COMPOSITE SLABS AND BEAMS USING STEEL DECKING:

GOOD PRACTICE FOR DESIGN AND CONSTRUCTION

SCI
Steel Knowledge

COMPOSITE SLABS AND BEAMS USING STEEL DECKING:

GOOD PRACTICE FOR DESIGN AND CONSTRUCTION

SCI (The Steel Construction Institute) is the leading, independent provider of technical expertise and disseminator of best practice to the steel construction sector. We work in partnership with clients, members and industry peers to help build businesses and provide competitive advantage through the commercial application of our knowledge. We are committed to offering and promoting sustainable and environmentally responsible solutions.

Our service spans the following areas:

Membership
Individual & corporate membership

Advice
Members advisory service

Information
Publications
Education
Events & training

Consultancy
Development
Product development
Engineering support
Sustainability

Assessment
SCI Assessment
SCI Product Certification

Specification
Websites
Engineering software

Publication Number: **SCI P300, 3rd Ed.**

ISBN: 978-1 85942-252-6

British Library Cataloguing in Publication Data.

A catalogue record for this book is available from the British Library.

Published by:
SCI, Silwood Park, Ascot,
Berkshire. SL5 7QN UK

T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com

www.steel-sci.com

To report any errors, contact:
publications@steel-sci.com

COMPOSITE SLABS AND BEAMS USING STEEL DECKING:

GOOD PRACTICE FOR DESIGN AND CONSTRUCTION

FOREWORD

Composite construction is popular because it combines structural efficiency with speed of construction to offer an economic solution for a wide range of building types. Applications include commercial, industrial and residential buildings.

This guide covers the design and construction of composite slabs and beams and addresses the good practice aspects of these activities. The practice presented is typical of the current U.K. market, and has developed over time in order to deliver the best results. It is therefore recommended that contractual arrangements for specific projects are aligned with these recommendations. This revision updates the previous MCRMA/SCI guide, which was published in 2000 and updated in 2009. This second update reflects the latest guidance for good practice and gives information on design to the Eurocodes as well as design to BS 5950, including comment on some changes that will be reflected in the so-called Generation 2 Eurocodes.

The standalone Section related to the design and construction of shallow floor construction that was contained in previous editions of this publication has been omitted from this update.

The principal authors of the first two editions of this publication were Dr J W Rackham, Dr G H Couchman, and Dr S J Hicks, and the principal author of this third edition is Mr L A Dougherty (all authors previously or currently from The Steel Construction Institute).

Input to the first two editions was received from the MCRMA decking group. Similar industry input to this third edition has been provided by the BCSA decking group, which includes the following companies. The contributions of the named representatives are gratefully acknowledged:

- Mr J Turner — Structural Metal Decks Ltd.
- Mr C Atkinson — Kingspan Ltd.
- Mr J Millar and Mr R Cresswell — Severfield (NI) Ltd.
- Mr J Nurser and Mr M Norton — Composite Profiles UK Ltd.
- Mr J Tranmer — Prodeck-Fixing Ltd.
- Mr A Sowter — Richard Lees Decking
- Mr M Davies — Tata Steel
- Mr A Cummins — Construction Metal Forming Ltd.
- Mr S Lemmon — Stud-deck services Ltd.
- Mr A Stokoe — MSW Ltd.
- Mr S Hall — Hadley Group.
- Dr A Girao-Coelho and Mr P Walker — British Constructional Steelwork Association

The preparation of this document was funded by SCI. Previous editions were funded and commissioned by the Metal Cladding and Roofing Manufacturers Association (MCRMA).

CONTENTS

EXECUTIVE SUMMARY

This guide covers the design and construction of composite floors, paying particular attention to the good practice aspects. Following a description of the benefits of composite construction and its common applications, the roles and responsibilities of the parties involved in the design and construction process are identified. The requirements for the transfer of information throughout the design and construction process are described.

The design of composite slabs and beams is discussed in relation to both the Eurocodes and BS 5950. Some changes that we anticipate will be introduced in the Generation 2 Eurocode 4 are presented. In addition to general ultimate and serviceability limit state design issues, practical design considerations such as the formation of holes in the slab, support details, fire protection, and attachments to the slab are discussed. Guidance is also given on the acoustic performance of typical composite slabs. Typical construction details are illustrated.

The obligations of designers according to the CDM Regulations are identified and discussed.

INTRODUCTION

Composite slabs consist of profiled steel decking with an in-situ reinforced concrete topping. The decking not only acts as permanent formwork to the concrete, but also provides sufficient shear bond with the concrete so that, when the concrete has gained strength, the two materials act together compositely.

Composite beams are normally hot rolled or fabricated steel sections that act compositely with the slab. The composite interaction is achieved by the attachment of shear connectors to the top flange of the beam. These connectors generally take the form of headed studs. It is standard practice in the UK for the studs to be welded to the beam through the decking (known as 'thru-deck' welding) prior to placing the concrete. The shear connectors provide sufficient longitudinal shear connection between the beam and the concrete so that they act together structurally. Latest developments use alternatives to welded studs so that the slabs and beams can be separated at end of life in demountable systems.

Composite slabs and beams are commonly used in the commercial, industrial, leisure, health and residential building sectors due to the speed of construction and general structural economy that can be achieved. Although most commonly used in steel framed buildings, composite slabs may also be supported off masonry or concrete components. They are also used in conjunction with light steel framing.

A typical example of the decking layout for a composite floor is shown in Figure 1.1. The lines of shear connectors indicate the positions of the composite beams.

Figure 1.1 A typical example of composite floor construction, showing decking placed on a steel frame

courtesy of Kingspan

1.1 Benefits of composite construction

Composite construction has contributed significantly to the dominance of steel frames in the commercial building sector in the UK. The main benefits of composite construction are listed below.

Speed of construction

Bundles of decking can be positioned on the structure by crane and the individual sheets then installed by hand. Using this process, crane time is minimal, and in excess of 400 m^2 of decking can be installed by one team in a day, depending on the shape and size of the building footprint. The use of the decking as a working platform speeds up the construction process for following trades. Minimal reinforcement is required, and large areas of floor can be poured quickly. Floors can be concreted in rapid succession. The use of fibre reinforced concrete can further reduce the programme, as the reinforcement installation period is significantly reduced.

Safe method of construction

The decking can provide a safe working platform and act as a safety 'canopy' to protect workers below from falling objects.

Saving in weight

Composite construction is considerably stiffer and stronger than many other floor systems, so the weight and size of the primary structure can be reduced. Consequently, foundation sizes can also be reduced.

Saving in transport

Decking is light and is delivered in pre-cut lengths that are tightly packed into bundles. Typically, one lorry can transport in excess of 1000 m^2 of decking. Therefore, a smaller number of deliveries are required when compared to other forms of construction.

Structural stability

The decking can act as an effective lateral restraint for the beams, during construction, provided that the ribs run transversally and the decking fixings have been designed to carry the necessary loads and specified accordingly. The decking may also be designed to act as a large floor diaphragm to redistribute wind loads in the construction stage, and the composite slab can act as a diaphragm in the completed structure. The floor construction is robust due to the continuity that can be achieved between the decking, reinforcement, concrete and primary structure.

Shallow construction

The stiffness and bending resistance of composite beams means that shallower floors can be achieved than in non-composite construction. This may lead to smaller storey heights, more room to accommodate services in a limited ceiling to floor zone, or more storeys for the same overall building height. This is especially true for shallow floor forms of composite construction, whereby the beams are integrated within the slab depth.

Sustainability

Steel has the ability to be recycled repeatedly without reducing its inherent properties. It also facilitates re-use of structural components. This makes steel framed composite construction a sustainable solution. ‘Sustainability’ is an increasingly important factor for clients, and 100% of all steel construction products can be either re-used or recycled upon demolition of a building. Recent improvements in steel and concrete technology will enable further significant reductions in the embodied carbon of composite slabs. See Section 4.2.1 for information on low carbon concrete and Section 7 for information on demountable composite construction systems.

Easy installation of services

Cable trays and pipes can be hung from hangers that are attached using ‘dovetail’ recesses rolled into the decking profile, thereby facilitating the installation of services such as electricity, telephone and information technology network cabling. These hangers also allow for convenient installation of false ceilings and ventilation equipment (see Section 4.2.8).

The above advantages often lead to a saving in cost over other systems, as reflected in the market share for composite construction. Cost comparisons are updated quarterly in Building magazine and called the ‘Costing Steelwork’ series (see Steel construction news - SteelConstruction.info and Cost comparison studies - SteelConstruction.info). The cost comparisons show that where steel-concrete composite construction is used, the cost of the steel composite solution is lower than alternatives when a comprehensive view is taken.

1.2 Applications

Composite slabs have traditionally found their greatest application in steel-framed office buildings, but they are also appropriate for the following types of building:

- Commercial buildings
- Industrial buildings and warehouses
- Leisure buildings
- Stadia
- Hospitals
- Schools
- Cinemas
- Housing; both individual and residential buildings
- Refurbishment projects
- Car parks.

1.3 Scope of this publication

This publication gives guidance on the design and construction of composite slabs and composite beams in order to disseminate all the relevant information to the wide and varied audience involved in the design and construction chain. Guidance is given on design and construction responsibilities, and requirements for the effective communication of information between the different parties are discussed.

The principal aim of the guidance given in this publication is to identify the most important issues. The reader is directed elsewhere, including to British Standards, Eurocodes, and SCI publications for detailed design guidance. Summary boxes are used to highlight how to achieve economic, buildable structures through good practice in design.


makita

THE DESIGN AND CONSTRUCTION TEAM

The aim of this Section is to identify typical activities and responsibilities for the team members involved in the design and construction of a building using composite components. The precise delegation of responsibilities will depend on the details of the contract for a specific project, with which all parties need to be familiar.

As an overriding principle, the CDM Regulations 2015[1] state that ‘Every person on whom a duty is placed by these Regulations in relation to the design, planning and preparation of a project shall take account of the general principles of prevention in the performance of those duties during all stages of the project’.

The CDM Regulations require the Duty Holders to ensure that construction projects are planned, managed and monitored during the pre-construction phase and coordinate matters relating to health and safety during the construction phase to ensure that, so far as is reasonably practicable, the project is carried out without risks to health or safety.

2.1 Duty holders

The Construction (Design and Management) Regulations 2015 (CDM 2015), came into force in April 2015.

Virtually everyone involved in a construction project has legal duties under CDM 2015. These so-called Duty Holders are defined as follows. It can be seen that these definitions could often apply to more than one party involved in the design of a building, so where possible subsequent sections of this publication aim to be more specific.

Client - Anyone who has construction work carried out for them. The main duty for clients is to make sure their project is suitably managed, ensuring the health and safety of all who might be affected by the work, including members of the public. CDM 2015 makes a distinction between domestic and commercial clients. The latter have construction work carried out as part of their business. Commercial clients could be an individual, partnership or company and includes property developers and companies managing domestic properties.

Designer - An organisation or individual whose work involves preparing or modifying designs, drawings, specifications, bills of quantity or design calculations. Designers can be architects, consulting engineers and quantity surveyors, or anyone who specifies and alters designs as part of their work. They can also be tradespeople if they carry out design work. The Designer's main duty is to eliminate, reduce or control foreseeable risks that may arise during construction work, or in the use and maintenance of the building once built. Designers work under the control of a Principal Designer on projects with more than one Contractor.

Principal Designer - A Designer appointed by the Client to control the pre-construction phase on projects with more than one Contractor. The Principal Designer's main duty under CDM 2015 is to plan, manage, monitor and coordinate health and safety during this phase, when most design work is carried out.

In this publication the term '*Primary Structural Designer*' (PSD) is used to describe the person who has the overall responsibility for the design of the building and the term '*Specialist Structural Designer*' (SSD) is used to describe a person who has responsibility for a specific part of the design, such as the overall responsibility for the design of the decking and slab. Both can be either a Designer or Principal Designer, according to CDM terminology, depending on the number of Contractors on the project. Roles that are identified as being for the PSD in this document may be delegated to an SSD but this must be clearly stated in the contract documents.

There is a responsibility in the CDM regulations for Designers to consider the sequence of construction and provide at least one safe erection scheme for a project in which they should consider both the permanent and temporary works in their design[2] .

Principal Contractor - A Contractor appointed by the Client to manage the construction phase on projects with more than one Contractor. The Principal Contractor's main duty under CDM 2015 is to plan, manage, monitor and coordinate health and safety during this phase, when all construction work takes place.

Contractor - An individual or business in charge of carrying out construction work (e.g. building, altering, maintaining or demolishing). Anyone who manages this work or directly employs or engages construction workers is a Contractor. Their main duty under CDM 2015 is to plan, manage and monitor the work under their control in a way that ensures the health and safety of anyone it might affect (including members of the public). Contractors work under the control of the Principal Contractor on projects with more than one Contractor.

Worker - An individual who actually carries out the work involved in building, altering, maintaining or demolishing buildings or structures. Workers include: plumbers, electricians, scaffolders, painters, decorators, steel erectors and labourers, as well as supervisors like foremen and chargehands. Their duties include cooperating with their employer and other duty holders, and reporting anything they see that might endanger the health and safety of themselves or others. Workers must be consulted on matters affecting their health, safety and welfare.

A summary of duties under CDM 2015 is available in this link.

2.2 Roles in design and construction

Form of floor construction

The choice of floor construction and the general beam and column arrangements are the responsibility of the Architect and the PSD. The Architect will be concerned with more general and spatial aspects of the building form, such as the column locations, the construction depth of the floors, and the soffit appearance (if it is to be exposed).

The PSD will determine the general loads to be considered in the design of the structure, based on the type of occupancy for each area specified by the Architect/ Client. Details of any specific loads, for example due to services, may need to be supplied by others. It is important to avoid over-specification of loads in order to achieve material efficiency. The PSD will also undertake scheme designs to identify beam and slab solutions with spanning capabilities to suit the Architect's requirements.

Composite beams

The detailed design of the composite beams (Section 5) is the responsibility of the PSD, who should recognise that there is an interaction between the beam and slab design, particularly regarding the decking and transverse reinforcement. In designing the composite beams, due consideration should be given to the construction stage load case.

Although it may be necessary to consult the decking manufacturer for practical advice on shear connector configurations, or if fibre reinforcement is to be used to supplement or replace fabric reinforcement, it is the responsibility of the PSD to specify the shear connector type and quantities required.

When considering composite beams, the Designer should be aware of practical considerations such as the access requirements for using stud welding equipment (see Section 5.3.1) and minimum practical flange widths for sufficient bearing of the decking and subsequently slab (see Section 4.1.4). These requirements may have serious implications on the economy of the chosen solution.

Composite slab

The design of the composite slab (Section 4) may be the responsibility of either the PSD or an SSD (the decking manufacturer/supplier). Contract documents should make this clear for a given project. Particular attention should be paid to areas where there are special loads, such as vehicle loads and loads from solid partitions and tanks. Construction stage loads should also be considered at the design stage, with particular attention paid to any concentrated loads from plant or machinery required to carry out the safe erection of the building and its structure (see Section 6.3 for further information). When designing and detailing any reinforcement, the SSD should ensure that the specified bars can be located within the available depth of slab and that the correct reinforcement cover depths for the design durability conditions can be achieved.

It is recommended that the PSD prepares general arrangement drawings (which may be in digital form) for the slab, in addition to the steelwork general arrangement drawings. In particular, these drawings should define the edges and thickness of the slab, and they should form the basis of the decking layout drawings and the reinforcement drawings.

The PSD and the SSD should work closely together to make sure all the reinforcement requirements, including those for the beams, are covered and shown in the reinforcement schedule. A reinforcement layout drawing should be produced for each bay of each floor. The reinforcement grade, location, lengths, minimum overlaps and minimum concrete cover should be shown (and appropriate information about fibres if they are to be used). On site, these drawings will be used to check that all the reinforcement has been fixed correctly (or fibres, both type and dosage, correctly incorporated). Bar bending schedules showing the type, size and number of bars required should be included, so they can be used by the concrete Contractor to procure the reinforcement.

Designing a concrete mix to provide the structural and durability performance defined in the specification produced by the PSD is normally the responsibility of the Principal Contractor.

Choice of decking

The choice of decking and its general arrangement is the responsibility of the PSD. The design must consider the fire resistance of the slab (which may depend on the decking type), the ability of the decking and composite slab to resist the applied loading, any propping requirements, and the deflections at both the construction and in-service (composite) stages. As well as influencing all of these, the choice and orientation of decking profile may have implications for the composite beam design.

Design software, or load-span information, provided by a decking manufacturer will normally be used to verify the decking, as its performance is complex and certain design parameters are best determined from tests. The PSD must be satisfied with the information supplied in this form by the SSD (decking supplier/manufacturer) and ensure that it is not used ‘out of context’. Consultation with the decking supplier/manufacturer is recommended if there is any doubt. Where decking is specified for unusual applications, the ‘standard’ design information may not be directly applicable (see Section 4).

Decking arrangement and details

The decking layout drawings (Section 3.2) are normally prepared by the Contractor who will fix the decking, acting as an SSD. Details should be checked by the PSD, who should advise the SSD of any special requirements, such as the need for extra fixings when the decking is required to act as a wind diaphragm, or of any particular requirements concerning the construction sequence. The PSD should check that the proposed bearing details and the interfaces with the other elements of construction are practicable, and that they permit a logical, buildable sequence.

In preparing the decking layout drawings, the decking Contractor may find it beneficial to refine the design. For example, it may be necessary to change some of the continuous spans to simple spans for practical reasons. This may have implications for any propping requirements during construction so on-going information exchange is essential.

The loads that may be applied to the decking in the construction condition, both as a temporary working platform and as formwork, should be clearly indicated on the decking layout drawings or general notes. The loads that may be applied to the composite slab should be shown on the load layout drawings (these will be included in the Health and Safety File for reference throughout the lifetime of the building). It is therefore essential that all loading assumptions and design criteria are communicated to the decking Contractor.

Temporary works

Propping should generally be avoided because it reduces the speed of construction and therefore affects the construction sequence and economy. When propping is unavoidable, it is usually necessary to prop through several floors to support the prop loads. This can prevent other operations over a large area. However, when the construction sequence permits, propping does increase the spanning capability of the decking and may allow a reduction in materials. Determining the propping requirements is generally the responsibility of the PSD (normally using information supplied by an SSD), although local propping needs may change when the SSD details the decking layout. The SSD should be satisfied that the decking can withstand the concentrated loads from the propping arrangement below, which typically consists of timbers bearers 75 mm to 100 mm wide.

The location of lines of props or other temporary supports should be shown on the decking layout drawings. The design and installation of the propping system is the responsibility of the Principal Contractor, and propping systems should be braced appropriately. Removal of props should not be carried out before the concrete has reached its specified strength, or, when specified in the contract, before the PSD gives explicit approval.

In addition, the PSD should supply the Principal Contractor with the propping loads, and the dead load that has been considered, to help them to draw up the propping scheme. When devising the scheme, consideration must be given to the fact that floors will need to be designed to carry the concentrated loads from props (see Section 6 for advice on possible loading). Further advice on propping is given in Section 4.2.7.

Fire protection

The Architect is normally responsible for defining the fire resistance period required for the building, in order to satisfy Building Regulations' requirements, and for choosing the type of fire protection. The PSD, who may delegate to a specialist, is responsible for the specific details of the fire protection. The PSD should also make it clear on the drawings when any voids between the profiled decking and the steel beams have to be filled (see Section 5.2.3). In some cases, a qualified fire expert (acting as SSD for that specific activity) will need to perform these roles.

Safety

Whilst all parties involved in the design and construction process are required to consider construction safety, the CDM co-ordinator has some specific obligations under the CDM Regulations[1]. [It is to be noted that the post of Planning Supervisor established under the previous Regulations has been revoked and replaced by the post of CDM co-ordinator.] These obligations include the creation of the Health & Safety Plan and the Health & Safety File.

2.3 Design and construction sequences

The following flowcharts describe typical design (Figure 2.1) and construction (Figure 2.2) sequences for composite floor construction.

