS), or interpolate values from reference tables [2] (that are based on numerical simulations) using “engineering judgment” to draw analogies.
**Construction details**

The two basic components of a composite member - the steel and the reinforced concrete - must clearly respect any application rules relevant to their individual domains. However, composite construction owes its success above all to the levels of fire resistance that can be achieved, and certain construction details are important in order to assure this fire resistance.

**Positioning of the reinforcement**

Traditional concrete rules must be respected when considering resistance and cracking requirements under normal service conditions. In particular, rules govern minimum diameters for bars and stirrups, the spacing of stirrups, the configuration of the stirrups as a function of the spacing between the reinforcing bars in compression, concrete cover requirements etc.

An additional risk must be considered for the fire condition - local spalling of the concrete may occur. Even if this appears to only cause a slight reduction in the area of the effective section, it is an important phenomenon because it leads to an acceleration in thermal penetration, which may lead to local weakening of the member.

As a consequence of the phenomenon described above, the part of Eurocode 4 (ENV 1994-1-2) dealing with fire proposes several additional rules for reinforcement detailing (Figure 26). In particular, for beams the spacing of the stirrups must not exceed 250 mm, and mesh must be added to the exposed faces of the infill concrete, with cover not exceeding 35 mm.
Mechanical shear connection to the web

Mechanical shear connection to the web can be achieved in different ways, as previously discussed; by welded studs, by welded stirrups, or by passing bars through the web to link the reinforcement cages. For beams with concrete infilling the maximum spacing of the shear connectors, and their dimensions, must respect certain design limits (Figure 26). The same is true for columns with concrete infilling between the steel flanges, unless closer spacing is required to achieve the necessary shear transfer under normal service conditions.

Continuous reinforcement in partially encased columns

The density of continuous reinforcement required in connection regions may make it difficult to accommodate the necessary erection bolts. This problem is eased when the steel section is locally strong enough to resist the applied loading, perhaps with the addition of strengthening plates. In such cases the reinforcement may be interrupted in these regions, provided that the necessary force can be transferred from the concrete to the steel components, and in the other way below the beam.

Web openings in beams

Openings in the webs of partially encased beams are achieved in exactly the same way as for bare steel sections. The openings are generally reinforced using a short length of tube, or by a surrounding “frame” made from plates. This local web reinforcement also performs the role of formwork during the placing of concrete between the flanges (photo).

Provided that the longitudinal reinforcing bars in the concrete remain suitably encased, the presence of web openings only influences the shear resistance of the section (due to increased heat penetration into the web). However, the excess in vertical shear capacity of an encased web during a fire is often considerable. An estimate of the reduced capacity next to an opening may be made considering the reduced yield strength of the web steel using a simplified method proposed in Eurocode 4 (ENV 1994-1-2). In principle this method is conservative, because radiation effects are in fact reduced when an opening is present.

Clearly it is also possible to “seal” regions were pipes pass through the beams using fire insulation after fixing of the pipework.
# Choice of column type

Comparison of the two most common solutions

<table>
<thead>
<tr>
<th>Fully encased columns</th>
<th>Partially encased columns</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Figure 27</em> Width ≥ 240 mm</td>
<td></td>
</tr>
<tr>
<td>- Perimeter formwork is required.</td>
<td>- In theory no formwork is required (unless there are difficulties associated with lifting, or specific requirements for a very smooth or special textured finish on the concrete).</td>
</tr>
<tr>
<td>- Normally concreted after erection.</td>
<td>- Concreted horizontally, on the ground, before erection.</td>
</tr>
<tr>
<td>- No exposed steel surfaces.</td>
<td>- Steel surfaces remain exposed.</td>
</tr>
<tr>
<td>- Preferably based on steel sections with thick flanges (HEM, HEB, HD).</td>
<td>- Preferably based on steel sections with thin flanges, in order to limit the volume of steel directly exposed to the fire (HEAA, HEA, HP).</td>
</tr>
<tr>
<td>- Reinforcement must be fixed around the steel member in its final, erected position.</td>
<td>- Reinforcement cages can be prefabricated, and rapidly positioned.</td>
</tr>
<tr>
<td>- Relatively small percentages of reinforcement are used. Bars should preferably only be positioned at the corners of the section.</td>
<td>- To achieve fire resistance in excess of one hour it is advisable to adopt the maximum allowable percentage of reinforcement (6% in the fire condition, of which only 4% is taken into account for design under normal service conditions).</td>
</tr>
<tr>
<td>- The steel member should not be painted.</td>
<td>- Paint is normally applied to the exposed surfaces of the steel flanges, generally for purely aesthetic reasons.</td>
</tr>
<tr>
<td>- A small number of mechanical shear connectors are usually sufficient. They are primarily needed in regions of load introduction.</td>
<td>- Mechanical shear connectors (studs, or similar) are normally needed along the whole length of the column, to prevent the concrete falling off during a fire.</td>
</tr>
<tr>
<td>- Structural fire resistance is inherently very high.</td>
<td>- Fire resistance, with possible “over-design” in excess of 60 minutes.</td>
</tr>
<tr>
<td>- On site the full structural resistance of the composite column is only achieved following encasement.</td>
<td>- Because of the over-design needed to satisfy fire resistance requirements, there is often considerable over-strength during the construction phase.</td>
</tr>
</tbody>
</table>
Pre-installed columns