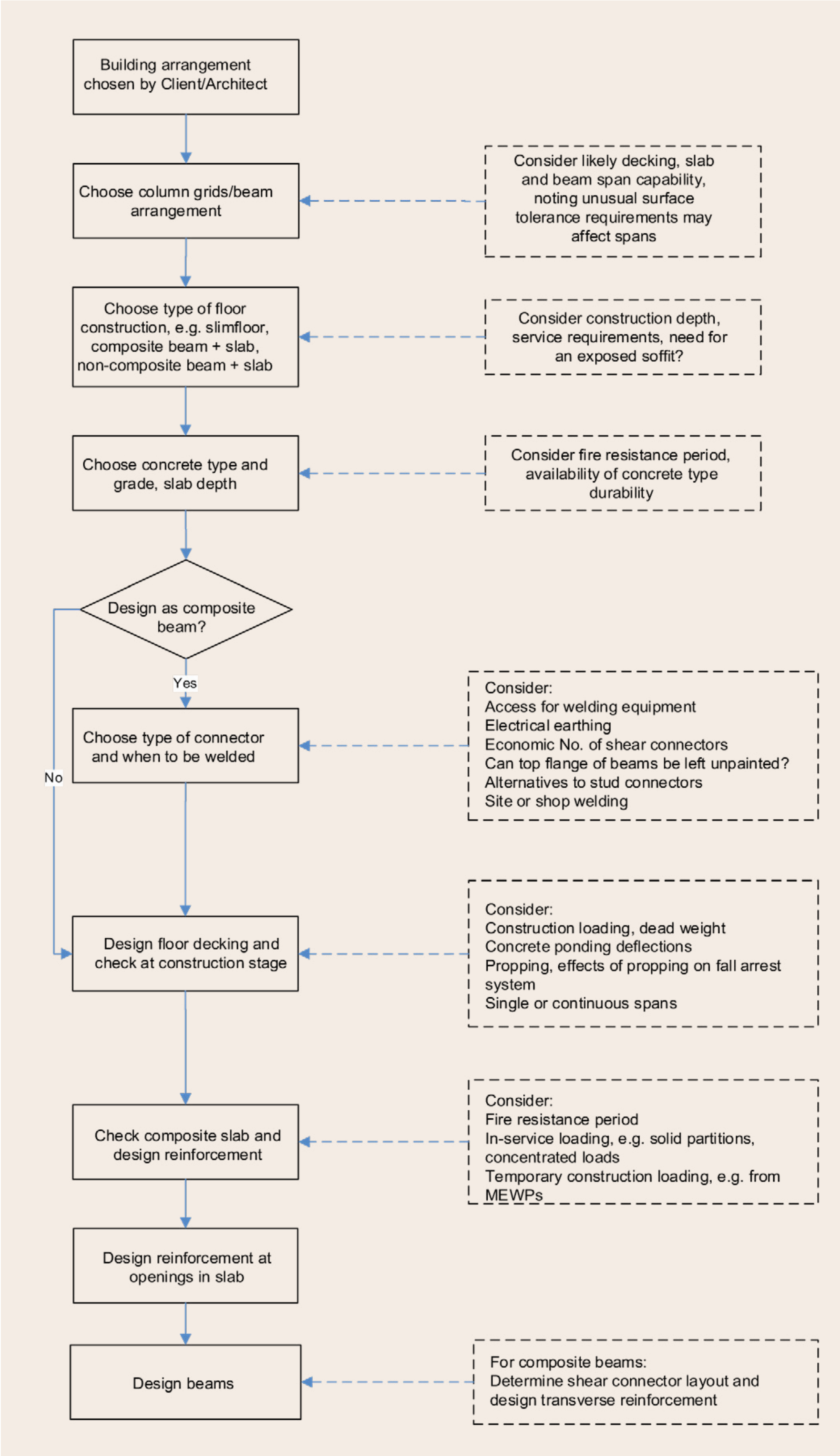


Figure 2.1
Sequence of design activities

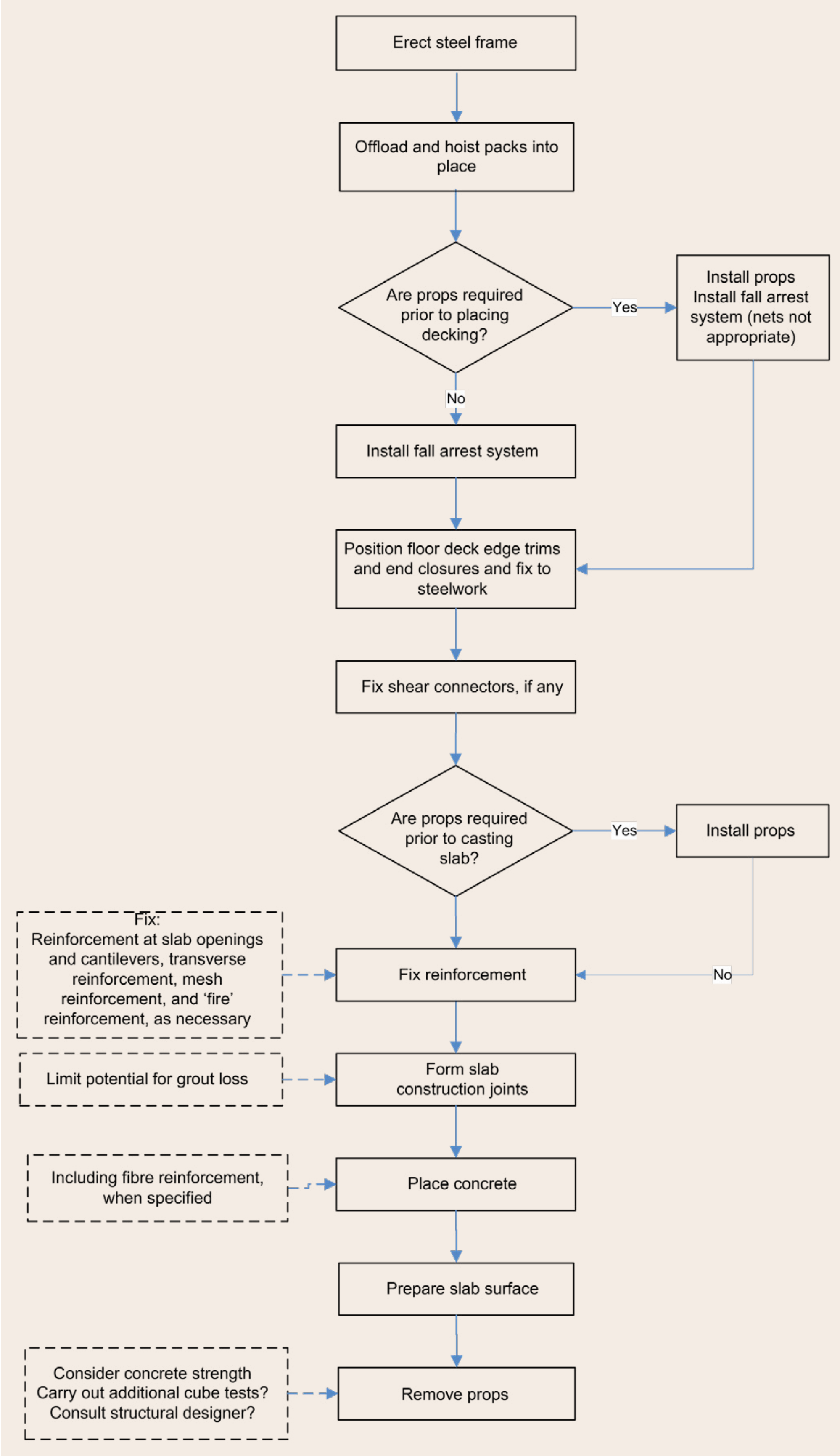


Figure 2.2 Sequence of construction activities

NLG
NLG

INFORMATION TRANSFER

Clear and timely communication of information is important given that several parties are involved in the building design process (see Section 2 for identification of typical responsibilities). There are also obligations placed on the key parties under the CDM Regulations[1] to exchange information during both design and construction.

3.1 Design stage

The design of composite beams and slabs is clearly influenced by spanning requirements, and the loads that are to be supported. In addition to grid layouts, it is therefore important that accurate details of all the loads (including construction stage) are established at an early stage. Unfortunately, some information, such as the loads due to the services, is often unavailable when needed, and the PSD has to use conservative values in order to give flexibility when the services are designed at a later stage. Such overdesign results in excessive use of material, and therefore excessive embodied carbon. Future designs will need to address this source of conservatism as the sector strives to use less material.

Knowledge of the position of services is also important, because it enables account to be taken of any opening requirements in the beam webs and/or slabs. Openings can have a significant effect on the resistance of a member.

The following list is a guide to the information required to design the composite slabs and beams:

- Column grid and beam general arrangement
- Position of slab edges
- Static and dynamic imposed loads (to include consideration of any temporary concentrated loads from plant/machinery that may be required during construction)
- Services and finishes loads
- Type of finishes (which may affect the reinforcement needed to control cracking)
- Special loads (e.g., walls, wind diaphragm loads)
- Fire resistance period
- Decking type (re-entrant or trapezoidal)
- Slab depth limitations
- Minimum mass requirements (for acoustic performance)
- Location of openings

- Requirements for soffit appearance and general exposure
- Requirements for service fixings
- Requirements for cladding attachments (which may affect the slab edge detailing)
- Construction tolerances (including requirements for the upper surface of slabs)
- Deflection limits
- Propping requirements or restrictions
- Any known restrictions on the use of thru-deck welding.

In order to prepare the decking layout drawings, an SSD will also need to know the:

- Concrete type and grade
- Shear connector layout and details
- Cladding support method (for edge trim design, etc.).

There are also specific issues of information transfer that arise because the design of the decking and composite slabs often relies on the use of information presented in decking manufacturers' literature or incorporated into their software. It is important that explanatory information associated with software and literature is comprehensive. For example, in load-span tables the following points should be clear:

- Are the loads that are given unfactored, or factored design values?
- What are the assumptions concerning ponding of concrete (e.g. is pouring to a constant slab thickness assumed)?
- What allowances, if any, have been made for service loads etc.?
- What fire performance do the tables relate to?
- Do specified reinforcement (fabric, bars or fibres) requirements imply any crack control capability?
- Do the tables imply adequate serviceability behaviour as well as resistance, and if so, what limiting criteria have been assumed?

If the slab design is split between the PSD and the design service of a decking manufacturer (SSD), it is essential that there is clear communication of all relevant design information including the specific design codes to be used.

3.2 Construction stage

An absence of comprehensive information transfer between the design and construction teams can lead to delays or, at worst, incorrect or unsafe construction.

The site personnel should check the information provided and confirm that it is complete, passing any relevant information to appropriate contractors. Any variations on site that might affect the design should be referred to the PSD.

Decking layout drawing

The layout drawings should be available for those lifting the decking, so that the bundles can be positioned correctly around the frame to minimise risk during installation and reduce manual handling. Clearly, they should also be available for the deck laying team so that the bundles are loaded out in accordance with the deck layout drawings.

Although different decking contractors' drawing details may vary slightly, the drawings should show (in principle) each floor divided into bays, where a bay is an area that is to be laid from a bundle as one unit. Bays are normally indicated on the drawing using a diagonal line. The number of sheets and their length should be written against the diagonal line. The bundle reference may also be detailed against this diagonal line. Further construction notes for the bay can be referenced using numbers in circles drawn on the diagonal lines. Figure 3.1 shows an example of a decking layout drawing, but with the shear connectors and fastener information omitted for clarity. Decking contractors' literature should be referenced for exact details.

The approximate starting point for laying the decking should be given on the drawings, together with the direction in which laying should proceed. All supports (permanent and temporary) should be identified, and whether they should be in place prior to laying the decking. The letters TP on the drawings typically indicate lines of propping. Column positions and their orientation should also be shown. The decking type, thickness and material strength should be indicated on the drawing.

The location of all openings trimmed with steelwork, and all slab perimeters, should be given relative to the permanent supports. This may be in the form of a reference box titled 'Edge Trim', with a reference number (for details shown elsewhere), the slab depth, and the distance from the edge of the slab to the centre line of the nearest permanent support, but decking contractors' literature should be referred to for the exact drawing details.

The shear connector layout should also be shown on the decking drawings, or on separate drawings for reasons of clarity. The information should include the type of shear connector, its length, orientation (if shot-fired) and position relative to the ribs. The minimum distance between the centre-line of the shear connector and the edge of the decking is typically given in a standard detail. Details of preparation, fixing and testing of shear connectors should be available on site. For more information on shear connection, refer to Section 5.3 and BCSA Code of Practice for Metal Decking and Studwelding[3].

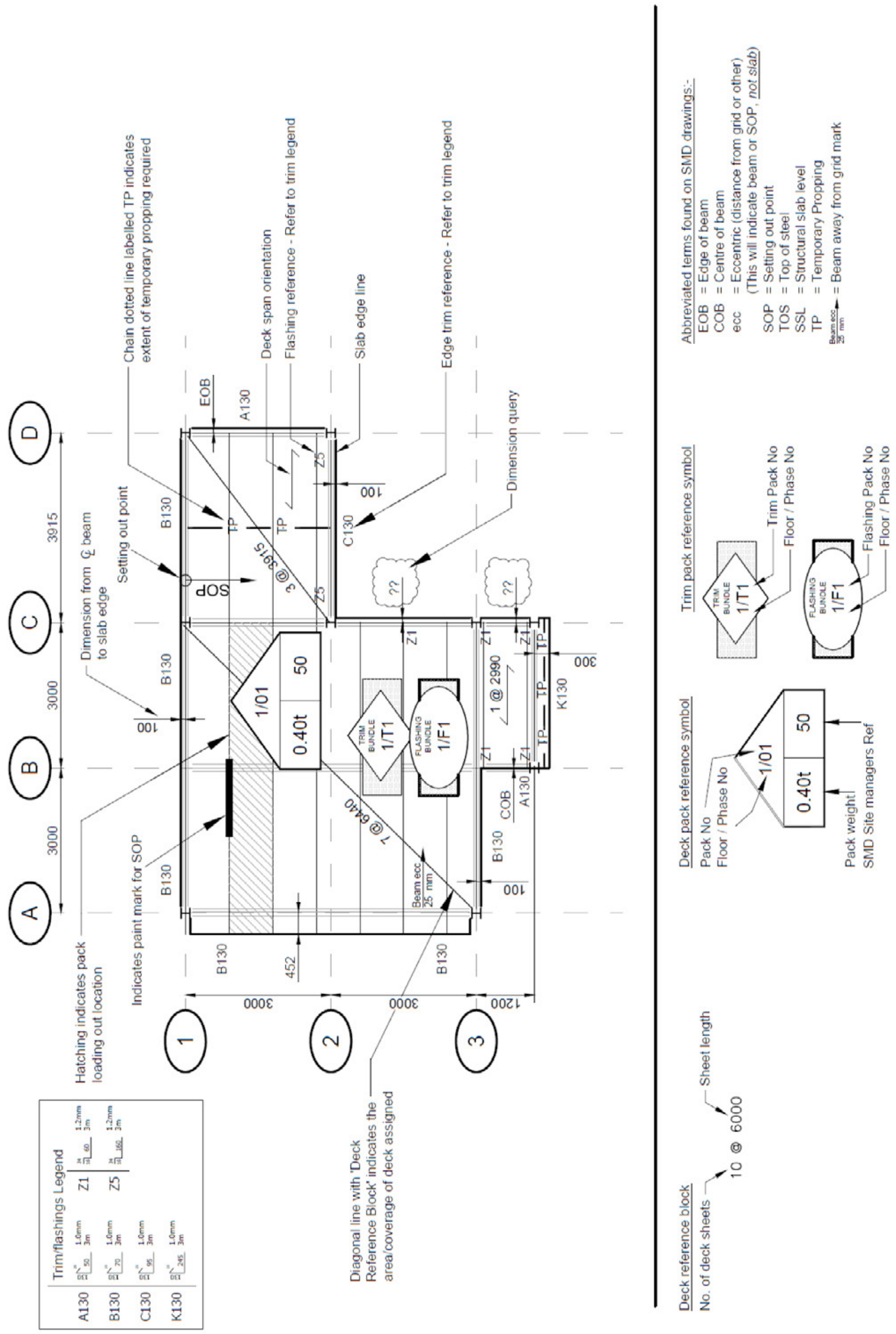


Figure 3.1
Typical decking layout drawing with the shear connector and fastener information omitted

courtesy of SMD

Fastener information should be given on the drawings. The fastener type for both seams and supports should be given, along with maximum spacings (or minimum number of fasteners per metre). Where the PSD has designed the decking to act as an effective lateral restraint to the beams and more or stronger fasteners than used in a manufacturer's normal fixing arrangement are necessary, this should be clearly indicated on the decking layout drawing and/or general notes. Prior to stud welding, the fasteners must also resist wind loads on the decking.

The general notes should include the design loads that the decking can support in the construction condition. Guidance on avoidance of overload prior to placing the concrete is given in Reference 3.

A copy of the decking layout drawings must be given to the Principal Contractor so that checks can be made that the necessary propping is in place. The Principal Contractor will also need to refer to these drawings for details of the maximum construction loading and any special loading.

Decking bundle identification

An identification tag should be attached to each bundle of decking delivered to site. The tag will normally contain the following information:

- Number of sheets, their lengths and thickness
- Total bundle weight
- Location of floor to receive bundle
- Deck type
- Bundle unique identification.

Product information on the decking should also be available on site, including the height of the ribs and their spacing, and other technical information.

Information required for laying the reinforcement, casting the slab and its use thereafter

Unless concrete reinforcement is provided uniquely by fibres, a reinforcement layout drawing should be prepared for each bay of each floor by the PSD. The location, length, minimum overlap and minimum concrete cover of all reinforcement should be indicated. The grade of all reinforcement should also be noted. This grade can be checked against the identification tag for each reinforcement bundle delivered to site. Appropriate information about fibres should be given, if they are to be used.

Important reinforcement details (such as at construction joints, support locations, openings and edges) should be referenced and placed on this drawing.

The floor slab general arrangement drawings (or the Specification) should include the concrete performance requirements or mix details (including any details for fibre reinforcement), surface finish requirements, level tolerances and any restrictions on the location of construction joints. They should also identify the minimum concrete strength at which temporary supports may be removed, the minimum concrete strength at which temporary construction loads may be applied, and, where appropriate, the maximum allowable vehicular axle weight (for punching shear). Minimum concrete strengths may be given in terms of days after concreting.

Propping information

As discussed in Section 2.2, the PSD should supply the Principal Contractor with the floor dead load value to allow a propping solution to be developed.

3.3 3D Modelling

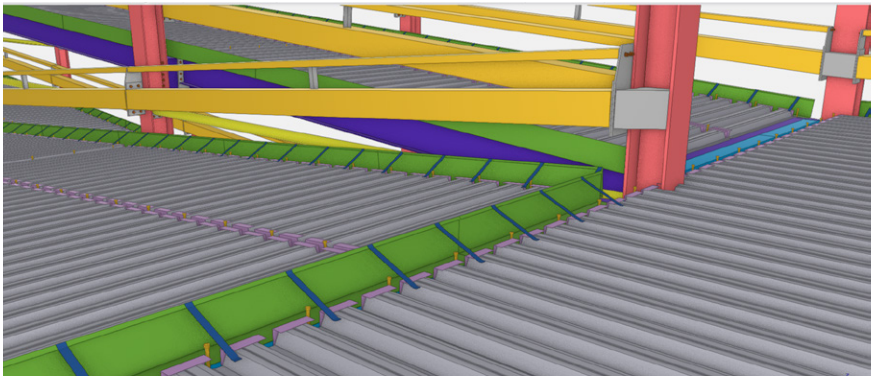
3D modelling can be used for all sizes of project and can be particularly beneficial if a project has some unusual aspects. It may also be necessary simply because it is the approach adopted by other parties for a given project. There can be many advantages to using 3D models, including design visualisation, clash detection and the production of drawings (Figure 3.2).

The visualisation aspect assists in identifying issues that may only otherwise be spotted during the installation stage on site, such as level inconsistencies, or support issues that would have required rectification through the late provision of trimming steel or additional welding during the deck installation. When needs are identified early, supports can instead be applied during fabrication. A 3D model allows the engineer to view the deck and stud layout as it would be installed on site and confirm that it is in accordance with their design.

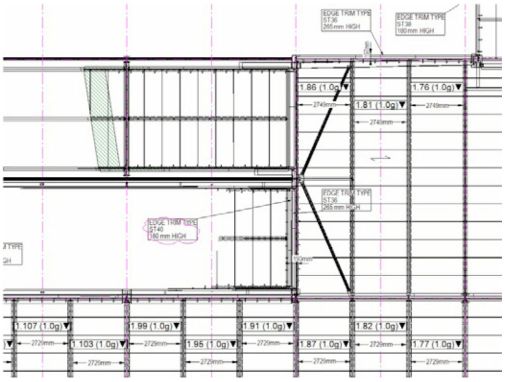
The model can also be used to identify any potential clashes. Some clashes may have been small enough to rectify on site, for example by notching around a column in a run of decking. However, spotting significant clashes before arriving on site, such as clashes with steelwork due to level inconsistencies, or clashing with an existing wall, means they can be resolved early and delays avoided.

The steelwork contractor, design engineer and various sub-contractors may each have their own model, which may be requested by any of the other parties to facilitate the benefits listed above. A decking model is created by the decking manufacturer/supplier and is set out specifically, so it lays directly onto the steelwork contractor's or main contractor's model using a common coordination point.

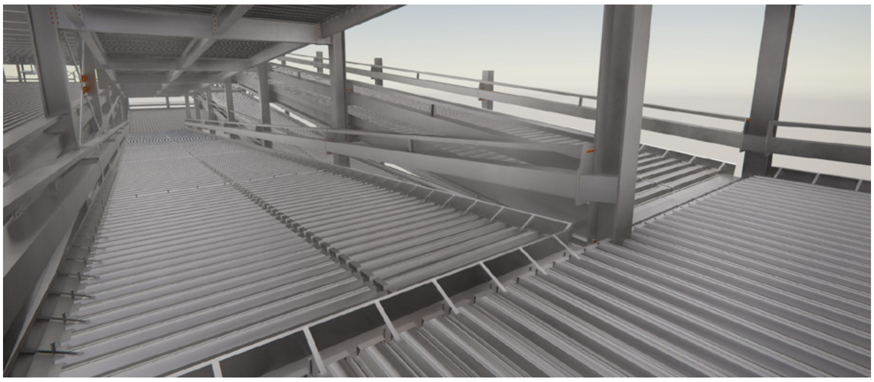
Typical 3D modelling software such as Tekla Structures has an integrated system which creates drawings based on the views created within the model space. A rendered model can be created which can show the materials with the correct lighting and texture to give a good indication of how it will look on-site, as can be seen in Figure 3.2.



Model

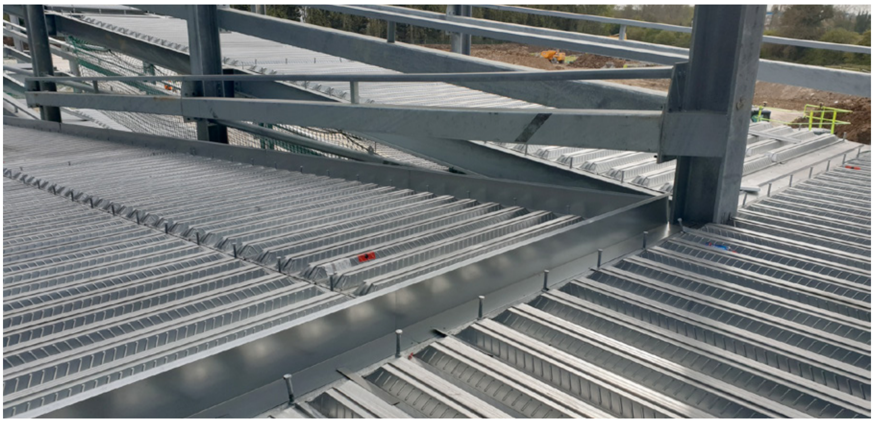


Deck GA



Rendered Model

Figure 3.2 Image of a 3D model, GA drawing, rendered model and picture of the installed deck



Installed Deck Picture

courtesy of MSW Ltd

DESIGN OF DECKING AND SLABS

This Section provides information about design principles and procedures, codified design rules, and guidance on good practice in design and detailing. Along with Section 5, it is aimed primarily at the PSD, and any SSD. Summary boxes are used to highlight particular issues of good practice, or areas where particular attention is needed.

4.1 Steel decking

The steel decking (referred to as sheeting in the Eurocodes) has two main structural functions:

- During concreting, the decking supports the weight of the wet concrete and reinforcement, together with the temporary loads associated with the construction process. It is normally intended to be used without temporary propping
- In service, the decking acts 'compositely' with the concrete to support the loads on the floor. Composite action is obtained by shear bond and mechanical interlock between the concrete and the decking. This is achieved by the embossments rolled into the decking – similar to the deformations formed in rebar used in a reinforced concrete slab - and by any re-entrant parts in the deck profile (which prevent separation of the deck and the concrete).

The decking may also be used to stabilise the beams against lateral torsional buckling during construction, and to stabilise the building as a whole by acting as a diaphragm to transfer wind loads to the walls and columns. If required it must be explicitly designed to achieve the intended role, in particular with regard to the adequacy of the fixings; see references 4 and 5 for recommendations on wind loading and design of temporary fixings. The decking, together with either welded fabric reinforcement placed in the top of the slab or steel/synthetic fibres throughout the slab (see Section 6.2.1), also helps to control cracking of the concrete caused by shrinkage effects.

4.1.1 Decking profiles

Decking is produced by a number of manufacturers in the UK. Although there are similarities between their profiles, the exact shape and dimensions depend on the particular manufacturer. There are two generic types of so-called shallow decking: re-entrant (dovetail) profiles and trapezoidal profiles. An indicative example of a re-entrant decking profile is shown in Figure 4.1. Indicative examples of trapezoidal decking profiles are shown in Figure 4.2.

The traditional shallow decking profiles are between 50 to 80 mm deep, with a rib spacing usually of 150 to 333 mm. This type of decking typically spans 3 m to 4.5 m, leading to frame grids that are multiples of 3 m or 4 m spacing, for which temporary propping is usually not required. Some manufacturers also offer profiles in excess of 80 mm deep, the deepest of which can span 6 m unpropped as a simply supported member. Such so-called deep decking profiles, which are over 200 mm deep, are mainly used in shallow floor construction, where the steel beams are integrated into the depth of slab.

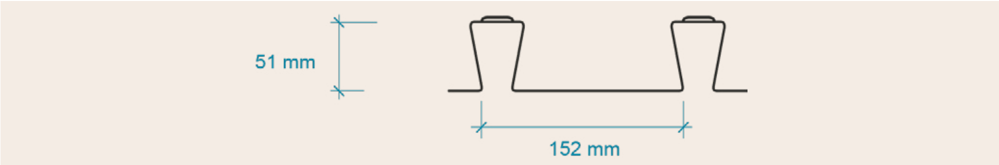


Figure 4.1 Example of a re-entrant decking profile

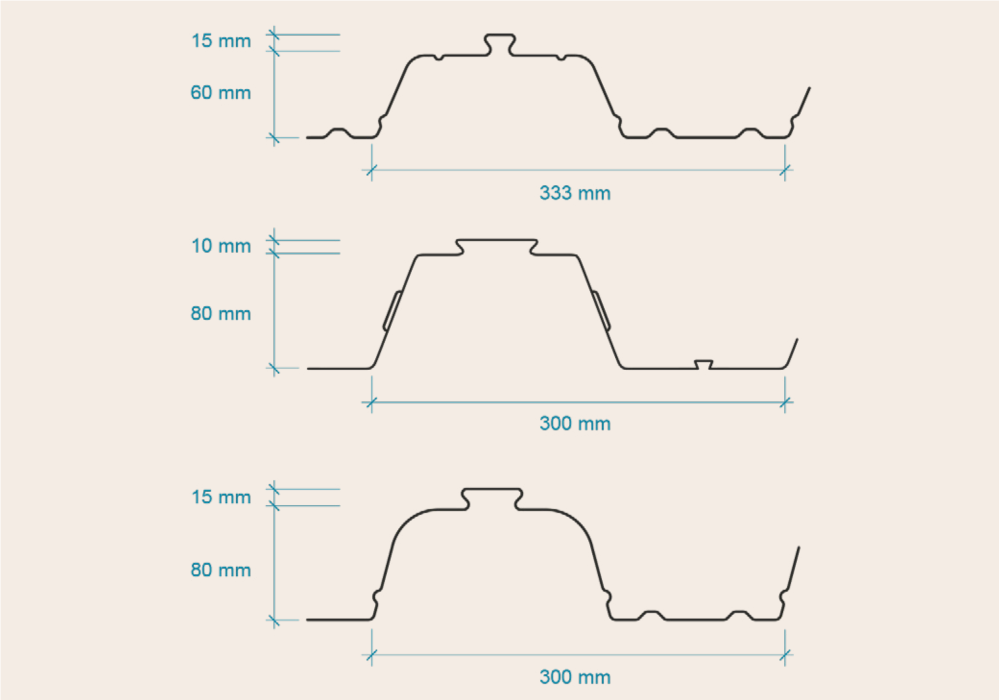


Figure 4.2 Examples of trapezoidal decking profiles

Some trapezoidal profiles include a single stiffener in the centre of each trough, whereas others have two stiffeners. The advantage of two stiffeners is that a stud can be placed centrally in the trough, whereas a single stiffener means that studs have to be placed off centre (see Section 5.3.1 for further information).

The height of the steel decking h_p is defined as the height to the shoulder of the profile, even if the profile has a re-entrant detail on the top flange. So, for the 80 mm deep profile, shown in Figure 4.2, h_p = 80 mm. When checking the fire resistance of composite slabs with trapezoidal decking, the slab thickness criterion for insulation performance excludes the height of the stiffener (re-entrant detail) on the top flange, the thickness of concrete in this instance is $h_s - h_p$, where h_s is the depth of the slab. However, when checking the depth of concrete that can act structurally above a transverse deck, the presence of the top flange stiffener does need to be taken into account. The overall height of the profile including the top flange stiffener is taken as h_d and the thickness of concrete h_c in this instance is $h_s - h_d$. For profiles that do not have top flange stiffener, $h_d = h_p$.

See links below for details of some manufacturers' specific profiles:

1. Richard Lees Decking
2. Tatasteel
3. Kingspan Structural Products
4. Structural Metal Decks Ltd.
5. Construction Metal Forming Ltd.

The grades of steel used for decking are specified in BS EN 10346:2015[6]. The most common grade in the UK is S350 although S450 is also available from several manufacturers (the designation identifies the yield strength of the steel in N/mm^2 or MPa).

Decking is generally rolled from 0.9 to 1.2 mm thick steel sheet (note this thickness includes that of the galvanising). The spanning capability of a given decking profile clearly increases as the steel thickness increases, however span increases are not in direct proportion to the strength. The steel is normally galvanized before forming, generally to a specification of Z275 which denotes 275 grammes of zinc per m^2. This results in a thickness of approximately 0.02 mm per face (and is sufficient to achieve an excellent design life in internal applications with mild exposure conditions). Alternative metallic coatings according to BS EN 10346:2015 are available, for example the high durability, zinc, aluminium and magnesium (ZM) coatings. These may be used where improved durability is required without increasing the coating thickness, which facilitates thru-deck welding. For example, Magnelis® ZM120 with a coating mass of 120g/m^2 is equivalent to Z275 with a thickness of 0.01 mm per face and Magnelis® ZM310 with a coating mass of 310g/m^2 is equivalent to Z600 and has a thickness of 0.025mm per face. Alternatively, thicker zinc coatings of 350 g m^2, and up to 600 g/m^2, are available but may have extended lead times and require a minimum quantity order. Thru-deck welding of shear studs is not recommended for galvanised decking with more than 350 g/m^2 of zinc coating because of the risk of substandard weld quality[7] , and in this case shot-fired connectors, such as Hilti X-HVB, may be considered. Polyester paints are sometimes applied over the galvanizing to provide a longer service life and have the advantage of prolonging the lifespan of the floor. Advice should be sought from the supplier/manufacturer when decking is to be used in a moderate or severe environment. Further advice on the use of composite construction in an aggressive environment is given in AD247[8], which despite its age it is still relevant.

> Standard thickness galvanizing (275 g/m^2) will give an excellent design life in most internal applications. High durability, zinc, aluminium and magnesium (ZM) coatings are available where improved durability is needed with a similar coating thickness to the standard galvanised coating to avoid precluding thru-deck welding.

4.1.2 Design for resistance

The temporary construction stage usually governs the choice of decking profile.

When designing to Eurocodes, the construction loading that should be considered in the design of the decking is defined in BS EN 1991-1-6:2005[9] and its National Annex. Although the provisions are a little unclear in the Eurocode itself; the following is understood to be the recommended construction loading, which should be treated as a variable load. As can be seen below, this interpretation is also consistent with the guidance given in BS 5950-4:1994:

i. 0.75 kN/m^2 generally

ii. 10% slab self-weight or 0.75 kN/m^2, whichever is greater, over a 3 m x 3 m 'working area'. This area should be treated as a moveable patch load that should be applied to cause maximum effect.

This is shown diagrammatically in Figure 4.3, which shows the load scenario for a single span condition.

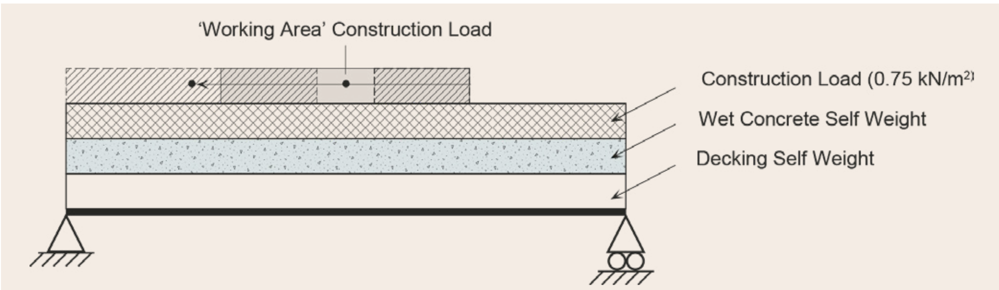


Figure 4.3 Loading on decking at the construction stage to BS EN 1991-1-6:2005

When the deck is continuous over a temporary or permanent support this must be considered when designing the decking profile[10]. The 'working area' construction load should be moved to the most onerous place to cause maximum effect as shown in Figure 4.4.

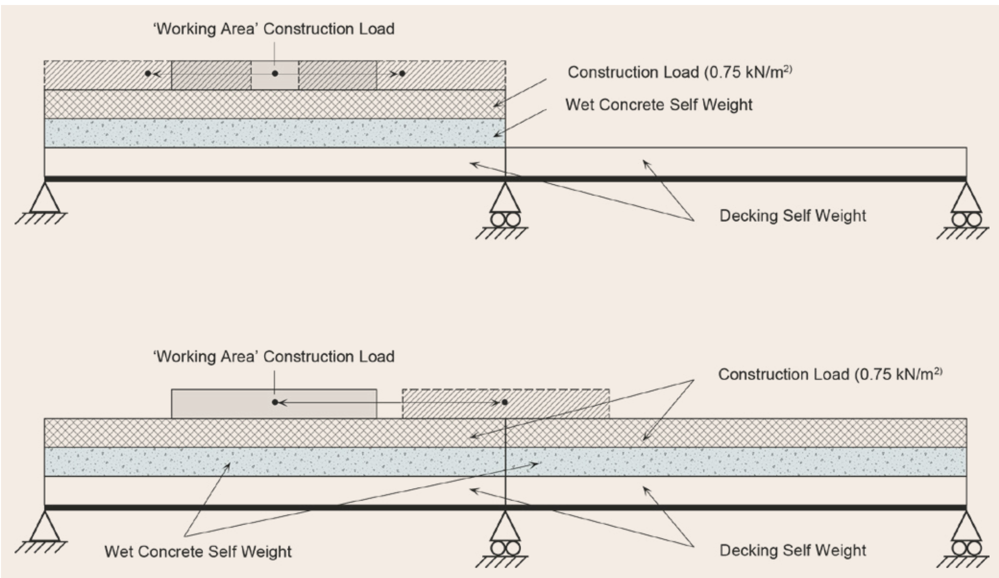


Figure 4.4 Loading on decking at the construction stage for continuous decks

When designing to BS 5950-4[11], the construction loading is defined as:

- A uniformly distributed load of 1.5 kN/m^2 acting over one span. For spans less than 3 m, the load should be increased to $4.5/L_p$, where L_p is the effective span of the decking
- A reduced load of 0.5 kN/m^2 on adjacent spans.

It is worth noting that, regardless of codified values, it may be prudent for the construction imposed load to be increased when fibre reinforced concrete is used. Not only may the fibre reinforced concrete be less fluid (so more prone to heaping), but the absence of reinforcing fabric means that the construction loads from operatives will be transferred directly to the decking.

The construction loads must be considered in addition to the self-weight of the slab (usually 2 to 3 kN/m^2), which may need to include an allowance for 'ponding' of the concrete (see Section 4.1.3). Care must be taken so that site practice reflects the assumptions made in design, as concrete thickness is used in the calculation of deflections of the decking and the supporting beams. Pouring the concrete to a constant thickness over the supporting beams will ensure that any excess volume (and therefore weight) is limited to the ponding values associated with deflection of the decking itself, and where necessary allowed for in design software. If the concrete is poured to achieve a flat upper surface then the beam deflections will also result in extra concrete.

The above load values allow for construction operatives, impact, the heaping of concrete during placing, hand tools, and small items of equipment and materials for immediate use. The loads are not intended to cover excessive impact or excessive heaping of concrete, pipeline or pumping loads.

For design to the Eurocodes, densities of the wet weight of reinforced concrete are given in BS EN 1991-1-1:2002[12] , and the data is classified as 'informative'. The data is for heavily reinforced construction associated with conventional reinforced concrete structures. The UK NA states that those values may be used, but it is recommended that the density of dry concrete used in composite floor construction should be 24 kN/m^3 for normal weight concrete and 19 kN/m^3 for lightweight concrete, increased to 25 kN/m^3 and 20 kN/m^3 respectively for wet concrete. The weight of the reinforcement should be added separately. The self-weight of the wet concrete is treated as a variable load for the construction condition, but the reinforcement may be considered as a permanent load.

In BS 5950-4, wet densities are given as 2400 kg/m^3 and 1900 kg/m^3 for normal and lightweight concrete respectively, and similarly 2350 kg/m^3 and 1800 kg/m^3 for dry concrete. The self-weight of the wet concrete is treated as a dead load.

Where construction equipment such as a power float is required to be used before the concrete has gained full strength, it is recommended that this additional loading does not exceed the allowable temporary construction loading of 1.5kN/m^2 over the

3 m × 3 m ‘working area’ (this approach is conservative as it ignores any strength that may be present in the concrete and resulting onset of composite action), see section 6.3.2 for further information.

Further information on the design actions during concreting and the recommended expressions for combinations can be found in AD 346[13] and AD 367[14].

The design of decking is covered in BS EN 1993-1-3:2006[15]. Some deeper profiles are outside the scope of this document, so the rules should be applied with care. The moment resistance of the section is established using an effective width model to take account of the slender steel elements in compression. Stiffeners (in the form of folds) are often introduced into the decking profile to increase the effectiveness of the section. The effective width approach is relatively conservative because the section behaviour is very complicated owing to local buckling, and so the section properties can be predicted neither easily nor accurately. The design of the decking is also covered in BS 5950-4 and BS 5950-6[11], where a similar approach is given.

As an alternative to analytical procedures, the Standards also allow the use of testing in order to determine the performance of the decking. Spans 10% to 15% in excess of the limits predicted by simple elastic analysis using effective section models are possible. For this reason, manufacturers generally determine section properties for use in software and load-span tables that are based on tests rather than on an elastic analysis approach. The tests should cover all relevant configurations and failure modes, noting that for continuous spans resistance to combined loading over intermediate supports is often critical. This is particularly the case for design to BS EN 1993-1-3:2006, which introduced an SLS verification as well as ULS.

In addition to tests under simulated uniform loading, further tests are normally carried out to check the resistance of the decking to localised loading. This provides information on the resistance to local loading from above as well as on the maximum allowable prop and support forces.

Decking design based on testing is more economical than design based on analytical models. Manufacturer’s (empirical) information should therefore be used whenever possible.

Empirical information must not be used for designs outside the scope of the tests on which it is based. Load-span tables will generally only cover uniformly distributed loading.

4.1.3 Design for serviceability

It is necessary to limit the deflections at the construction stage to limit the volume of concrete that is placed on the decking; excess deflections will lead to ‘ponding’ of the concrete, and this will increase the dead loads on the structure. Deflection limits for the decking are given in BS EN 1994-1-1:2004[16], and in BS 5950-4. According to BS

EN 1994-1-1:2004, if the central deflection of the decking δ is greater than 1/10 of the slab thickness, ponding should be allowed for. In this situation the nominal thickness of the concrete over the complete span may be assumed to be increased by 0.7δ.

For the serviceability limit state, the recommended value of the deflection $\delta_{s,max}$ of steel decking under its own weight plus the weight of wet concrete is the span/180 in BS EN 1994-1-1:2004. In BS 5950-4, the limit on the residual deflection of the soffit of the deck (after concreting) is also given as span/180 (but not more than 20 mm), which may be increased to span/130 (but not more than 30 mm) if the effects of 'ponding' are included explicitly in the design. The deflection of the steel decking has an impact on the surface flatness/level that can be achieved, so where tight surface tolerances are specified tighter deflection limits are likely to be needed.

The limits given in standards may be increased 'where it can be shown that greater deflections will not impair the strength and efficiency of the slab', although this is rarely applied. As a further check, it is recommended that the increased weight of concrete due to ponding should be included in the design of the support structure if the predicted deflection, without including the effect of ponding, is greater than one tenth of the overall slab depth.

The requirement for verification of the profiled decking at SLS in BS EN 1994-1-1:2004 is expressed simply in terms of deflection under the weight of wet concrete and there is no requirement to check that such deflection should be elastic. However, it is recommended that there is also a check to ensure that there is no premature local buckling of the profile under the weight of wet concrete and the construction loading, to prevent irreversible deformation. This applies particularly to the intermediate support regions of continuous spans.

> Excess deflections of the decking (and beams) may lead to 'ponding' of the concrete and therefore increased self-weight of the slab. The decking and propping requirements should be chosen to minimise ponding. Excess deflections may also affect the ability to achieve tight tolerances for the upper surface of the slab.

4.1.4 Supports

Minimum bearing length

The bearing length is the longitudinal length of decking or slab in direct contact with the support. In each case, this length should be sufficient to satisfy the following relevant criteria. For decking, it should be sufficient to avoid excessive rib deformations, or web failure, near the supports during construction. For the slab, it should be sufficient to achieve the required load carrying capacity of the composite slab in service.

The recommended minimum bearing lengths shown in Figure 4.5 should be observed. The values given in this figure are based on the requirements of BS EN 1994-1-1:2004, but similar requirements are given in BS 5950-4. These limits should also be respected for temporary supports. The limits given represent nominal values that should be considered in the design and detailing, i.e., they include an allowance for construction deviations leading to slightly reduced values on site.

The recommended bearing lengths and support details differ depending upon the support material (steel, concrete, etc.), and they are different for interior and exterior (end) supports.

Typical values and details are given in Figure 4.5 for the following:

- **Steel or concrete supports** - Composite slabs on steel or concrete supports should have a minimum end bearing length of 75 mm for the slab, and a minimum end bearing length of 50 mm for the decking (see Figure 4.5(a) and Figure 4.5(b)). For continuous decking, the minimum overall bearing length should be 75 mm. When two discontinuous sheets bear onto a steel beam flange, the minimum bearing distance should be increased to 60 mm (i.e., requiring a 120 mm support width) to ensure that minimum distances associated with detailing around the shear studs can be achieved (these concern the distance from the stud to the edge of the flange, and the distance from the stud to the end of the sheet, the combination of which is greater than the bearing required for the decking).
- **Masonry and other support types** - Composite slabs on supports made of materials other than steel and concrete should have a minimum end bearing length of 100 mm for the slab and a minimum end bearing length of 70 mm for the decking (see Figure 4.5(c) and Figure 4.5(d)). For continuous decking, the minimum overall bearing length should be 100 mm.

The flange width of supporting steel beams should be sized to provide the minimum bearing, by assuming that erection tolerances sum up unfavourably.

Details of how the decking should be fixed to supports are given by BCSA in Reference 3.

If ‘thru-deck’ welding of the studs is to be used to anchor the decking, for example so that it may contribute to the transverse shear reinforcement (see Section 5.3.2), the dimensions specified in Figure 4.5 may need to be increased (see Figure 5.10).

In cases where the slab must transfer wall loads from one storey to the next (rather than simply sitting on the top of a wall), the relatively lower volume of voids in a slab formed using a re-entrant profile means it may be better able to satisfy the design requirements because the trough width in contact with the supporting surface is larger than that of a trapezoidal profile.