Basic principle

“Pre-installed”, or “plunge” columns are frequently used in urban locations because they eliminate a number of problems associated with deep excavations on sites adjacent to existing buildings. Each column, of a height equal to the depth of the basement to be constructed, is lowered into a bored hole. The foot of the column is then embedded in concrete, which is poured into the bottom of the shaft, either at, or below, the final foundation level. A slab is then used to link the individual columns, and the building superstructure is erected on top of this slab. It is then possible to excavate below the slab to form successive basement levels whilst the superstructure is being erected.

Construction method

To prevent buckling of the columns during construction, the bored holes are filled with gravel, amongst which may be interspersed weak concrete plugs at predefined locations.

The position of the heads of the columns, in both plan and level, can be controlled relatively accurately on site. It is more difficult to adjust the verticality of the columns in their shafts, and variations on site may be more substantial than with more “conventional” construction methods. It is, however, possible to reduce problems of non-verticality using horizontal hydraulic pistons which can be placed near the base of the columns and controlled from the surface.
Steel members are ideal for this type of application because they are light and easy to manipulate. Unfortunately, fire resistance requirements for basement locations (typically R90 or R120) do not permit the use of steel members without additional fire protection, which is not always welcome in locations that are mostly used for parking. Composite columns are often preferred because of their compactness, and inherent fire resistance. The choice between a fully or a partially encased composite section depends on the considerations discussed below.

**Fully encased sections**

Although a fully encased section may be completely prefabricated before dropping it into its bored hole, it is more common to utilise the bare steel section during the construction stage (when the loads are less than in the final service condition). The steel member is then progressively encased in concrete, to form the composite section one level at a time as the excavation progresses and floor slabs are formed.

For this type of application composite columns offer the following benefits and disadvantages:

- The member to be lifted is relatively light and robust, and need not be painted.
- In principle it is not necessary to over-design the column relative to the final service loads. However, it is necessary to verify that the member is adequate for each stage during construction. Excessive slenderness, or delayed encasement of the lowest sections of the column, may necessitate the use of a larger steel section. In some instances the bare steel section may be sufficient for the service loading, in which case the concrete encasement serves merely as fire protection. It is clearly necessary to evaluate the speed of construction both below and above the ground in order to determine the different load combinations and corresponding buckling lengths during construction.
- The section is very compact, and can be formed in different shapes (for example square or circular). However, excessively wide members (>40 cm) may prove difficult to arrange efficiently between parking spaces.
- Formwork can generally be reused several times. By placing the formwork eccentric to the steel member it may be possible to compensate for any slight misalignment (up to several centimetres) of the latter during its placing.
- Fixing of the formwork and reinforcement, and placing of the concrete, are not particularly easy operations. In particular, it is necessary to leave openings or ducts in the slabs to permit subsequent placing of concrete in the lower level of column. Correct filling and local load transfer capacity must be ensured. The need to achieve lapping of the longitudinal reinforcing bars means that they must extend into the ground below the level being concreted, so that they act as “starter bars” at the top of the following level.
Floors

Steel or composite columns used in this type of construction are compatible with all types of floor structure. When it is possible to use beams these may also be composite, either partially encased or fire protected. It is easy to provide connection pieces, or box-outs to allow the subsequent passage of continuous reinforcement. When necessary, such connection details can be provided with a means of accommodating any differences in the final levels of the columns that may occur on site.