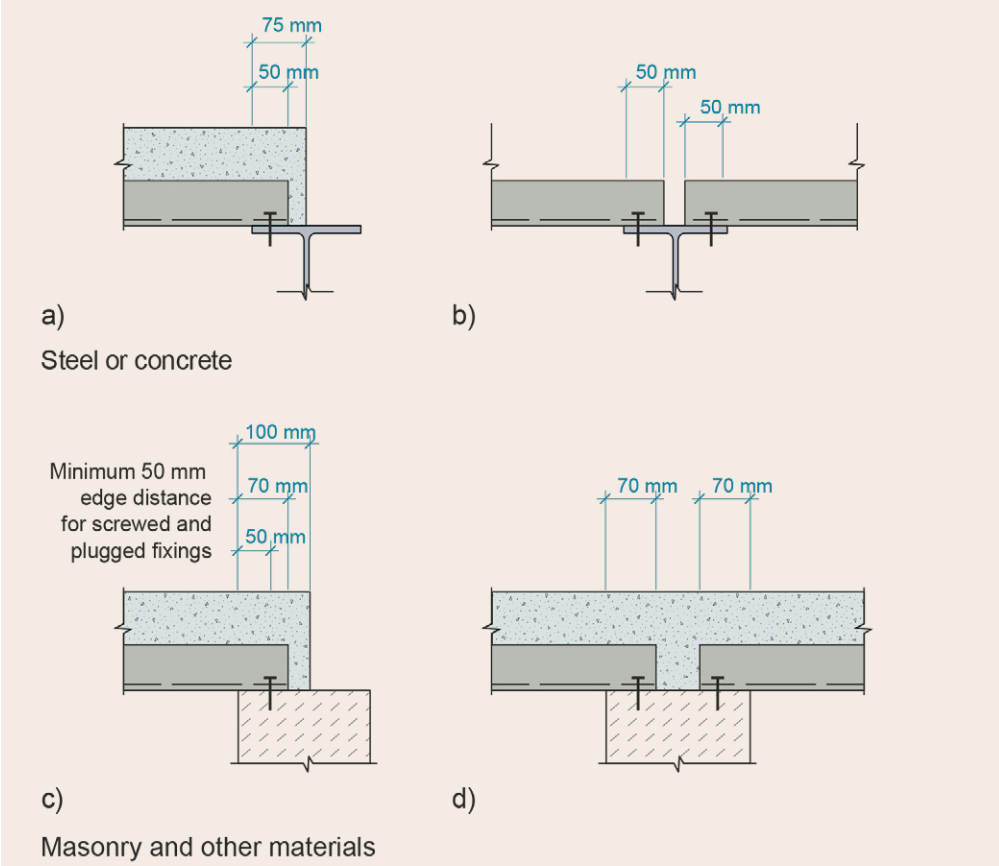


Figure 4.5 Minimum bearing lengths for permanent supports

Recommended support details

In addition to the most common detail of a slab bearing on a steel beam or wall, there are a number of other commonly occurring support conditions which need to be considered at the design stage in order to avoid problems or delays on site. Some typical details are shown in Figure 4.6.

There are two basic cases at the interface of the decking with beams; where end support is required (Figure 4.6(a)), and where side support is required (Figure 4.6(b)). In both cases a steel 'shelf angle' is normally detailed as the decking support, and it is preferable to fix this during fabrication. Angle flashing can be used provided its adequacy is confirmed by structural design. To enable fixing of the decking, particularly in the case of an end support, it is important that the leg of the angle extends a minimum of 50 mm beyond the flange of the beam. To facilitate execution increasing this dimension to 75 mm is advised in some situations. The support angles should be continuous and extend as close as is practical to the beam end connections, to minimise the unsupported length of the decking.

Support is also required when the decking interfaces with a concrete or masonry wall. This may be provided by attaching a steel angle, flashing, or timber batten to the wall, preferably by using cast-in fixings (Figure 4.6(c)). Provision may need to be made to achieve reinforcement continuity between the wall and slab.

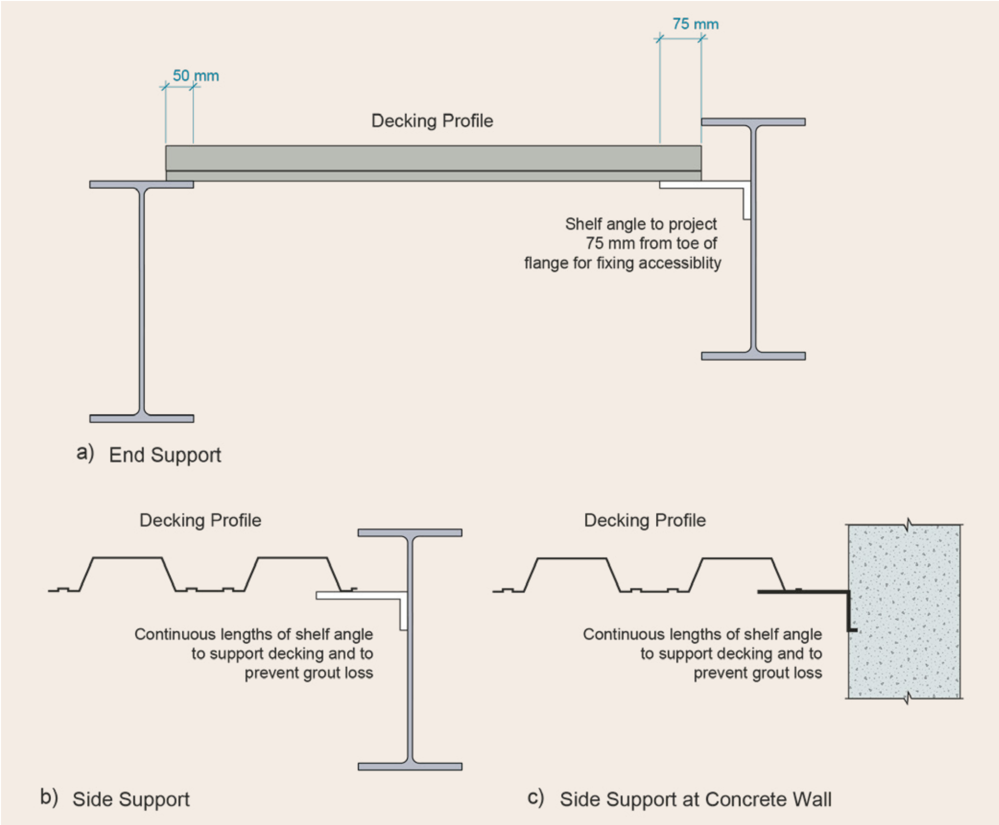


Figure 4.6 Decking support details at a beam web and at a concrete wall

The decking should not cantilever beyond a support more than 600 mm (or ¼ of the span, if less) when spanning perpendicular to it. When the decking is spanning in a parallel direction, no cantilever is possible without extra support being provided – although the edge trim may cantilever a short distance (see Section 4.2.6).

The decking may also need to be supported around penetrations which reduce, or prevent, the effective bearing. Supports should be provided as part of the permanent steelwork, for example in the form of cleats or angles. Examples of when such supports are necessary include when the decking is penetrated by columns that are greater than 250 mm wide (without incoming beams on both axes), or by columns supported off beams. Note this 250 mm dimension applies when the penetration runs along the edge of the decking, but when there is a penetration against the end it should be reduced to 50 mm. Figure 4.7 shows a recommended detail using a shelf angle to support the decking around a column.

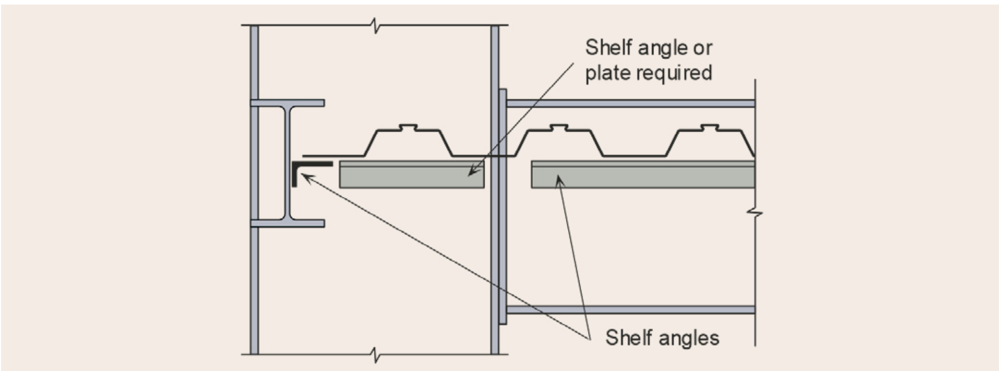


Figure 4.7 Decking support details at a column web

A less common detail is one in which the column is supported by a beam, in which case special detailing may be required to achieve sufficient bearing for the decking around the perimeter of the column. Where the deck is spanning in a direction perpendicular to the beam, the minimum bearing of 50 mm required to support the end of the decking may not be available because of the presence of the column base plate. Therefore, the beam flange may need to be extended by welding plates to the sides at the column position, as shown in Figure 4.8(a). If the column position does not coincide with a butt joint in the decking, the continuous decking sheet may have to be cut to fit around it. At this position, the decking should then be treated as if it was simply supported, and props may be required locally. A similar situation may arise when flange splice plates are fixed to the top of the steel section, as shown in Figure 4.8(b).

Supports may also be needed if the decking is to be penetrated by temporary works structures (depending on the size of the penetration). To avoid problems in such situations, it is vital that there is good communication between the Principal Contractor, who is responsible for the temporary works, and the PSD, who should specify the appropriate steelwork.

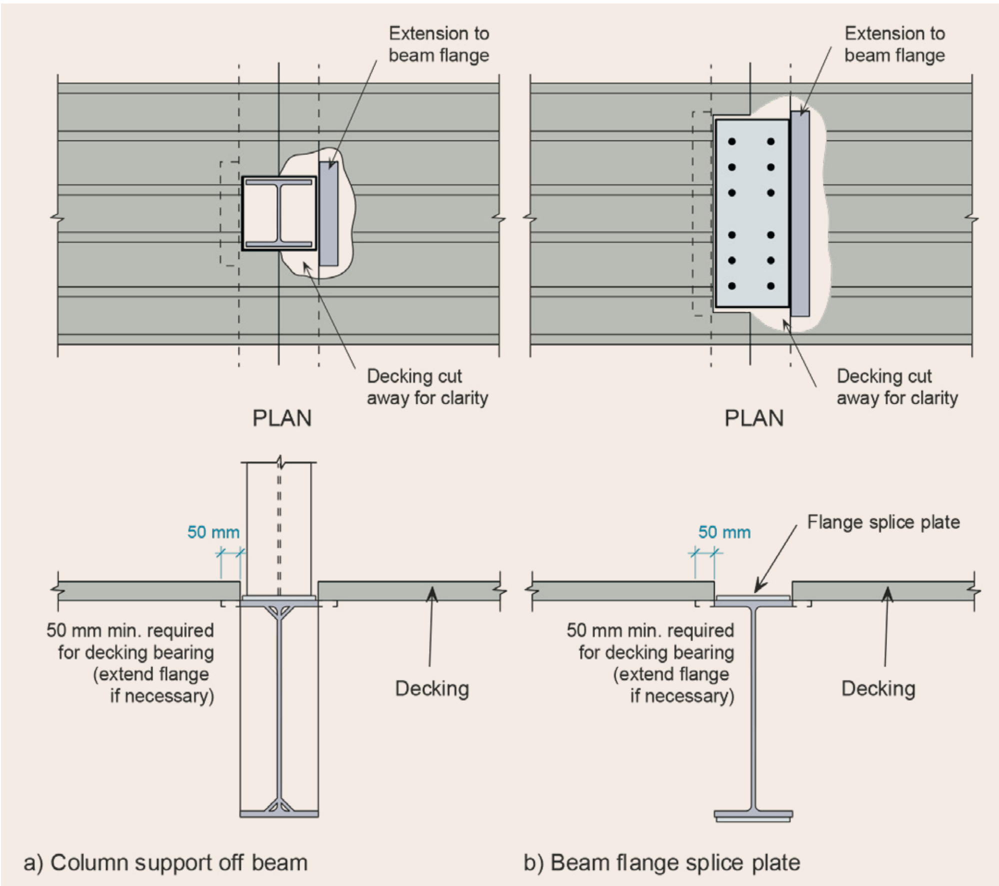


Figure 4.8 Decking details where a column is supported off a beam and where a beam flange plate occurs

The decking should be cut to fit around any penetration. A typical detail, with a column, is shown in Figure 4.9.

If temporary propping is proposed as a support around a penetration, this will clearly only be present during the construction stage, i.e., to support the decking.

Figure 4.9 Typical detail of decking installation around a column

courtesy of SMD

The completed slab may then need to include additional reinforcement, as might be necessary around any untrimmed opening in a reinforced concrete slab (see Section 4.2.6), in order to support the in-service loads. This reinforcement should be specified by the PSD.

Sheet lengths

BS EN 1090-4:2018[17] specifies the functional tolerance in the sheet lengths for decking up to 3 m in length as +10 mm and – 5 mm. A positive tolerance may result in accumulations in length when sheets are butted in a long run. Long sheets could lead to the butt joint positions becoming increasingly displaced thus giving inadequate bearing for the sheets near the end of a run. Cutting on site might be needed to overcome this problem. It is, therefore, easier for the decking to be installed when sheets are slightly short and good practice would be to aim for +0 mm. A small gap between sheets above the supporting beams is of no structural significance.

Preformed ends

Preformed ends (sometimes called ‘engineered ends’ or ‘crushed ends’) are the result of the process of closing the ends of the steel sheet deck by compressing the rib to create a slope at the end of the decking and forming a trapezoidal shape with a flat ‘toe’ at either end of the panel. An example is shown in the Figure 4.10. Decking profiles with preformed ends are used in single span conditions. The advantages of using preformed ends include:

- Reduces the installation time in single span situations as it means that no end caps need to be fitted, saving labour costs
- Avoids the need for acoustic or fire profile fillers
- Stud resistance may be increased because less volume of concrete is ‘removed’ ahead of the studs than when transverse ribs run across the beam (and, according to Eurocode 4, a reduction factor k_t must be applied). With preformed ends the concrete around the studs is similar to that found in a solid slab, or when (wide) parallel ribs are present.

There are also some disadvantages, including:

- Implications on sheet bundling and layout configurations.
- The vertical shear resistance of the decking may be reduced because the process of compressing the rib to create a slope potentially reduces the proportion of decking contributing to the shear resistance (those parts that are 'vertical'). However, the design of single span decking is normally governed by limits placed on the allowable deflection, so any reduction in shear capacity is unlikely to be critical.
- The necessary bearing width may be affected by the form of the decking ends. As this will be dependent on geometry, the particular manufacturer should be consulted.

Figure 4.10
Preformed ends

courtesy of Kingspan

4.2 Composite slabs

Composite slabs are normally used to span between 3 m and 4.5 m onto supporting (downstand) beams or walls. The ability of the decking to support the construction loads, without the need for temporary propping, generally dictates the maximum span that can be achieved (longer spans are possible when props are used to avoid the construction stage being critical). Slab thicknesses are normally in the range of 100 mm to 250 mm for shallow decking. When deep decking is used to span between shallow floor beams, the slab span is typically 6 m and depth, in the range of 280 mm to 320 mm.

When the concrete has gained sufficient strength, it acts in combination with the tensile strength of the decking to form a 'composite' slab. It can be considered as a reinforced concrete slab, using the decking as external reinforcement.

The load carrying capacity of composite slabs is normally dictated by the (mechanical) shear bond between the decking and the concrete, rather than by yielding of the decking. From tests, it is known that this shear bond generally breaks down when a 'slip' (relative displacement between the decking and the concrete) of 2 to 3 mm has occurred at the ends of the span. In practice, this will not occur below ultimate load levels. An initial slip, which is associated with the breakdown of the chemical bond, may occur at a lower level of load. The interlock resistance that is relied upon in design is therefore solely due to the performance of the embossments in the deck (which cause the concrete to 'ride-over' the decking), and the presence of re-entrant parts in the deck profile (which prevent the separation of the deck and the concrete).

Information on improving the bending resistance of composite slabs by providing additional reinforcement, or end anchorage such as shear connectors to further control slip, can be found in BS EN 1994-1-1:2004 and BS 5950-4.

If the slab is unpropped during construction, the decking alone resists the self-weight of the wet concrete and construction loads. Subsequent loads are applied to the composite section. If the slab is propped, all of the loads have to be resisted by the composite section. Surprisingly, this can lead to a reduction in the imposed load that the slab can support, because the applied horizontal shear at the decking-concrete interface increases. However, for both unpropped and propped conditions, resistances well in excess of loading requirements for most buildings can be achieved.

Composite slabs are usually designed as simply supported members in the normal condition (i.e. at ambient temperature), with no account taken of the continuity offered by any reinforced concrete that is continuous over the supports. Two methods of design are generally recognised, both of which use empirically derived information on the 'shear bond' resistance of the slab from uniformly distributed loading arrangements. The more traditional method, and one which is given in both BS EN 1994-1-1:2004 and BS 5950-4, is the so-called '*m* and *k*' method (see Section 4.2.3). However, this method has limitations and is not particularly suitable for the design of slabs subject to concentrated line and point loads. An alternative method of design is included in the Eurocode, which is based on the principles of partial shear connection. This method provides a more logical approach to determine the slab's resistance. It is likely that the '*m* and *k*' method will not be retained in the Generation 2 BS EN 1994-1-1, as by now all manufacturers know the shear bond properties needed to apply the partial connection method for their decks. It is not normally necessary for designers to understand the design methodology in detail, as manufacturers normally present the design data in the form of load-span tables or software. Tables are normally only applicable for uniformly loaded conditions.

4.2.1 Concrete

Concrete types

Both normal weight concrete and lightweight concrete are used in composite slabs, but in the Eurocodes, these are now referred to as normal concrete and lightweight aggregate concrete respectively. Normal concrete is made using dense aggregates. Lightweight aggregate concrete contains artificially produced aggregates such as expanded pulverised fuel ash pellets. The cement and water contents are higher in lightweight concrete because of the absorption of water by the aggregate. For normal weight concrete, strength classes C25/30, C28/35 or C32/40 are normally chosen; for lightweight concrete, strength classes LC25/28, LC28/31 or LC32/35 are typical. Some of these strength classes are not specified in EN 1992-1-1:2004[18] or BS EN 206:2013[19] but are in the complementary British Standard BS EN 8500-1[20]. Designers should be aware that different codes cover different classes.

Lightweight concrete has the obvious advantage that (typically) 25% weight savings can be made and provide economic benefit for the overall design of the structure and its foundations (see Section 4.1.2 for concrete densities used for design). Lightweight concrete also has better fire insulating qualities than normal weight concrete, and so thinner slabs may be possible when the 'fire condition' criterion of insulation governs the slab design (see Section 4.2.5). Unfortunately, lightweight concrete is not always readily available, and is more expensive than normal weight concrete. Also, it may not be appropriate if it is to be used in heavily trafficked areas; to achieve a good wearing surface, the finishing process must cover the particles of lightweight coarse aggregate with an adequate, well-trowelled dense surface mortar layer. It also has poorer sound insulation properties than normal weight concrete.

> Lightweight concrete offers several performance advantages, but it is not always readily available and is more expensive.

Low Carbon Concrete

So-called low carbon concrete may be used to significantly reduce the embodied carbon in a structure, although care is needed to ensure all properties of such material are satisfactory. All concretes to BS 8500 are based on Portland cement, or CEM1, but mostly contain additions, or other cementitious materials. These include:

- Ground granulated blast-furnace slag (GGBS)
- Fly ash
- Silica fume
- Limestone powder
- Pozzalana.

These additions have a much lower embodied carbon than CEM1. Table 4.1 gives an indication of the savings that can be achieved by different alternatives.

Table 4.1 Embodied CO_2 of UK concretes complying with BS 8500 (based on a cement content of 320 kg/m^3 of concrete).

Broad designation of cement type in concrete	Percentage of addition	Embodied CO_2 $kgCO_2/m^3$ of concrete
CEM1	0	283
IIA	6 – 20	228 – 277
IIB	21 – 35	186 – 236
IIIA	36 – 65 GGBS	120 – 198
IIIB	66 – 80 GGBS	82 – 123
IVB	36 – 65 fly ash for pozzalana	130 – 188

To supplement concretes in accordance with BS 8500, most of the larger concrete producers have low carbon proprietary concretes. These are formulated to keep the embodied carbon down to a given level and may therefore be particularly interesting to the specifier. Although the potential benefits may be significant, care is needed when specifying alternatives to concrete covered by BS 8500. This is because concrete tends to be specified on the basis of compressive strength alone (other than any special requirements for pouring etc). However numerous other concrete characteristics will, or could, affect the behaviour of composite construction; including:

- Tensile strength
- Long-term behaviour
- Fire behaviour (loss of strength with temperature and insulation)
- Density
- Rate of strength gain.

The relationships between these various characteristics are only guaranteed, such that the material can be specified on the basis of compressive strength alone, for concrete mixes complying with BS 8500.

Therefore, any designer, sub-contractor or manufacturer considering using concrete that is not covered by the scope of BS 8500 – whether to reduce carbon or for any other reason - should ensure that all relevant properties are known for the concrete they are considering, and justify the assumed performance of the composite construction. Doing this correctly, in consultation with all relevant parties involved in the design, material supply and construction of the project, should ensure that significant benefits are achieved without structural performance being compromised. For further information see *Low carbon concrete – what you need to know*[21].

Concrete grade

The PSD chooses a concrete specification that is suitable for the intended application. This specification is normally chosen on the basis of the:

- Overall structural requirements
- Floor finish, if any, to be placed on the slab
- Exposure conditions.

The concrete strength class designations according to BS EN 206:2013 and BS 8500-1 relate to the characteristic strength (95% probability of being exceeded) achieved after 28 days. This is the strength that would be found in a cylinder or cube test. The cylinder strength is about 80% of the strength of a 150 mm cube. Design standards provide rules that relate the design strength to the concrete grade.

As a minimum, concrete of strength class C25/30 or LC25/28 should be specified. In the case of concrete used as a wearing surface, the minimum strength class should be C28/35 (although C32/40 is preferred).

Surface finishes

There are two basic performance conditions; concrete to be used as a wearing surface, and concrete that is to be covered by raised floors, screeds, carpets, tiles, sheet vinyl, etc. When the concrete is to be used as a wearing surface, the concrete is first power floated. The specification should then require the slab to be allowed to stiffen for a short time prior to power trowelling, which compresses and polishes the surface material, resulting in a harder and more durable surface. Recommendations for power floating and power trowelling are given in BS 8204[22] and Concrete Society Technical Report 34[23].

When the concrete is not to be used as a wearing surface, it is recommended that a wood floated, skip floated or power floated finish is specified.

Drying

Because the concrete is only exposed on the upper surface of a composite floor, it can take a longer period than a traditional reinforced concrete slab to dry out. If moisture sensitive floorings and/or adhesives are to be applied, many months may be needed before the slab is sufficiently dry to accept them. Measures such as the specification of special concrete, dewatering or surface vapour-proof membranes, may need to be considered if inadequate time for drying is allowed in the contract programme.

If surface vapour-proof membranes are used, moisture will be trapped in the slab. This trapped moisture will not be detrimental to the concrete or the decking, as the steel in contact with the concrete is prevented from corrosion by its high pH. Although sometimes considered, the provision of small holes, perforations, in the decking to aid drying is ineffective; the area represented by the holes is insufficient to have any significant effect on drying times.

AD 163[24] provides additional guidance on provisions for water vapour release.

Level and flatness

Construction tolerance specifications available for composite floor slabs are BS EN 13670:2009[25], which is adopted by NSCS[26], and BS 8204-2. Information taken from all three is presented in Table 4.2. NSCS Basic or BS 8204-2 SR3 are considered as a default. If a tighter tolerance than Basic or SR3 is required, then a stiffer structure may need to be considered. This could result in a combination of larger steel sections, shorter deck spans, more frequent support columns and or heavier gauge steel decking. Where a strict control to datum is required, it is suggested that the defection of the steel design be limited to 10 mm. The deviation in surface flatness is determined by measuring the maximum gap beneath a straight edge laid on the surface. For composite slabs, the straight edge must always be positioned parallel to the supporting beams, i.e. perpendicular to the decking span.

	NSCS		BS EN 13670		BS 8204-2		
	Basic	**Ordinary**	**Non-moulded**	**Moulded**	**SR3**	**SR2**	**SR1**
Permitted global deviation (mm) under a 2 m edge	12	9	15	9	10	5	3
Permitted local deviation (mm) under a 200 mm edge	5	3	6	4	na	na	na
Difference in height across a joint (mm)	na	na	na	na	2	2	0

Notes:
SR1 is unlikely to be achievable on suspended floors of any construction.
SR2 may be achievable on parts of a composite floor, but will not be achievable over all of a floor, owing to deflections. This is a tight flatness tolerance and high levels of workmanship are required to achieve SR2 on any type of suspended floor.
SR3 may be achievable over most of a floor, **depending on the deflections of the supporting beams**.

Table 4.2 Flatness defined by deviation under a straight edge

For floor slabs used by manual handling equipment, tighter tolerances may be required as specified in TR34. Some modern applications require even tighter tolerances, for example to facilitate the smooth running of robots in distribution centres. However, it is recommended that a precisely level and flat concrete floor is not specified unless it is absolutely necessary. When tamping rails are used, noting they are not so common in current practice, they are usually positioned along the support beams, which deflect under the self-weight of the finished floor. To achieve greater accuracy, it is necessary to estimate the central deflection of the beams and to set the tamping rails along each beam to allow for this deflection. This can result in errors because, in practice, the beams may not deflect as much as expected (e.g., because of the stiffness of the beam-to-column connections). It is reasonable to set the rails on the basis that the beams will deflect 30% less than predicted by simple theory. Within the span of the decking the slab depth will increase as the decking deflects – however this increase is likely to have been accounted for in the design through consideration of so-called ponding. Dipping is a more effective way of achieving constant thickness, and commonly used.

In propped construction, further deflection occurs on removal of the props. Subsequent deflections will be greater the earlier the props are removed (due to the lower stiffness of the ‘immature’ concrete). Therefore, props should not be removed until the concrete has reached 75% of its design strength.

As deviations in level of the upper surface are dependent on the deflection of the decking and the supporting beams, tolerances within which these deviations must lie should only be specified at points where there is negligible deflection of the supporting structure, i.e., at columns. The Principal Contractor will be able to do little to correct matters if deviations exceed tolerances specified at other points.

The following tolerances are recommended, for verification only at such points in the structure:

Top surface of concrete, level to datum	± 15 mm
Top surface of supporting steel beams, level to datum	± 10 mm

Specifying a uniform thickness, rather than level of the upper surface, is the preferred solution. It means that the self-weight of the concrete will remain close to that assumed in the design.

Further information on level and flatness can be found in AD 344[27], AD 410[28], TR75[29] and Concrete advice sheet No. 65[30].

4.2.2 Reinforcement

4.2.2.1 Bar and fabric (mesh) reinforcement

Types and details

Reinforcement in composite slabs usually takes the form of a relatively light welded fabric, supplemented by bar reinforcement where required. The fabric reinforcement is required to perform a number of functions:

- Provide bending resistance over internal supports of the slab in the fire condition (this reinforcement is usually ignored under ‘normal’ load conditions)
- Reduce and control cracking at the supports, which occurs because of flexural tension and differential shrinkage effects
- Distribute the effects of localised point loads and line loads (see Section 6.3.2)
- Strengthen the edges of openings (see Section 4.2.6)
- Act as transverse reinforcement for the composite beams (see Section 5.3.2).

The most common fabric sizes are A142 and A193 (using designations according to BS 4483[31]), with the numbers indicating the cross-sectional area (mm^2) of reinforcing bars per metre width. The fabric is normally manufactured in ‘sheets’ that are 2.4 m wide and 4.8 m long. ‘A’ type fabric has layers of bars equally spaced in both directions (known as ‘square’ fabric) and is most commonly used. It is possible to order special fabric with heavier wires or closer spacing in one direction, such as ‘B’ or ‘C’ type fabrics. ‘B’ type ‘structural’ fabrics have longitudinal bars at 100 mm centres and transverse bars at 200 mm centres. These can be used when highly reinforced areas are required for structural or fire resistance purposes. ‘C’ type ‘highway’ fabrics are intended for highway use and have only very light reinforcement in the transverse direction. C type fabrics should not be used in composite floors.

> Fabric sizes less than A142 are not recommended because of their poor performance as fire reinforcement and inability to control shrinkage. They are considered as non-structural.

Bar reinforcement may be used to supplement the fabric:

- To achieve longer fire resistance periods
- To reinforce the slab around significant openings
- When additional transverse reinforcement is needed
- To achieve greater crack control.

Reinforcement should comply with BS 4483[31] (fabric) or BS 4449[32](bar) and the detailing of it should be in accordance with BS EN 1992-1-1:2004 or BS 8110[33] and BS 8666[34]. Bar reinforcement is produced in three ductility grades: A, B or C. As noted in Section 2.1.1 and 5.6.3 of BS EN 1992-1-1:2004, to ensure that the reinforcement has sufficient ductility for plastic analysis, Class B or Class C should be specified (B being the most common). The ductility grade of the reinforcement has no effect on the lap and anchorage lengths. The bars in fabric supplied to BS 4483 are ribbed, and this will reduce the required anchorage lengths compared to plain bars. BS EN 1992-1-1:2004 assumes that bars are ribbed, but BS 8110 allows for the use of ribbed and plain bars.

In shallow composite slabs, the reinforcement should be supported sufficiently high above the top of the decking to allow concrete placement around the bars. When the slab depth does not allow this, the effect of localised loss of anchorage over the crests of the deck should be taken into account. The required cover to the upper surface depends on the concrete class and the exposure. Recommendations are given in Table A.4 and A.5 of BS 8500-1. The PSD should determine the relevant exposure condition for the top of the floor, considering the following:

- For a floor in a dry protected environment, e.g., in enveloped buildings such as offices, the exposure class for the concrete is XC1
- For an external floor exposed to high levels of humidity, the exposure class for the concrete would be XC3 or XC4
- For a floor exposed in a marine environment, the exposure class would be XS1, XS2 or XS3
- For a floor that is exposed to freeze-thaw cycles, the exposure class would be XS (see BS 8500-1 for recommendations for this class).

Table A.4 in BS 8500-1 applies where the intended working life is 50 years and Table A.5 applies where the intended working life is 100 years (which is not normally applicable to buildings).

In car parks, where the slab is exposed to chlorides and freeze-thaw attack, the exposure class is XD3 or, if the intended design life does not exceed 30 years, the exposure class is XF3 or XF4, provided that the concrete surface is protected by an effective, durable and long-lasting waterproof membrane. Any membrane should be a waterproof coating that prevents the ingress of water containing dissolved de-icing salts into the concrete, including at any joints and cracks in the concrete.

Recommended covers for XC1 and XC3/4 exposure classes are given in Table 4.3. Reference should be made to BS 8500-1 for covers and concrete specifications for other exposure classes.

The recommendations for durability in this section only relate to the concrete and reinforcement. The corrosion protection of the metal decking is covered in Section 4.1.1

Table 4.3 Concrete properties and reinforcement cover for various levels of exposure

Aggregate Type	**Normal weight**					**Lightweight**		
Concrete Strength Class	**C25/30**	**C28/35**	**C32/40**	**C35/45**	**C40/50**	**LC25/28**	**LC28/31**	**LC32/35**
Max water cement ratio	0.65	0.60	0.55	0.50	0.45	0.65	0.60	0.55
Min cement content for 20 mm aggregate (kg/m^3)	260	280	300	320	340	260	280	300
Min cement content for 14 mm aggregate (kg/m^3)	280	300	320	340	360	280	300	320
Min cement content for 10 mm aggregate (kg/m^3)	300	320	340	360	360	300	320	340
Nominal cover in mm to reinforcement according to the exposure level:								
XC1	25	25	25	25	25	25	25	25
XC3/4	45	40	35	35	30	45	40	35

Notes:

(a) These values are taken from BS 8500-1 and BS EN 206

(b) The exposure conditions are defined in BS 8500-1. For internal floors inside enclosed buildings, with dry conditions the exposure condition would be XC1. For floors subject to high humidity or cyclical wet and dry conditions the exposure condition would be XC3/4. More severe exposure conditions may be applicable in some conditions, e.g., car parks.

(c) Nominal Cover: BS 8500-1 lists minimum covers not nominal covers. The nominal covers listed in Table 4.3 are the minimum covers given in BS 8500-1 plus a fixing tolerance (Δc) of 10 mm. The covers listed are for an intended working life of 50 years. For an intended working life of 100 years no change is required to the XC1 exposure class covers.

(d) In practice, nominal covers less than 30 mm with light fabrics are not recommended owing to practical difficulty in supporting the light fabric in the correct location.

(e) The listed covers are for durability purposes. Greater covers may be needed for fire resistance considerations

Recommended tension laps and anchorage lengths for welded fabric and bars for design to BS 8110 are given in Table 4.4, and for design to BS EN 1992-1-1:2004 in Table 4.5.

Traditional practice in the UK uses any continuity over the supports, combined with a small contribution from the decking in the span, to provide composite slab hogging and sagging moment resistances that are sufficient to support the loads in a fire situation. When there is no physical continuity at the slab ends the sagging resistance alone will not be sufficient to resist loads in a fire unless bars are placed in the troughs to act as an additional lower layer of reinforcement and ensure adequate sagging resistance. The Generation 2 BS EN 1992-1-1:2004 provides the facility to consider shorter anchorage lengths when the stress in reinforcing bars is below yield. For situations with uniform loading, or predominantly uniform loading, it can be concluded that straight bars placed in the troughs with no special provision for end anchorage will be adequate. As with all composite elements, when the loading is heavily non-uniform, specific checks should be carried out. For further information see AD note 461[35]

Aggregate Type		Normal			Lightweight		
Strength Class		**C20/25**	**C25/30**	**C28/35**	**LC28/35**	**LC32/35**	**LC32/40**
Reinforcement Type	**Wire/Bar Type**						
Grade 500 Bar of diameter *d*	Deformed Type 2	44*d*	40*d*	38*d*	56*d*	54*d*	50*d*
A142 Fabric (6 mm wires at 200 mm centres)	Deformed Type 2	275	250	250	350	325	300
A193 Fabric (7 mm wires at 200 mm centres)	Deformed Type 2	300	275	275	400	375	350
A252 Fabric (8 mm wires at 200 mm centres)	Deformed Type 2	350	325	300	450	425	400
A393 Fabric (10 mm wires at 200 mm centres)	Deformed Type 2	440	400	375	550	550	500

Notes:
(a) Table 4.4 is based on information given in BS 8110-1[33], assuming fully stressed bars/fabric. It should be noted however that the recommendations determined in accordance with BS EN 1992-1-1:2004 (as shown in Table 4.5, below) may differ from the above.
(b) Where a lap occurs at the top of a section and the minimum cover is less than twice the size of the lapped reinforcement, the lap length should be increased by a factor of 1.4.
(c) Deformed Type 2 Bars/Wires: Bars with transverse ribs of substantially uniform spacing, which protrude beyond the main round part of the bars/wires. There may be longitudinal ribs. Note: The majority of deformed high yield reinforcement available in the UK is Type 2.
(d) The minimum Lap/Anchorage length for bars and fabric should be 300 mm and 250 mm respectively.

Table 4.4 Recommended tension laps and anchorage lengths for welded fabric and bars to BS 8110

			Reinforcement in tension, bar diameter, *d* (mm)							
		Bond condition	**8**	**10**	**12**	**16**	**20**	**25**	**32**	**40**
Anchorage length, l_{bd}	Straight bars only	Good	230	320	410	600	780	1010	1300	1760
		Poor	330	450	580	850	1120	1450	1850	2510
	Other bars	Good	320	410	490	650	810	1010	1300	1760
		Poor	460	580	700	930	1160	1450	1850	2510
Lap length, l_0	50% lapped in one location (α_6 = 1.4)	Good	320	440	570	830	1090	1420	1810	2460
		Poor	460	630	820	1190	1560	2020	2590	3520
	100% lapped in one location (α_6 = 1.5)	Good	340	470	610	890	1170	1520	1940	2640
		Poor	490	680	870	1270	1670	2170	2770	3770

Notes:
(a) Nominal cover to all sides ≥ 25 mm, [i.e. $\alpha_2 \leq 1$]. At laps, clear distance between bars ≤ 50mm
(b) $\alpha_1 = \alpha_3 = \alpha_4 = \alpha_5 = 1.0$. For the beneficial effects of shape of bar, cover and confinement see Eurocode 2, Table 8.2
(c) Design stress has been taken as 435 MPa. Where the design stress in the bar at the position from where the anchorage is measured, σ_{sd}, is less than 435 MPa the figures in this table can be factored by $\sigma_{sd}/435$. The minimum lap length is given in cl 8.7.3 of Eurocode 2
(d) The anchorage and lap lengths have been rounded up to the nearest 10 mm
(e) Where 33% of bars are lapped in one location, decrease the lap lengths for '50% lapped in one location' by a factor of 0.82
(f) The information in this table is taken from *How to design concrete structures to Eurocode 2* . This publication should be consulted for further guidance
(g) The figures in this table have been prepared for concrete class C25/30; for other classes see Reference 36 or use the following factors for other concrete classes.

Concrete class	C20/25	C28/35	C30/37	C32/40	C35/45	C40/50	C45/55	C50/60
Factor	1.16	0.93	0.89	0.85	0.80	0.73	0.68	0.63

Table 4.5 Recommended tension laps and anchorage lengths for welded fabric and bars to BS EN 1992-1-1:2004 in C25/30 concrete

4.2.2.1 *Fibre reinforcement*

Fibre reinforcement consists of short fibres made from steel, polypropylene or a combination of both, which are mixed into the concrete prior to placement. Under controlled circumstances, fibres may be substituted for some or all of the fabric reinforcement. Use of fibre reinforcement results in a three-dimensional reinforced concrete composite slab. Only steel fibres are used for structural applications, polypropylene fibres are primarily used to control plastic shrinkage cracking and BS EN 1992-1-2 suggests the addition of polypropylene fibres as one of the methods used to reduce spalling for high strength concrete[37].

The performance of fibre reinforcement is verified empirically, specifically for fire resistance and for longitudinal shear transfer, using the same testing regimes that are used to validate the use of traditional reinforcement within steel deck composite floors.

Considerable benefits can be achieved using fibre reinforcement, including a reduction in labour costs and a saving on the construction programme. The requirement for longitudinal shear reinforcement to composite beams, in the form of bars or fabric, can be dramatically reduced and only a minimal amount of fabric reinforcement need be purchased, transported and stored. There is less usage of the crane as there are fewer lifting operations. The installation of the floor is easier and safer because there is less reinforcement to obstruct the floor working area to handle, fix and check, and this can reduce installation times by up to 20%.

Independent testing has shown that fibre reinforcement systems can provide an equivalent or superior performance to traditional welded wire fabric solutions, although local reinforcement may be necessary in locations of concentrated loads. Fibre reinforcement provides resistance to plastic shrinkage, settlement cracking and toughness, but the level of performance is related to the specific fibre.

BS EN 14889[38] covers the requirements for fibres used as concrete reinforcement. BS EN 14889-1:2006 covers steel fibres, and BS EN 14889-2:2006 polymer fibres. Care should be taken when selecting fibres as a replacement for traditional fabric reinforcement in steel deck composite slabs.

It is very important to note that fibre reinforced composite slabs are not a generic product. A specific type and dosage of fibres must be used according to the fibre manufacturer's specification for a particular slab, and other fibres cannot be substituted. Provided they are correctly specified, they are unlikely to change the situation commonly found when fabric is used, whereby the construction stage governs. If the fibres are not already in a ready mixed concrete when delivered to site, care should be taken to ensure their effective mixing and even distribution during pouring. It can be difficult to control and check the even distribution of fibres. Although offsite mixing is associated with high quality control, contractors and suppliers should ensure that every effort is made for checking the quality of the pour and the even distribution of fibres in the concrete mix during construction of the slab[39].

More information on fibre reinforcement is given in Section 6.2.1. Further guidance on the use of steel or macro-synthetic fibre-reinforced concrete can be found in Concrete Society Technical Reports No. 34, 63[40] and 65.

> Fibre reinforcement is normally provided within the concrete that is delivered and ready to pump on site. This can reduce installation times by up to 20%.

4.2.3 Design for resistance

The performance of a composite slab with a particular decking profile can only be assessed readily by testing. Test procedures are set out in both BS EN 1994-1-1:2004 and BS 5950-4. Sufficient tests must be undertaken to provide information covering the full range of application. The specimens are first subject to dynamic load (5,000 load cycles up to 1.5 times the working load are specified in BS EN 1994-1-1:2004, but 10,000 cycles are required by BS 5950-4). Following this, a static load is applied and increased until failure occurs. The objective of the dynamic part of the test is to break any adhesion bond, so that only the more stable mechanical interlock remains. It is anticipated that the Generation 2 EN 1994 will recognise that far fewer than 5,000 cycles are needed to achieve this objective, and the requirement will be adjusted accordingly.

The test procedure is such that all loads are applied to the composite section to simulate a uniformly loaded condition. The test results are then presented in terms of empirical constants, either (m and k) or Tau (τ), that can be used to quantify the interaction between the steel and concrete.

As far as slab design for a specific project is concerned, the PSD will not undertake tests to determine the m and k or Tau (τ) factors. These constants are used by the decking manufacturers themselves in their software, or in load-span tables for uniformly loaded conditions for their specific products.

Concentrated line or point loads pose a particular problem for composite slabs because only part of the slab can be mobilised to support them. One of the issues with concentrated loads is that the width of longitudinal strip (in the direction of the slab span) that is assumed to carry the load can become quite large, when the load is near to mid-span. Such a width may require significant reinforcement for it to be able to carry the load in transverse bending. This situation is further complicated by the fact that, often, construction loads may be significant and not allowed for at the design stage (so the required transverse reinforcement is not present). The need to mobilise a significant area of slab also has implications when more than one concentrated load is applied. SCI has produced guidance (AD 450[41] and AD 477[42]) that shows how to design the transverse reinforcement, and indeed how to reduce the width of the longitudinal strip in order to find the best compromise for transverse requirements. It may also be used to justify the transverse reinforcement that is present when an 'unexpected' construction load, such as that from a MEWP, is applied. This process is implemented in an SCI Tedds module. The use of load-span tables, based on uniform loading, is clearly also unjustified when significant concentrated loads are present. See Section 6.3.2 for further information on concentrated line or point loads.

The vertical shear resistance of a reinforced composite slab using bar or fabric reinforcement has traditionally been assessed as for a reinforced concrete slab, using guidance given in BS EN 1992-1-1:2004 or BS 5950-4. However, Generation 2 EN 1994-1-1 will include a revised model that recognises a contribution from the decking, that supplements the concrete resistance. Whilst the Eurocode is still under development, several teams of researchers have published work that supports the general conclusion that the decking also contributes. SCI currently advocates the use of a 'first principles' model that is closely based on what has been used in France for almost a decade. This combines the resistance of the concrete part, in accordance with BS EN 1994-1-1, with the shear buckling resistance of the decking webs determined in accordance with BS EN 1993-1-3. As advocated by SCI, the model includes a proposed reduction in the contribution of the decking to recognise that some coincident moment may be present (this reduction brings the method in-line with what is expected to be the final method presented in EN 1994). It is assumed that the end anchorage needed to generate this level of shear buckling resistance is present. The method is fully described in SCI's Eurocode Nugget *Vertical resistance of composite slabs*[43]. 'Punching' shear resistance, against localised loads, should also be assessed using these Standards. When fibre reinforcement is used, designers should seek guidance from the manufacturers.

> Manufacturers' load-span tables for slabs are normally based on testing. Designers should take care to ensure that they do not use this information for situations that are not covered by the scope of the testing especially if concentrated line or point loads are applied to the slab. Designers should always read the notes associated with the load-span tables presented by the manufacturer as these may differ.