The framework formed by the floor beams is used to stabilise the basement walls during construction. Partial encasement of the floor beams ensures they have a significant buckling resistance.

It is also worth remembering that the use of galvanised decking as permanent formwork for the floor slabs practically eliminates the need for cranage during construction of the basement, and that the decking is an ideal complement to the steel or composite beams.

Partially encased sections

Partially encased sections are concreted at ground level prior to being lowered into bored holes. They have the following characteristics:

- The composite members are relatively heavy.
- It is necessary to identify an area big enough for concreting to take place. If necessary, substantial lengths of column can be achieved using several pieces, which are bolted together as needed during the lowering operation. The joints are normally located within the basement floor slabs, and are therefore invisible in the final condition. This means that the joints can be made using cover plates and external bolts. Lateral buckling restraints should be placed in the shaft, according to the spacing of the joints.
- Design for fire resistance often leads to an over-strength under normal service conditions. However, this can be exploited during construction to increase the allowable buckling length and possibly permit excavating of several levels at a time.
- Because the width of the steel sections is typically only approximately 300 to 400 mm, this type of composite column may offer a greater usable floor area between parking spaces than alternative solutions.
Connections

The connections between composite members are almost always formed between the steel components, and are designed and detailed according to the usual rules for steel construction.

Their conception is driven by the philosophy of placing the bolts, or lengths of weld, in positions where they are sheltered from direct heat in a fire. Clearly it is also necessary to maintain sufficient access for bolting or welding during erection of the frame, and to avoid the need for additional fire protection as much as possible.

As an example, bolts placed within the depth of the concrete slab (plus any finishing screed) will be buried in the mass of concrete and therefore protected without any need for supplementary fire protection in the final condition.

The general considerations described above have led to the development of several relatively common basic types of composite connection, as described below.

Beam to column connections

- using a bracket (figure 28): the bracket may be placed either below or within the depth of the beam. Additional bolts are added within the depth of the slab to aid erection. There is no need to fire protect the bracket provided the upper weld, which is not exposed directly to the fire, is reinforced, and that the bracket is sufficiently thick. Alternatively, fire protection can be avoided by adding shear studs to the bracket, and passing these through holes drilled in the column flange so that they can be embedded in the column concrete.
- **using a web plate** (figure 29): the bolted connection must be fire protected after erection, either using specific fire protection materials or by embedding the connection in concrete. The latter operation is facilitated by oblique cutting of the top flange of the beam, to allow filling of the cavity during casting of the slab.

- **using an end plate with “upper” bolts** (figure 30): when possible the bolts should be concentrated within the depth of the slab, or at least a sufficient number of sufficient diameter bolts to resist the combination of loads applied in the accidental fire condition. A spacer plate may be used to ensure that the connection is free to rotate.
- **Using direct support from the column** (figure 31): this type of detail has been used with large prefabricated columns, which are interrupted at each floor level. In order to transfer the loads it is necessary to include thick capping plates and various other large steel components locally within the slab.

Large prefabricated piles (as found in the basements of multi-storey buildings) can be made with web openings at each floor level to accommodate the floor beams. It is possible to include a means of adjusting the level of such openings to accommodate any variations on site (figure 32).
Beam to beam connections

- **using direct support** (figure 33): this very simple type of detail results in a relatively deep floor, which does however provide good flexibility when it comes to the layout of the services etc. A continuity plate may be welded to the flanges of the secondary beams after erection. Bolts used to aid erection can be left in place, without fire protection. The upper flanges of the primary beams, which are not connected to the slab, may be provided with an insulating plate to increase their fire resistance and thereby reduce the area of reinforcement required in the composite section.