4.2.4 Design for serviceability

Crack control

There is a risk of cracking in the concrete in all composite slabs due to the restraint to drying shrinkage provided by the steel decking and primary steelwork, even though the decking effectively acts as reinforcement and helps to distribute the shrinkage strains so that large cracks do not form. However, cracks do not normally pose a durability or serviceability hazard for situations where composite slabs are commonly used. Only where the surface of the slab is used as a wearing surface, or where terrazzo or other 'rigid' floor coverings are to be used, may specific reinforcement (in addition to the 'standard' fabric) be required in order to control the cracking. When cracking is an issue, reinforcement percentages in excess of 0.3% will normally be required in order to limit crack widths to the recommended values. Fabric, rather than bars, is generally used to control cracking.

According to BS EN 1994-1-1:2004, when continuous slabs are designed as simply supported in the normal condition, the minimum cross-sectional area of the anti-crack reinforcement within the depth of the concrete cover to the decking should be as follows:

- 0.2% of the cross-sectional area of the concrete above the ribs for unpropped construction
- 0.4% of the cross-sectional area of the concrete above the ribs for propped construction.

It is possible that larger crack widths will occur over the intermediate supports with propped construction, because the full self-weight of the slab is applied to the composite slab on removal of the props, which explains the higher minimum percentage reinforcement required.

The above amounts do not automatically ensure that the crack widths are less than the typical value of 0.3 mm given in BS EN 1992-1-1:2004 (and the UK National Annex to this code) for certain exposure classes. If the exposure class (or the floor finish) is such that cracking needs to be controlled, the slab should be designed as continuous, and the crack widths in hogging moment regions evaluated according to BS EN 1992-1-1:2004.

Experience shows that the greatest risk of cracking is over supporting beams, owing to the combination of restrained drying shrinkage and flexural action[44]. 'Induced' joints may be used to reduce the risk of random cracking at these locations. Such joints can be formed by sawing the slab, but clearly, care is needed to prevent the reinforcement required for fire resistance and longitudinal shear resistance of the supporting beam from being damaged or severed. However, this method is not recommended and where cracking has to be controlled, a more reliable measure is to use additional crack control reinforcement at the support[45].

When fibre reinforcement is to be used in place of bar/fabric reinforcement the suppliers of the fibres should be consulted regarding crack control measures.

Deflections

Deflections due to loading applied to the composite slab should be calculated using elastic analysis, neglecting the effects of shrinkage. For an internal span of a continuous slab, where the shear connection is achieved by either mechanical or frictional interlock, or end anchorage by thru-deck welded studs, the deflection may be determined using the average value of the cracked and uncracked second moment of area. This applies both for design to the Eurocodes and design to BS 5950, but the modular ratio for long-term and short-term effects is calculated slightly differently in BS EN 1994-1-1:2004 and BS 5950-4 (which refers to BS 5950-3).

BS EN 1994-1-1:2004 permits calculations of the deflection of the composite slab to be omitted if both the following conditions are satisfied for external or simply supported spans:

- the span/depth ratio of the slab does not exceed certain limits specified in BS EN 1992-1-1:2004 for lightly stressed concrete (shown here in Table 4.6); and
- the load causing an end slip of 0.5 mm in the (long span) tests on composite slabs exceeds 1.2 times the design service load.

Table 4.6 General rules for the slab maximum span-to-depth ratios in accordance with BS EN 1992-1-1:2004

	Normal concrete	Lightweight concrete
Single spans	20	18.8
End spans	26	24.5
Internal spans	30	28.3

For cases where the end slip exceeds 0.5 mm at a load below 1.2 times the design service load, two options exist for the designer:

- end anchors should be provided; or
- deflections should be calculated including the effect of end slip.

Should the behaviour of the shear connection between the sheet and the concrete not be known from tests on composite slabs with end anchorage, BS EN 1994-1-1:2004 permits a tied-arch model to be used. The model gives the deflection that should be considered; however, the method can be over-conservative. Guidance for designers on this case can be found in the Designer's guide to BS EN 1994-1-1[46].

For design to BS 5950-4, there are also simple design rules to ensure adequate deflection behaviour of a composite slab. Calculation of deflections is not necessary when designing to this code if the span-to-depth ratios are not greater than those given in Table 4.7. Confirming that the slab satisfies these limits will ensure that excessive deflections are avoided. The effective span of the decking is defined in BS 5950-4 as the smaller of:

- the distance between the centres of the supports, and
- the clear span between the supports plus the effective depth of the composite slab.

Table 4.7 General rules for the slab maximum span-to-depth ratios in accordance with BS 5950-4

	Normal concrete	Lightweight concrete
Single spans	30	25
End spans	35	30
Internal spans	38	33

Note: Values apply to supported spans with (where relevant) nominal continuity reinforcement, subject to uniformly distributed loading

The values in Table 4.7 apply to slabs under uniformly distributed loading, with nominal continuity reinforcement (0.1%) over the intermediate supports for end and internal spans, i.e., they are designed as simply supported at both ends. For slabs with full continuity reinforcement over the supports, reference should be made to BS 8110. Deflections should be calculated explicitly for slabs that fail to satisfy the span-to-depth ratio or reinforcement limits. BS 5950-4 recommends that the deflection of the composite slab should not normally exceed the limits in Table 4.8.

Table 4.8 Recommended limits for the maximum deflection of composite slabs given in BS 5950-4

Criterion	Recommended Limit
Deflections due to the imposed load	L_s/350 or 20mm, whichever is the lesser
Deflection due to the total load less the deflection due to the self-weight of the slab plus, when props are used, the deflection due to prop removal	L_s/250

Notes:
L_s is the effective span of composite slab, which is the smaller of:
a) distance between centres of permanent supports, and
b) clear span between permanent supports plus effective depth of composite slab d_s

The stiffness of slabs reinforced with conventional fabric reinforcement can be determined using 'normal' reinforced concrete design rules (assuming fully effective bond between the decking and the concrete). When fibre reinforced concrete is used, advice on the slab stiffness should be sought from the manufacturer.

Dynamic sensitivity

The dynamic sensitivity of composite slabs is not normally critical, because they are relatively stiff compared with the beams, although the dynamics of the whole floor should be considered, as explained in Section 5.2.2.

> Cracking of internal concrete surfaces will generally not compromise the structural performance of a building, so for economic design its consequences may often be ignored.

4.2.5 Design for fire resistance

Deemed to satisfy ways of demonstrating the fire performance of floor slabs required by Building Regulations are defined by Approved Document B to the Regulations (and equivalent documents for other parts of the UK). The Approved Document requires the slab performance to be assessed based on criteria for insulation 'I', integrity 'E' and load bearing capacity 'R'. For design to the Eurocodes, BS EN 1994-1-2:2005[47] and the UK National Annex to BS EN 1994-1-2:2005[48] provide guidance on how composite slabs may be designed to meet these criteria. For UK design, guidance is available in BS 5950-8[11].

The insulation criterion is satisfied by providing adequate slab thickness to ensure that the average temperature rise over the whole of the non-exposed surface is limited to 140°C and the maximum temperature rise at any point of that surface does not exceed 180°C. BS 5950-8 and PN005: *Fire resistance design of composite slabs*[49] (for Eurocode design) provide a table of recommended concrete thicknesses to satisfy the insulation criterion for common periods of fire resistance. The minimum thickness of concrete required to satisfy the insulation requirements is shown in Table 4.9 for trapezoidal decks and Table 4.10 for re-entrant decks. Figure 4.11 shows that the insulation depth depends on the type of profile, and it is the concrete cover to the main crest of the deck for trapezoidal profiles and the full slab depth for re-entrant profiles.

Table 4.9 Minimum thickness of concrete, measured above the steel deck, for trapezoidal profiled steel deck exposed to the standard fire

Concrete type	Minimum thickness of concrete (mm) for a fire resistance period (mins) of:			
	30	60	90	120
Normal concrete (all cases)	60	60	70	80
Lightweight concrete (all cases)	50	60	70	80

Thickness is measured above the profile (See Figure 4.11)

Table 4.10 Minimum thickness of slab for re-entrant profiled steel sheets exposed to the standard fire

Concrete type	Minimum thickness of concrete (mm) for a fire resistance period (mins) of:			
	30	60	90	120
Normal concrete (all cases)	100	100	110	125
Lightweight concrete (all cases)	100	100	105	115

Thickness is measured above the profiled decking soffit (See Figure 4.11)

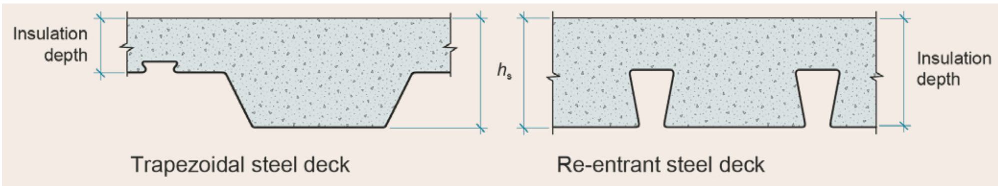


Figure 4.11 Minimum insulation depth measurement

BS EN 1994-1-2:2005 permits the designer to calculate the bending resistance and insulation properties assuming that composite slabs fulfil the integrity criterion.

Fire tests have been carried out on slabs with conventional fabric reinforcement, and on fibre reinforced slabs.

The load bearing resistance of the slab at elevated temperatures can be determined by calculation in accordance with the principles given in BS EN 1994-1-2:2005. The UK National Annex provides additional guidance on determining design temperatures for UK decking geometries.

Depending on the span required, an increased size of fabric may need to be used, or extra bars may need to be placed in the troughs of the deck, to satisfy the load bearing criteria (R) for the fire condition. In either case, the additional reinforcement is used to compensate for the loss of strength of the (exposed) decking at elevated temperatures. Design guidance covering this aspect is normally given by the decking manufacturers in their design tables and software. These tools are based on the extended application of fire test results and provide product specific guidance which will result in the most economic solutions for fire design. The extended applications of the fire test results are based on a design model for plastic resistance that is in accordance with the principles of BS EN 1994-1-2:2005 Section 4.3.1 and the recommendations of the UK National Annex to BS EN 1994-1-2:2005.

In the UK National Annex, the use of Informative Annex D of BS EN 1994-1-2:2005 is rejected as many UK decking profiles are outside the limits of the field of application. It was found that when the methods in Annex D were applied to these decking geometries, unsensible answers were obtained. SCI has produced NCCI in the form of document *PN005: Fire resistance design of composite slabs*[49] . This provides guidance on temperature profiles, and how to determine slab resistance for the resulting temperatures.

Slab designs that comply with the recommendations of BS EN 1994-1-1:2004 for room temperature design are deemed to have 30 minutes fire resistance, when assessed under the load bearing criteria 'R', but these slabs still need to be checked for compliance with the insulation criteria.

Further information on the calculation of load bearing resistance of composite slabs can be obtained from *Fire resistance design of steel framed buildings (P375)*[50] .

4.2.6 Openings and edges

Openings

Openings can be accommodated readily in composite slabs. Some advice as to limits on the size of openings, and the provision of any extra reinforcement that may be required, is normally provided by the decking manufacturer. Further advice is given here on issues relating to design for openings in slabs with shallow decking. Advice relating to the construction of openings is given in the *BCSA Code of Practice for Metal Decking and Stud Welding*.

Openings may be categorised by their size:

- **Small** - openings up to 300 mm wide. These are unlikely to present a problem structurally and do not normally require additional reinforcement. When additional reinforcement is present in the trough of the deck this must be taken into account when considering the implications of an opening (see below for further details)
- **Medium** - openings between 300 mm and 700 mm wide. These normally require additional reinforcement to be placed in the slab. This is also the case if the openings are placed close together
- **Large** – openings greater than 700 mm wide. These should be trimmed with additional permanent steelwork back to the support beams.

The dimensional limits need only apply to the width of the opening (perpendicular to the direction of span of the slab). This is because they are based on the ability of the slab, without additional measures for small openings and with additional measures for anything larger, to transfer self-weight and loads transversally between ribs[51].

It should be noted that slightly different dimensions from those given above may be quoted in a manufacturer's literature for specific profiles, in which case the manufacturer's guidance should be followed.

For small and medium size openings, normal practice is for the Principal Contractor to form an opening by 'boxing-out' an area of decking using timber or polystyrene inserts before concreting, as shown in Figure 4.12. The decking should not be cut until the concrete has gained 75% of its design strength. Then it may be cut or burnt away to form the opening, and the cut edges bent up or ground off. There are also proprietary products available which are cut to the shape of the decking profile that can be used for boxing-out and for construction joints.

If cutting the deck prior to casting is unavoidable, temporary propping is likely to be required. This may have implications on the slab design, and the PSD should be consulted.

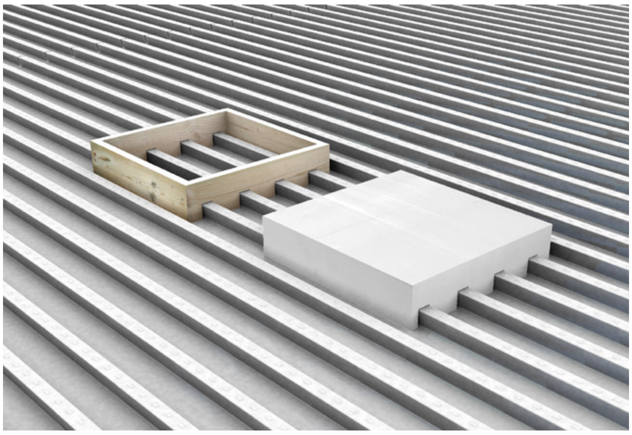


Figure 4.12 Typical examples of boxing out openings

courtesy of SMD

For large openings, the supporting trimming steel should be in position prior to placing the decking. The opening should then be trimmed prior to casting the slab, as shown in Figure 4.13.

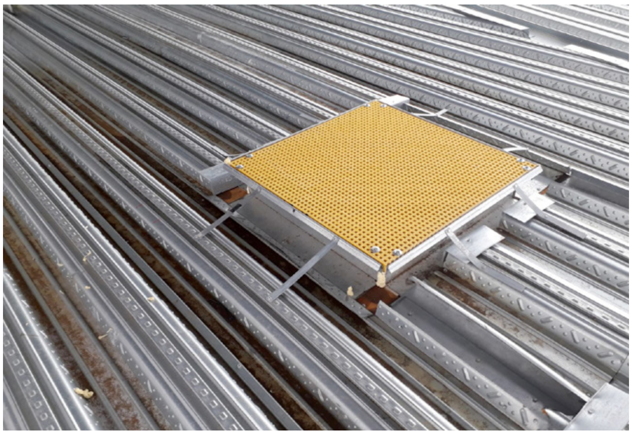


Figure 4.13 A typical trimmed opening with a GRP floor grating system immediately after deck laying, prior to fixing the reinforcement around the opening

courtesy of VoidSafe®, SMD

Cutting the slab after concreting (post-forming the openings) may cause a loss of bond between the concrete and the decking and is not recommended. When post-forming is unavoidable, non-percussive cutting methods such as diamond core drills or saws should be adopted, so that the disturbance to the mechanical interlock between the decking and the concrete is kept to a minimum. The structural implications of the location and size of the opening need careful consideration and should always be referred to the PSD.

The need for extra reinforcement in the slab, or additional trimming steelwork, depends on the size of the opening. Requirements should be determined by the PSD, who may delegate this responsibility to a slab or steelwork specialist. The PSD will identify reinforcement requirements on the contract drawings and should be consulted if there are any doubts about the location of openings or the amount of reinforcement needed. Any additional reinforcement that may be required should be designed in accordance with BS EN 1992-1-1:2004 or BS 8110.

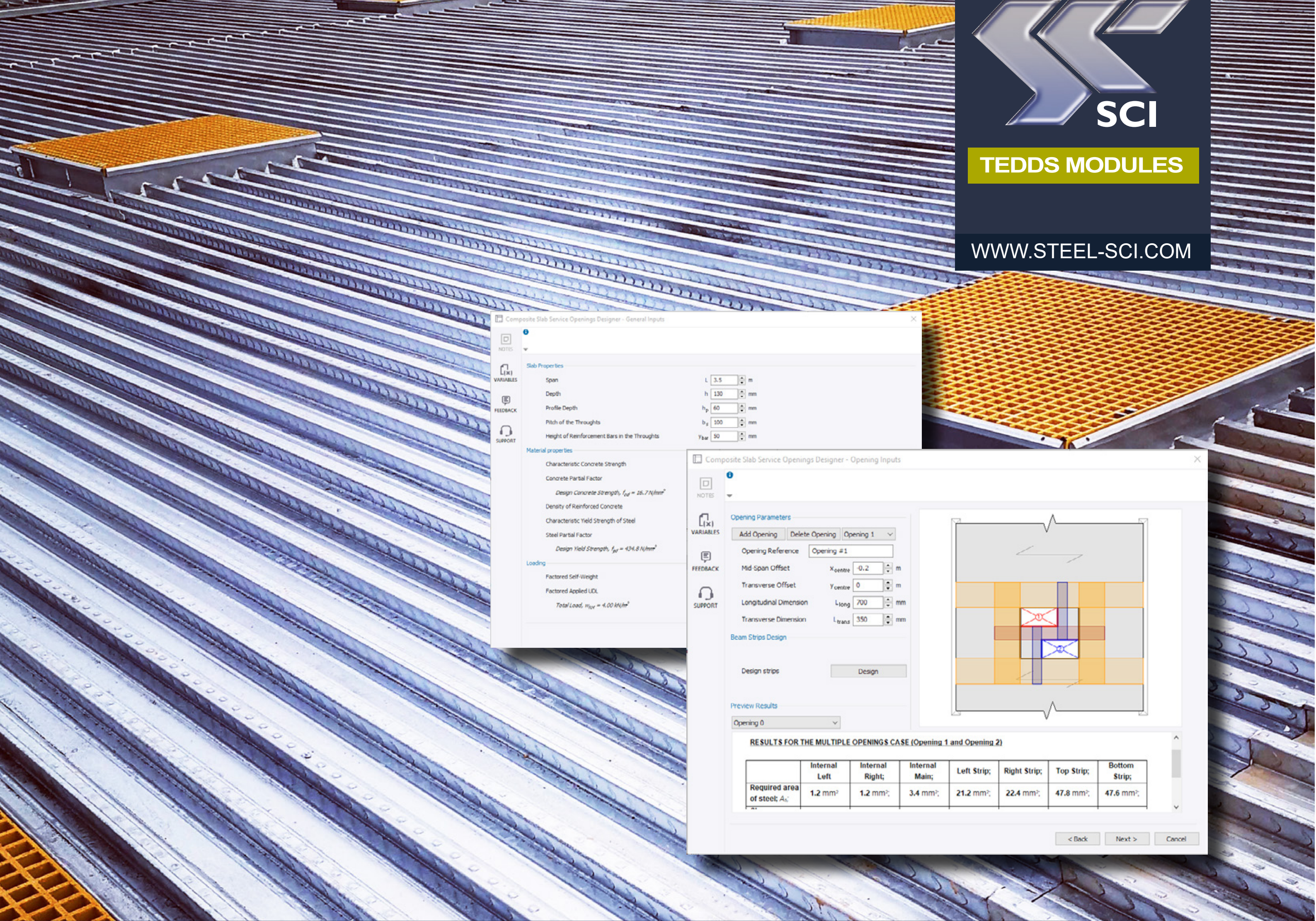
Site operatives should be made aware that additional reinforcement is required around medium sized openings. This often takes the form of bars placed in the troughs of the decking adjacent to the opening, with additional transverse bars used to 'smooth out' the load transfer around the opening (see Figure 4.15). The distance between an opening and an unsupported edge should not be less than the greater of either 500 mm and the width of the opening. If the opening falls within the usual 'effective breadth' of concrete flange of any composite beams (typically span/8 each side of the beam centre line), the beam resistance should be checked assuming an appropriately reduced effective breadth of slab.

In the absence of any specific manufacturer's guidance on the provision of extra reinforcement, it may be assumed that an effective system of beam strips span the perimeter of the opening, as shown in Figure 4.14. The effective breadth of the beam strips should be taken as $d_o/2$, where d_o is the width of the opening in the direction transverse to the decking ribs.

The transverse beam strips are assumed to be simply supported and span a distance of $1.5d_o$. The longitudinal beam strips are designed to resist the load from the transverse beam strips, in addition to their own proportion of the loading. Extra reinforcement is provided within the beam strips to suit the applied loading. For the transverse strips only the concrete above the ribs is assumed to be effective. For the longitudinal strips this reinforcement often takes the form of bars placed in the troughs of the decking (see Figure 4.15). Additional transverse or diagonal bars may be used to improve load transfer around the opening. Reinforcement bars in these beam strips will need to extend at least an anchorage length beyond the centre line of the supporting beam.

Although a small opening (up to 300 mm wide) is unlikely to present a problem structurally and openings of this size do not normally require additional reinforcement, for the unusual (in the UK) case of a slab with extra bars in the troughs, their positioning relative to the opening needs to be considered. A 300 mm wide opening could very easily 'interrupt' a bar in a trough. Such interruption would need to be compensated for by placing additional longitudinal bars in the adjacent troughs using an adaptation of the beam-strip model normally used for medium-sized openings.

For medium-sized openings it is also worth remembering that some of the reinforcement in the beam-strips will be relatively susceptible to fire. Bars in troughs may have



SCI Tedds Modules

P300 - Composite Slab Openings Designer

SCI has launched the second in a range of specialist **SCI Tedds modules**. This, and other modules in the range, complement and extend the capability of the widely recognised Tekla Tedds software.

SCI is uniquely placed to produce such tools with over **30 years of experience** in providing information and global expertise to the construction sector.

This new Module: **SCI P300 — Composite Slab Openings Designer** is available now and implements the guidance and design procedures described in this SCI publication, "P300 - Composite Slabs and Beams using Steel Decking: Good Practice for Design and Construction", supplemented by SCI AD 447.

The module addresses the topic of **service openings** in composite slabs, providing a method to design the reinforcement required in 'reinforced concrete beam strips' around both individual and multiple service openings.

This makes the calculations easier and quicker to perform and has **credibility** with checking Authorities and warranty providers due to its SCI provenance.

The SCI is committed to helping the steel construction industry meet design, manufacture, installation and commercial objectives.

Click or Scan
for more info

+44 (0)1344 636501

teddsmodules@steel-sci.com

sufficient concrete cover to keep them cool, but bars (and fabric) in the slab between the ribs will become hot and loose considerable strength. Fire protection may be needed to ensure that the beam-strips retain their integrity in a fire.

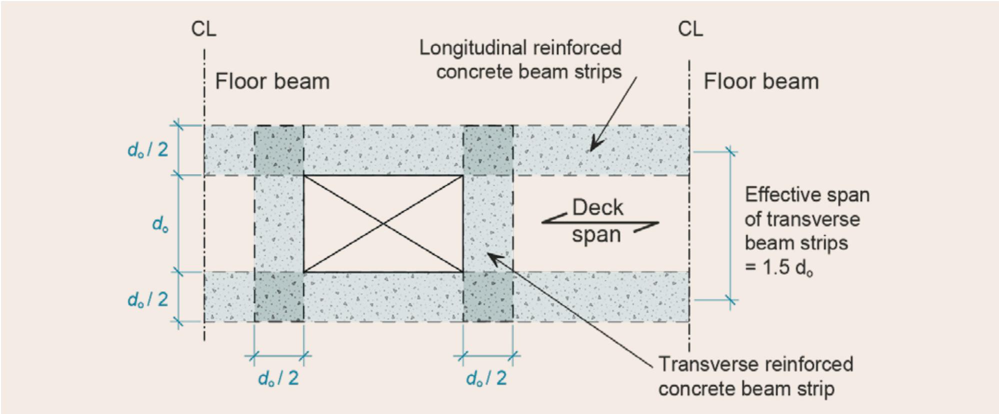


Figure 4.14 Load paths and beam strips around medium to large openings

In some situations, with multiple small or medium-sized openings it will not be possible to accommodate beam-strips (of the dimensions noted above) between adjacent openings. It may be necessary to treat a group of openings as one (larger) effective opening, and the reinforcement should be designed accordingly. *SCI P300 – Composite Slab Opening Designer Tedds Module* extends the guidance presented herein and in AD 447 to design the reinforcement around individual and multiple service openings. Based on the size and layout of the openings, the SCI Tedds module will determine the category of the service openings as well as the design scenario that needs to be considered. If it is not possible to accommodate beam strips between adjacent openings to carry the additional loads around the opening, the module considers the multiple openings acting as one large effective opening and beam strips are designed around the actual openings and the larger effective opening, picking up any local areas of otherwise unsupported slab. When the size or layout of the openings is such that trimming steel is needed below the slab, the module outputs the forces needed to design this steelwork.

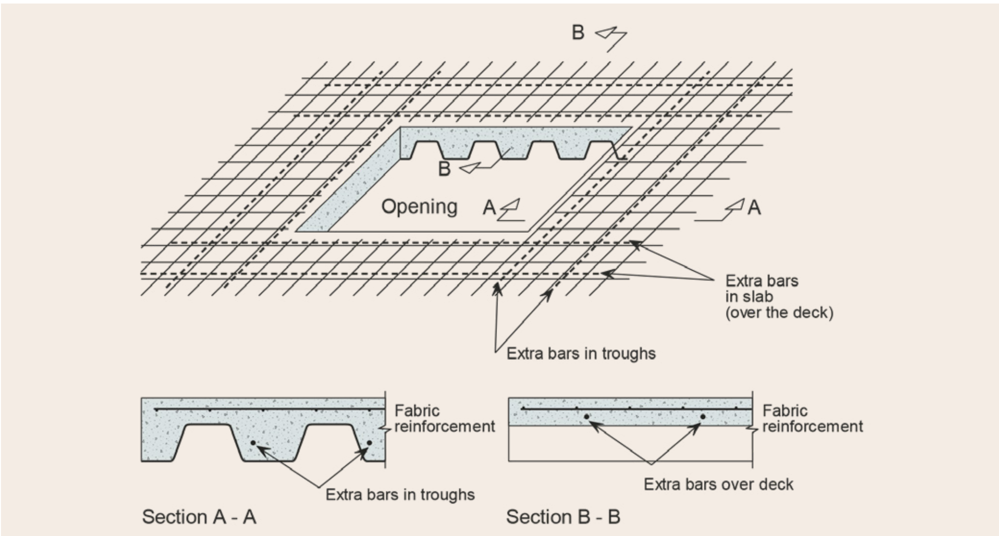


Figure 4.15 Typical reinforcement detailing around an opening

Slab edges

The edges of the floor are usually formed using 'edge trims' made from pressed strips of light gauge galvanized steel. Edge trim is delivered to site to the required depth and normally in standard 3.0 m long strips. The thickness of the steel used may vary with location, but is normally no more than 2 mm. The strips are cut to length on site to suit column centres. The trim is usually set out from the edge beam centre line (rather than the grid lines, which cannot be set out easily on site), as shown in Figure 4.16. The trim should be fixed in the same way as the decking. It should not be used as a tamping rail because it may be damaged.

Figure 4.16 Setting out of edge trim

courtesy of SMD

Restraint straps are always specified to tie back the upper part of the trim at 0.6 m to 1 m spacing, depending on the slab depth and overhang. Typical details are given in Figure 4.17, covering two distinct cases. Where the decking (with transverse ribs) runs over the edge beam and cantilevers out a short distance, the edge trim can be fastened in the manner suggested in Figure 4.17(a). The cantilever projection should be no more than 600 mm, depending on the depth of the slab and deck type used. This will also depend on where the hand rial is positioned.

The more difficult case is where the decking ribs run parallel to the edge beam, and the finished slab is required to project a short distance, so making the longitudinal edge of the sheet unsupported Figure 4.17(b). When the slab projection is more than approximately 200 mm (depending on the specific details), the edge trim should span between stub beams attached to the edge beam, as shown in Figure 4.17(c). These stub beams are usually less than 3 m apart and should be designed and specified by the PSD as part of the steelwork package. If stub beams are not provided in this case, then additional support to the edge of the decking, such as by propping from the floor beneath, may be required and this information must be highlighted in the information passed to the contractor. Non-standard edge trims (for example those used to support cladding, or those forming a curved edge) will require more accurate setting out procedures than standard trim.

Trims fixed to a curved edge are normally formed by cutting and bending the standard lengths to form a continuous faceted 'curve' with, typically, 300 mm straight sides.

Further information on how edge trims should be attached and supported is given in Reference 3.

Achievable tolerances for the position of the top of standard edge trims relative to the steelwork (after concreting) are ±10 mm horizontally, and ±5 mm vertically. Tighter tolerances than these may need to be specified for edge trims that incorporate sockets for cladding supports. It will also be necessary to ensure that these trims do not deflect excessively during concreting. The PSD must specify requirements for any such 'non-standard' trims.

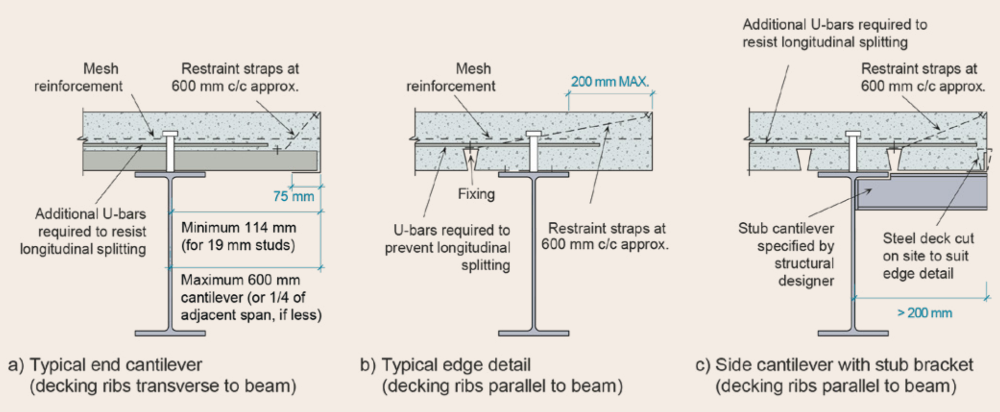


Figure 4.17 Typical edge details

4.2.7 Temporary supports

Decking is usually designed to be unpropped for spans up to 3.5 m for profiles up to 60 mm deep, and up to 4.5 m for profiles 80 mm deep. For longer spans temporary propping may be required, but this will depend on the depth of the concrete, the profile used and whether the decking is multiple or single spanning. The limitations of a particular decking should be checked with the manufacturer at an early stage so that any propping requirement can be planned. The PSD should check whether beam deflections during construction and the method of levelling of the slab would lead to significant additional concrete loads (from concrete ponding) that have not been allowed for in the design of the structure and decking (see Section 4.1.3). Propping may be necessary to minimise ponding.

In general, traditional 'shallow' decking does not require a propping system to be in place, braced and levelled, before placing the decking. However, there may be instances where this is required due to the product type and length of span. Deep decking will generally require such propping for spans greater than 7.5 m. Propping will reduce the deflections from the self-weight of the decking, which would otherwise be difficult to remove if props were installed after laying the decking. It will also minimise the risk of overloading the decking under loads during construction, e.g., from operatives and storage. Despite these benefits the implications of using a design which requires propping should be considered at an early stage, because it can have sequence and programming implications, and can preclude the use of safety netting.

When temporary propping is positioned after the decking is placed, it is particularly important to check that the props are set in accordance with the PSD's requirements, and that any increased weight due to concrete ponding is allowed for in the design of the propping system.

There may be small areas in a building where propping is necessary, even when the main areas of the floor remain unpropped. These propped areas may include bays that are in-filled after the removal of climbing cranes or lift shafts and have non-standard span lengths. The decking layout drawings should show the extent and lines of temporary supports.

Normally, props are placed at either mid-span (one line of props) or at third points (two lines of props) within a span. Isolated props should not be used, and all props should be suitably braced (in the direction of the line of the props and perpendicular to this) to prevent dislodgement during construction operations.

Props normally consist of lengths of timber and/or steel plates supported by adjustable length steel tubes ('Acrows'). The minimum bearing length of the timber and/or plates depends upon the thickness of the slab, the span length and the decking rib geometry. Bearing lengths are typically in the 75 to 100 mm range. The timber bearer should be continuous and should extend the full width of the bay. The decking sheets should never be interrupted (cut) at the location of a temporary support, and the decking should not be fastened to the temporary supports. It is good practice to carry out a final check of the propping system before pouring the concrete.

A typical temporary support is shown in Figure 4.18. Props of this nature are normally placed about 1 m apart, according to the PSD's requirements.

Figure 4.18 Temporary support using 'Acrow' props

courtesy of SMD

Props may be supported off the floor directly beneath the floor being concreted, but the designer should check that the design capacity of the lower floor is not exceeded (the supporting floor should have achieved its design strength before props are installed). If the lower floor does not have sufficient resistance, further 'back' propping will be needed (i.e., using props to support the floor that is supporting the props).

Props should never be placed directly on the decking alone as this could result in localised buckling of the deck. Further guidance on back propping can be found in Reference 52.

Props should not be removed until the floor has reached 75% of its design strength. This is normally achieved in 7 to 8 days, but the PSD should be consulted specifically before removal, unless general guidance has already been given. Where crack control is essential, props should not be removed until the floor has achieved its specified strength.

Other temporary support details may be needed for special situations, such as end supports in refurbishment projects or where there are concrete encased beams. Particular care should be taken with non-standard details to ensure that the sequencing of the construction is practicable. A typical detail for the temporary support of the decking at an encased beam is shown in Figure 4.19. In this case, the decking is supported initially off the steel beam, and a temporary prop is inserted under the decking close to the beam. This must be of sufficient width to avoid crushing of the decking during concreting (see Section 4.1.4). The decking is then cut back to allow access for the reinforcement and the shuttering to be positioned, and the concrete to be poured. This detail is only suitable for re-entrant decking because the decking can interlock into the concrete by the ‘dovetails’, without needing support underneath in the permanent condition. The decking cannot contribute to the shear resistance of the finished slab. Use of this detail should be confirmed by the PSD and indicated on the drawings.

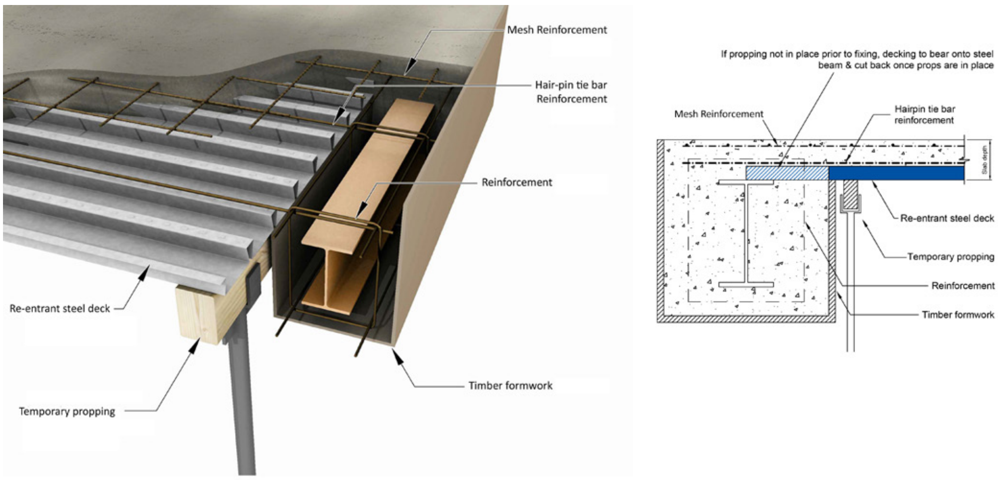


Figure 4.19 Special temporary support detail for re-entrant decking abutting a concrete encased beam

courtesy of SMD

4.2.8 Attachments

Hangers

The best way to eliminate the hazardous activity of post-drilling concrete to attach services is to use hangers, and designers are encouraged to specify them. Many decking profiles have re-entrant slots into which proprietary wedges can be inserted to receive threaded rods. The rods serve as hangers to the services, and they have a safe load-carrying capacity of, typically, 100 kg to 200 kg each. Some examples of these attachments are shown in Figure 4.20 for trapezoidal and re-entrant decking profiles.

Figure 4.20 Examples of wedge attachment fixings for ceilings and services

courtesy of SMD

Designers wishing to make use of such attachments should seek information, including safe load capacities, from the decking and/or hanger supplier. For services that are too heavy for the capability of the hangers, it may be necessary to use drilled expanding anchor fixings into the composite slab (taking care to ensure that the resistance of the fixing is appropriate for this type of use).

For detailed information on the interface between the services and the floor, refer to *Interfaces: Design of Steel framed buildings for service integration*[53].

Cladding supports

Brackets cast into the edge of the slab may be used to support the cladding. The brackets may form part of a proprietary edge trim. The trim may then need to be set out more accurately on site than a standard edge trim, but prior to this the support system should be co-ordinated early during the design process to allow for horizontal deviations in the edge of slab position of at least ±25 mm (more for high rise buildings). Such an allowance is necessary because not only may the allowable tolerances for the cladding be considerably more stringent than those for the frame, but also because the brackets may move during concreting. An example of decking and edge trim with an integral channel section that can be used with a brickwork support system is shown in Figure 4.21. The maximum overhang and propping details are shown in Figure 4.22, where the 125 mm minimum dimension is recommended so that all typical sizes of edge trim, anchors and local reinforcement can be accommodated. It is important that the edge trim can be effectively anchored into the concrete.

For buildings exceeding three storeys in height, it is important that the material/coating used for the brickwork supports is limited to austenitic stainless steel or aluminium bronze, phosphor bronze or copper in a non-aggressive environment, and limited to austenitic stainless steel in an aggressive environment[54].

As an alternative to cast-in brackets, drilled fixings may be used to achieve greater accuracy, although these require the use of power tools and may be more time-consuming. For further information on cladding attachments see Interfaces: *Curtain wall connections to steel frames*[55] .

Figure 4.21 Typical metal deck edge trim and integrated brickwork support system

courtesy of Ancon

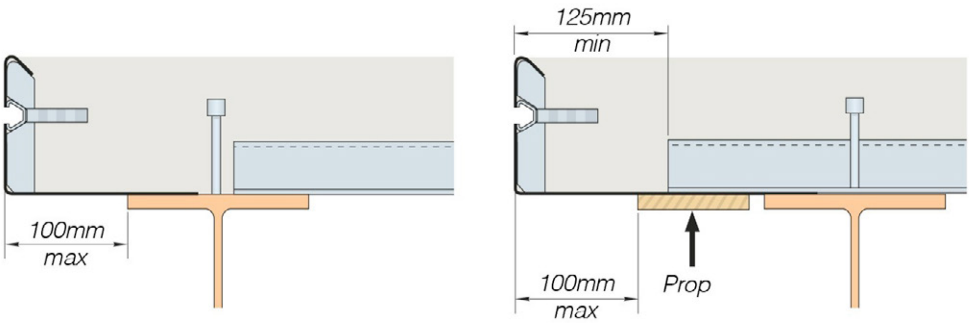


Figure 4.22 Maximum overhang dimensions and propping details

courtesy of Ancon

> Cladding fixing brackets should be provided with provision for adjustment, in recognition of different tolerance requirements for the slab edge and the cladding. This may avoid problems and additional cost on site.

4.3 Acoustic insulation

Building designers should consider two basic 'types' of sound transmission, airborne and impact. The acoustic insulation (attenuation) of both types of sound, and particularly airborne sound, is partly related to the mass of the element through which the sound is passing. It is also affected by the presence of any 'soft' layers, which increase the sound absorption.

High levels of sound insulation are typically required in:

- Multi-occupancy residential buildings
- Hotels
- Schools and other education-related buildings
- Hospitals and other health-related buildings.

For composite floors with shallow decking, the bare composite slab will normally provide a similar degree of acoustic insulation to a reinforced concrete slab with a thickness equal to the average thickness of the composite slab. However, the floor should not be considered as an element in isolation because the acoustic performance of the walls, and the junction details between the walls and floors, also need consideration. The junctions need to be detailed to minimise sound which may travel around the floor; known as flanking sound. Further guidance on this is provided in SCI Publication P372[56] . Approved Document E to The Building Regulations[57] requires residential buildings to undergo pre-completion acoustic testing to demonstrate compliance unless the walls, floors and their interfaces have been constructed in accordance with 'Robust Details'[58].

Enhancing the acoustic performance of the floor by adding mass is not very efficient and is not always practical. This is particularly true for impact sound. A much more effective means of enhancing sound attenuation is by the use of layers above and below the floor slab. A 'resilient layer' of material applied above the bare slab and beneath the walking surface is an effective way of reducing the impact sound transmission through the floor. The walking surface material, such as a screed or chipboard, forms a 'floating layer' above the resilient layer; the sound energy is absorbed by the resilient layer rather than being transmitted through the floor. A ceiling layer suspended below the floor slab will reduce airborne and impact sound transmission. In addition, the impact sound transmission can be reduced by suspending the ceiling on resilient bars, which reduces the sound transfer from the slab. Numerous proprietary acoustic floor and ceiling systems are available which incorporate these features.

As a guide to the acoustic performance of typical composite floor constructions, the attenuation that can be expected is given in Table 4.11 (these are indicative values based on test results). All the values given in the table are for normal weight concrete, which is recommended for dwellings and buildings where acoustic insulation is important. Lightweight concrete will generally give slightly less sound insulation than normal weight concrete.

Details of a composite floor with shallow decking, that complies with Robust Details, is shown in Figure 4.23.

Form of Floor Construction	Sound pressure level (dB)	
	Airborne Sound Attenuation $D_{nT,w} + C_{tr}$ (dB)	Impact Sound Level $L'_{nT,w}$ (dB)
Approved Document E requirement for purpose built dwellings	≥ 45	≤ 62
Shallow Deck Floors		
An 18 mm chipboard walking surface over 25 mm of dense mineral wool over a 175 mm slab on a 60 mm trapezoidal deck. A 12.5 mm plasterboard ceiling suspended from the slab on a metal framed grid.	55	43
An 18 mm MDF walking surface over a 10 mm dense fibre resilient layer over a 200 mm slab on a re-entrant deck. A 30 mm (2 x 15) plasterboard ceiling suspended from the slab on a metal framed grid.	56	34
A 70 mm screed on 5 mm foam resilient layer over a 150 mm slab on 50 mm re-entrant deck. A 12.5 mm plasterboard ceiling suspended from the slab on a metal framed grid with 85 mm of mineral wool in the ceiling void.	56	40
A 18 mm chipboard walking surface supported on softwood timber battens on a 25 mm acoustic quilt over a 150 mm slab on a re-entrant deck. A 25 mm (2 x 12.5) plasterboard ceiling suspended from the slab on a metal framed grid.	60	34

Table 4.11 Indicative acoustic data for composite floors

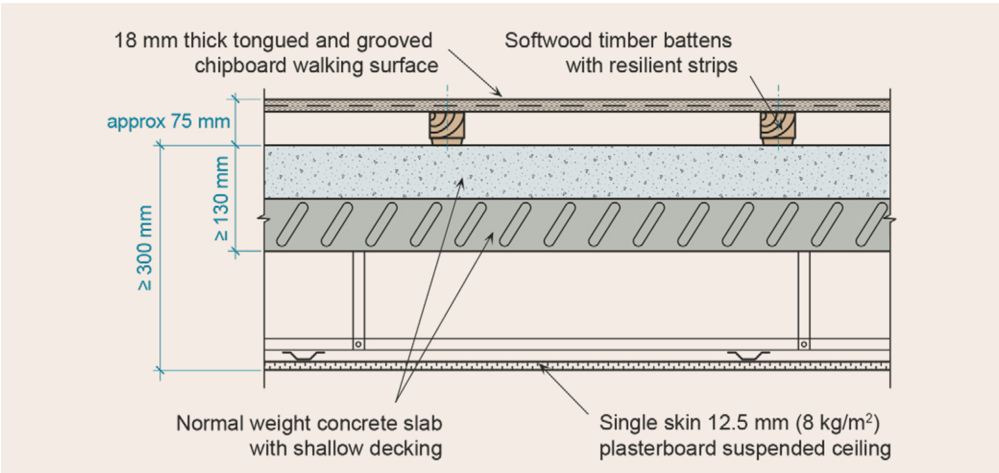


Figure 4.23 Typical shallow composite slab with a battened resilient floor system

4.4 Further reading

The references given below mainly relate to slab design. Detailed information on authors and publishers is given in Section 8, References.