- **using a web plate** (figure 34): the ends of the secondary beams, which are left free from concrete during prefabrication, are attached to plates which protrude beyond the concrete encasement of the primaries. These plates do not interfere with the reinforcement in the primary beams provided they do not extend too far towards the lower flange; the upper bars used to facilitate prefabrication of the reinforcement cages can simply be cut at these locations during placing of the cages. As for beam to column connections of this type, it is necessary to fill-in with concrete, or otherwise protect, the region around the bolts after erection.
- **using a bracket** (figure 35): as for the beam to column detail, it is possible to make a beam to beam connection using a bracket, with an upper fixing to aid erection. However, if the bracket is placed too low in the primary beam it may interfere with the main reinforcement, which should then be installed in the workshop of the steel contractor.

*Figure 35: Beam to beam connection using brackets*
- **using a nib on the upper flange** (figure 36): a thick steel nib may be welded to the upper flange of the secondary beam. This simply rests on the primary, with a bolt used for location during erection. This very common detail allows all the members to be completely filled with concrete, and does not hinder in any way the reinforcement in the members.

![Figure 36](image)

Preconcreted beam using a nib on the upper flange before erection

Beam to beam connection of preconcreted beams using a nib on the upper flange
Frame stability

Diaphragm action of the slabs

The monolithic nature, and resulting in-plane rigidity of the concrete floor slabs means that they can be used to transfer horizontal loads to the vertical members that provide frame stability. It is however necessary to provide other ways of maintaining the stability of the structure during construction, before the concrete achieves sufficient strength.

Vertical bracing

Although composite members can be used for diagonal bracing, the nodal connections are normally rather complicated. There are currently several solutions to the problem of providing vertical bracing:
- bracing is configured so that it may be left unprotected against fire because there is at least one bracing system outside the fire compartment.
- the bracing is placed behind a wall, which protects it from any fire.
- simple steel bracing is embedded within a concrete wall following erection, the wall being cast between and around the columns.
- the frame is stabilised by attachment to a concrete element (such as a lift shaft or stair case), or to nodes that are unaffected by fire. Clearly any such elements must be in place at the time of beginning erection of the steel frame.
Structural models

The structural model applicable under normal service conditions may evolve for the fire condition, depending on the construction details used. Several examples are considered below:

- A beam which is assumed to be simply supported in normal service may be allowed to benefit from some assumed continuity during a fire. The presence of continuity reinforcement in the slab above the beam supports will prevent excess rotation of the beams in a fire.

- When the lower bolts are not fire protected, connections that are assumed to be rigid in normal service may either remain rigid or tend towards pinned behaviour during a fire, depending on the sense of the applied moment. It is clearly necessary that the protected upper bolts are sufficiently strong to transfer the appropriate loads to the column in the accidental fire condition.

- When the slab is relatively thick (for example to achieve the required acoustic performance) and the secondary beams are at relatively close centres (for example to avoid propping during construction) it is possible to consider fire protecting only one joist in two. It is then necessary to check that the slab can achieve a *double span* under fire loading, and provide the necessary reinforcement. The protected joist must also be checked for the fire condition, considering an appropriate loaded area and load combination for this accidental condition.

In summary, it is necessary to verify the resistance of the structure for both normal service and fire, considering not only different loads appropriate to each condition, but also potentially different structural models.

Expansion joints

Notwithstanding certain requirements for the slab reinforcement being satisfied, buildings with a surface area in excess of 6000 m², and up to 120 m in length, have been built using composite construction without any expansion joints. It is however more common to respect the usual limits for steel structures when considering the frequency of expansion joints in a composite beam and column frame. Intermediate joints may be necessary in the reinforced concrete floor slabs according to concrete code requirements.
Structural integrity

The structural integrity and robustness of a building frame are fundamental requirements which are part of the basic philosophy embodied in the Eurocodes. According to this philosophy, an accidental load (explosion, impact etc) must not lead to disproportionate damage of a structure.

One of the measures needed in order to ensure appropriate integrity is to tie together the frame members horizontally. All the connections are therefore required to possess at least a minimum resistance to horizontal tensile forces. In some countries, such as the UK, the levels of load needed to satisfy this requirement have been quantified in an annex to the National Application Document for the Eurocode.

A steel frame normally responds very well to this fundamental requirement for integrity, because of the inherent tensile strength of the frame members and of traditional connections. For composite construction it may be necessary to verify the suitability of some types of bracketed connection. In practice the number and size of the bolts used to aid erection should be kept at a reasonable level, or low tensile resistance of some components can be compensated by detailing additional bars that are suitably anchored in the slab.
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