Concrete Society Technical Report No. 63, 2007 - Guidance for the design of steel-fibre-reinforced concrete[40].
This publication summarises the range of applications for steel-fibre-reinforced concrete and includes practical aspects such as production and quality control.

Concrete Society Technical Report No. 65, 2007 - Guidance on the use of macro-synthetic fibre reinforced concrete[37].
This publication reviews the current range of applications for Macro-synthetic Fibre Reinforced Concrete and gives guidance on quality control.

Interfaces: Curtain wall systems for steel frames (P101)[55]
This outlines the main considerations in the design of a curtain wall attachment system, and reviews some of the systems available for steel framed buildings.

Composite design of steel framed buildings (P359)[59]
This design guide provides a comprehensive design methodology for composite slabs on I section steel beams, for design in accordance with the Eurocodes. The guidance is complemented by a full numerical worked example.

Acoustic detailing for steel construction (P372)[56]
This guide explains the principles of acoustics in a very readable way. Sound insulation values are given for many examples of floor and wall construction, and typical details are recommended which are compatible with the Robust Details Handbook[58].

Advisory Desk

The following ‘Advisory Desk’ items, published in New Steel Construction and available on SteelBiz, provide further information relevant to this Section:

AD 150, Vol 1, December 1993[60], *Composite floors - wheel loads for fork lift trucks*. This note gives a procedure for checking the behaviour of slabs under heavy point loading. [Further advice is given in BS EN 1994-1-1:2004, clause 9.4.3.]

AD 163, Vol 2, December 1994[24], *Provision for water vapour release in composite slabs*. This note explains the problems sometimes associated with water vapour release in composite roof slabs. It is suggested that perforated roof felt may be used, but the use of perforated decking is not recommended.

AD 247, Vol 9, March 2001[8], *Use of composite construction in an aggressive environment*. This note gives advice on improving the corrosion resistance of decking and beams for an aggressive environment. It recommends using coated steel or additional paint protection for decking and discusses the advantages and disadvantages of using shot-fired studs or pre-welded studs for beams.

AD 447, Vol 28, July/August 2020, *Openings in Composite slabs*. This note provides guidance on the provision of openings in composite slabs with clarification on the critical dimension of a small opening and dealing with situations with multiple openings.

AD 450, Vol 28, October 2020[61], *Resistance of composite slabs to concentrated loads*. This note considers EN 1994-1-1:2004 clause 9.4.3 of which has been the cause of much confusion; and in particular part 5, the limits of 7.5 kN and 5.0 kN/m^2.

AD 461, Vol 29, May 2021, *Anchorage of bars in the troughs of composite slabs*. This note provides guidance on the anchorage length of bars that are placed in troughs to provide moment resistances in a fire situation.

AD 477, Vol 30, February 2022[42], *Transverse bending of composite slabs subjected to point loads*. This note provides guidance on the design of composite slabs subjected to concentrated point loads. It proposes a more sophisticated approach, as a supplement rather than a replacement for the approach in AD 450.

AD 485, April 2022, *Continuity of additional bar reinforcement for fire in composite floors*. The purpose of this AD note is to clarify the function of the reinforcement and address the issue of continuity in the design of composite floors.

DESIGN OF COMPOSITE BEAMS

This Section provides information covering design principles and procedures, codified design rules, and guidance on good practice in design and detailing. It (along with Section 4) is aimed primarily at the PSD, and any SSD. Summary boxes are used to highlight particular issues of good practice, or things that the designer should be aware of.

Composite beams typically consist of downstand steel 'I' sections acting structurally with a concrete slab by means of shear connectors attached to the top flange of the steel section, as shown in Figure 5.1. The scope of this Section does not cover beams that are integrated within the slab depth, in so-called shallow floor construction. An effective width of slab is taken as acting as part of the composite section on either side of the centreline. Fabric reinforcement is normally placed in the slab; its main role, possibly supplemented by individual bars, is to act as transverse reinforcement in order to transfer the forces between the shear connectors and the slab. Alternatively, fibre reinforcement may be used to fulfil this role. Fabric or fibre reinforcement may also serve as a means of controlling crack widths. The beams are generally designed to be simply supported.

Traditionally, advice was that it was necessary to position fabric reinforcement below the head of the stud shear connectors to ensure that the design shear resistance and ductility of the connectors could be realized. However, on the basis of recent test evidence, it is now considered satisfactory (and better from a buildability point of view) to place the fabric above the studs and use it for both longitudinal shear resistance, and for integrity in the fire design situation. A proviso is that when assessing the shear connection, an appropriate reduction factor as given in PN001[62] and outlined in P405[63] is applied to the value of stud resistance given by BS EN 1994-1-1:2004. The situation regarding design to BS 5950-3 is simpler as there is no restriction on the position of the fabric[64] (the rules for stud resistance are based on tests with the most onerous positioning, and so can be used to cover all cases).

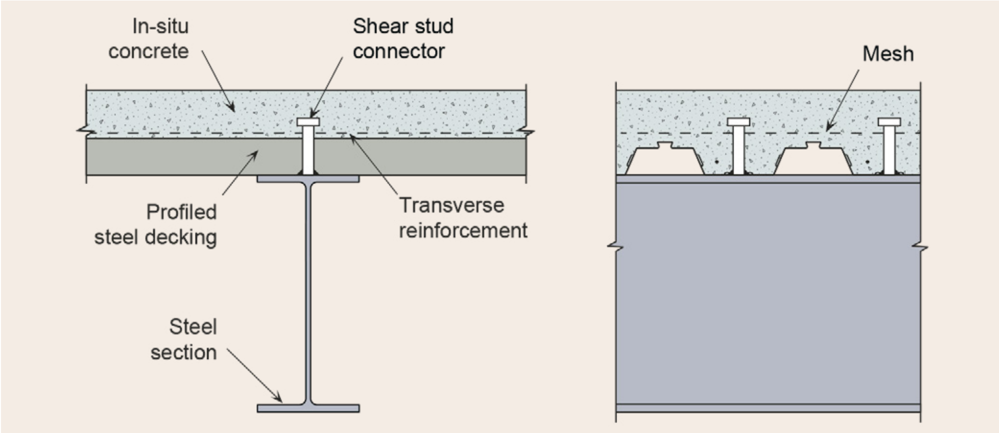


Figure 5.1 Typical cross section through a composite beam

The composite action developed between the steel beam and concrete slab significantly increases the load carrying capacity and stiffness of the beam, by factors of up to two and 3.5 respectively[65]. These benefits can result in significant savings in steel weight and/or structural floor depth.

It is often found that the size of the steel section is governed by serviceability considerations because composite beams tend to be used for long span applications (in excess of 9 m). This makes deflection and dynamic criteria more likely to be critical. Controlling deflections is particularly important where brittle ceiling finishes are specified, or for edge beams, where excessive deflections can damage the cladding.

Designers should note that where edge beams are designed as composite beams, care is required to ensure that the decking, edge trim, shear connector, slab edge and reinforcement details are practicable. Edge beams are sometimes designed non-compositely to avoid transverse reinforcement requirements, which may result in problems of reinforcement congestion.

5.1 Construction stage

The steel sections are normally designed to be unpropped during construction so must be sized to support the self-weight of the slab, and other construction loads, in their non-composite state. Design is generally in accordance with BS EN 1993-1-1:2005[66], although BS 5950-1 may also be used.

The weight of extra concrete from ponding of the slab should be allowed for in the design of the beams when the deflection of the decking under the wet weight of the concrete exceeds one tenth of the depth of the slab, in accordance with both BS EN 1994-1-1:2004 and BS 5950-4. Careful consideration should be given to the correct allowance for the weight of the concrete when ‘mass flood’ levelling techniques are adopted – see Section 6.2.1. As well as checking the resistance of the steel beams, this will involve an assessment of their stiffness. Beams that are not suitably stiff will deflect excessively during concrete placement which may impact on the ability to achieve the required surface tolerance, and the extra concrete should be allowed for in the design.

When designing to the Eurocodes, the construction load is defined in BS EN 1991-1-6:2005 and is taken as the same construction load as for designing the decking, as described in Section 4.1.2. However, in the U.K. it is recommended to ignore the 0.75 kN/m^2 patch load and simply design for a uniform load of 0.75 kN/m^2. This is recommended on the proviso that there is good site control, and (as can be seen below) still represents a 50% increase in the equivalent load used when designing to BS 5950. The self-weight of the wet concrete is treated as a variable load. See AD 346 for further information on design actions during concreting and the recommended expressions for combinations.

When designing to BS 5950, the construction load should be taken as an 'imposed load' of not less than 0.5 kN/m^2 applied uniformly over the supported area. The construction loading should be applied in addition to the self-weight of the concrete, reinforcement and decking. This non-composite check may dictate the final choice of section size if subsequent imposed loads are low.

To use a steel beam economically, the top (compression) flange needs to be restrained laterally. The restraint provided by the decking to the beams depends on the decking orientation and the fixings. The restraint provided by decking spanning in a direction parallel to a beam is normally assumed to be negligible, but decking spanning perpendicularly to a beam can provide restraint if it is adequately connected. In this latter case, continuous lateral restraint occurs when thru-deck welded shear connectors are provided (irrespective of other fixings), but when there are no shear connectors, restraint is limited by the resistance of the fixings. This will depend not only on the shear resistance of an individual fixing (refer to manufacturer's specification for shear resistance values), but also on their spacing along the beam. The PSD should ensure that the restraint assumed in the design is provided by the fixing arrangement; guidance on the force that must be resisted is given in the SCI publication *Lateral stability of steel beams and columns*[67] and BS EN 1993-1-1:2005 (or BS 5950-1).

Check that the steel beam size chosen is capable of supporting the wet weight of the concrete, and other construction loads, in its non-composite state.

Check that beam deflections during construction will not lead to significant additional concrete loads (due to ponding) that have not been allowed for in the design.

The decking will only provide lateral restraint to the beams during construction if the resistance of the fixings is adequate, and where the decking ribs run perpendicular to the beams.

5.2 Composite stage

5.2.1 Design for resistance

Composite beams, in buildings, are generally designed in accordance with BS EN 1994-1-1:2004 (or BS 5950-3). In both cases, the bending resistance of the section is normally evaluated using 'plastic' principles (provided the cross section will not be subject to local buckling). The calculated resistance is then independent of the method of loading, i.e., whether the beam is propped or unpropped during construction. The resistance must be adequate for the maximum total design moment at the ultimate limit state.

The plastic moment resistance is calculated using idealised rectangular stress blocks, as shown in Figure 5.2. In BS EN 1994-1-1:2004 it is generally assumed that stresses of f_{yd} and 0.85 f_{cd} can be achieved in the steel and concrete respectively, where f_{yd} (=f_y/γ_{M0}) is the design yield strength of the steel and $0.85f_{cd}$ is the bending compression resistance of the concrete. It is 0.85 times the design cylinder strength of the concrete f_{cd}, where $f_{cd}= f_{ck}/\gamma_c$. Equivalent strengths in BS 5950 are p_y for the design yield strength of the steel and $0.45f_{cu}$ for the design bending compression strength of the concrete (where f_{cu} is the cube strength of the concrete). In some cases, when higher strength steel is used, BS EN 1994-1-1:2004 introduces a beta factor that reduces the plastic moment resistance. Using this adjustment a simple plastic approach can still be used even though the concrete may have insufficient strain capacity to avoid crushing before the steel section fully yields.

The plastic neutral axis may fall within the depth of either the slab or steel section, depending on the relative areas of these two components.

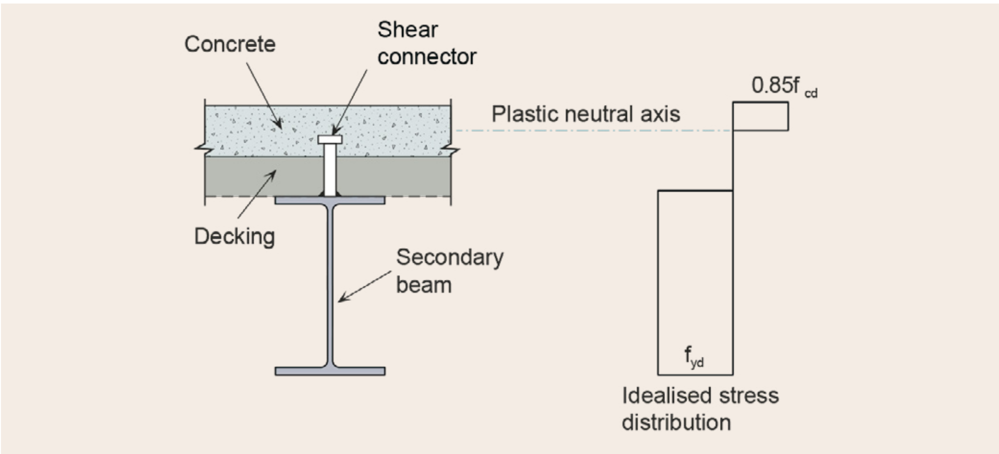


Figure 5.2 Plastic analysis of composite section (using BS EN 1994-1-1:2004 notation)

The area of concrete in compression is limited by its effective breadth. This breadth varies continuously along the length of a beam, as shown schematically in Figure 5.3. Its form depends on the type of loading, and the end conditions (simply supported or continuous). However, a simpler form may be assumed for design.

In BS EN 1994-1-1:2004, the effective breadth is defined as constant for the middle portion of the span and tapering towards each end, as shown in Figure 5.3. The distance between centres of pairs of shear connectors, b_0, is also included. For the serviceability limit state, a constant effective breadth can be assumed to act over the entire span, based on the mid-span value. In BS 5950-3, the effective breadth is a constant value for a simply supported beam with decking perpendicular to the beam. For both BS 5950-3 and BS EN 1994-1-1:2004 the maximum value of the effective breadth is span/8 on each side of the centre-line of the beam for both serviceability conditions and for the ultimate limit state. As well as considering this limit, the width assumed in design must not exceed the actual slab width available. This is particularly relevant for edge beams and beams adjacent to openings, where there may be only a narrow width of slab on one side.

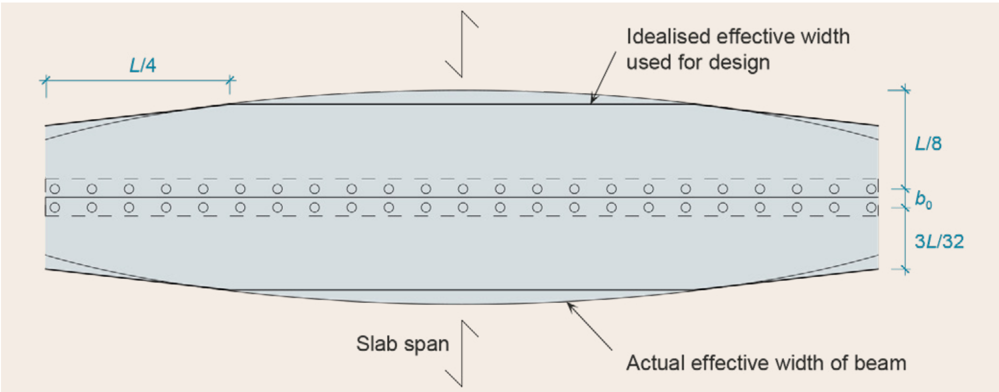


Figure 5.3 Effective width profile of a simply supported beam according to BS EN 1994-1-1:2004

The compressive area of the concrete also depends on the orientation of the decking. When the decking ribs run perpendicular to the beam, the concrete contained within the depth of the decking must be neglected (see Figure 5.4(a)). When the decking ribs run parallel to the beam, the total cross-sectional area of the concrete may be considered, provided that it lies above the neutral axis of the composite section (Figure 5.4(b)).

The stress blocks that can be generated in the steel and concrete, and used to calculate the moment resistance, may be limited by the amount of longitudinal shear force that can be transferred between the materials at their interface (which is a function of the number of shear connectors and their resistance). If this limit governs, it is called ‘partial interaction’ (see Section 5.3.3). It is worth noting that a lower bound exists for the amount of shear force that can be transferred, defined in terms of a minimum degree of shear connection. This limit is not related to the amount of force transfer that is needed to resist the applied loads, but rather a minimum level of stiffness that is required at the interface to limit slip at the beam ends to a value that can be accommodated by the shear connectors. Limiting values for degree of shear connection are given in BS EN 1994-1-1:2004 (and BS 5950-3), but a more comprehensive consideration of the subject allows much lower values to be given in SCI P405. The values in P405 pre-empt what is likely to appear, in a more simplified way, in Generation 2 EN 1994-1-1.

It is normally assumed in both BS EN 1994-1-1:2004 and BS 5950-3 that all of the vertical shear force applied to the beam is resisted by the steel section alone. Design checks should therefore be in accordance with BS EN 1993-1-1:2005 or BS 5950-3, which give guidance on the consideration of combined bending and shear. In addition, the width of the top flange of the steel section must be sufficient to ensure that the decking does not fail in bearing over the beam.

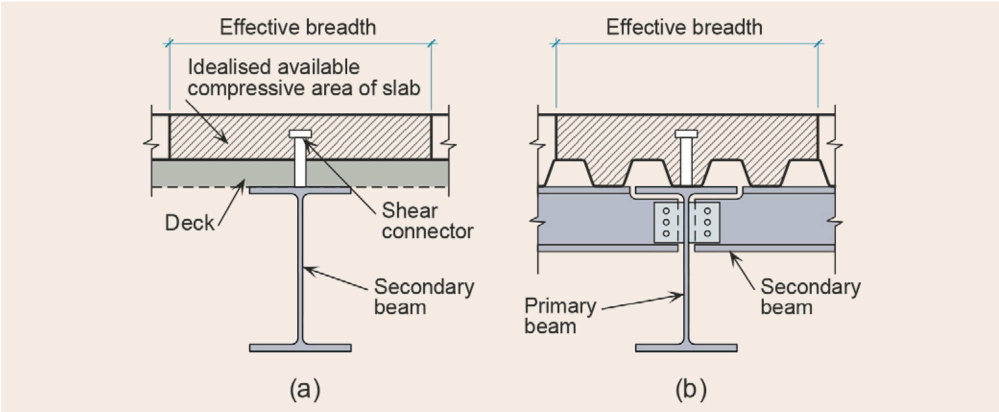


Figure 5.4 Composite beams incorporating composite slabs. (a) Deck perpendicular to secondary beam. (b) Deck parallel to primary beam

The concrete adjacent to the steel beam forms a structural flange, and the presence of openings in the slab will therefore influence the performance of the beam.

Avoid placing slab openings next to beams (within the effective flange width) wherever possible. If such openings cannot be avoided, their effect must be included in the beam design.

5.2.2 Design for serviceability

Deflections

Composite beams are generally shallower (for a given span and loading) than non-composite beams, and they are used commonly in long span applications. Consequently, deflections are often critical. BS EN 1990:2002[68] (and BS 5950-1) recommends that deflections should not affect the appearance, the comfort of users or the functioning of the structure.

In addition to the 'traditional' deflection check under imposed loads, it is also prudent to check deflections due to the following:

- **Total (in-service) loads** – the combined dead and imposed loads should be considered to ensure that floor curvatures will not be unacceptable (see comments below on deflection limits). This is particularly important for long spans if there is limited depth available in the zone for under-floor services
- **Construction loads** – although not a serviceability deflection limit, it is necessary to check that excessive concrete ponding will not apply significant extra loading to the structure.

For edge beams supporting cladding, it is important that the deflections are checked under cladding and imposed floor loads to ensure that the deflection of the beams does not compromise the performance of the cladding.

'Uncracked' elastic section properties should be used to calculate the deflection of simply supported beams (the total area of concrete within the effective flange width is considered, even that part which in reality will be cracked in tension). A transformed section is used; the effective width of the slab is reduced using a modular ratio equal to the elastic modulus of the steel divided by that of the concrete. The effect of creep of the concrete is taken into account by choosing a modular ratio between one based on short term concrete properties and one based on long term properties, according to the mix of long and short-term loading.

When designing to BS EN 1994-1-1:2004, the increased flexibility of the composite beam caused by greater slippage between the concrete slab and the steel section when using partial shear connection may be ignored if the degree of shear connection is not below 50%. However, BS 5950-3 requires an additional deflection to be included using a modification factor. When SCI publication P405 is used (as NCCI to complement Eurocode design) to justify lower minimum degrees of connection, its guidance also requires that deflections are increased to allow for the greater interface flexibility that this may result in. P405 uses the same approach as BS 5950-3. Generation 2 EN 1994-1-1 will include lower limits for minimum degree of shear connection than the current code (not dissimilar to the values in P405), and will make a similar allowance for loss of beam stiffness.

For the calculation of deflections where props are used, all the loads are applied to the composite section.

Deflection limits for beams subject to imposed load are recommended in the National Annex to BS EN 1993-1-1:2005 (or BS 5950-1). As a more comprehensive guide, the deflection limits given in Table 5.1 may be considered in design.

Table 5.1 Suggested deflection limits for composite beams

Beam Type	Load Case	Limit	Absolute Limit (mm)
Internal beams	Imposed load	Span/360	To suit finishes
	Total load	Span/200	To suit finishes
	Dead load at construction stage	-	25‡
Edge beams supporting both floor and cladding	Imposed load	Span/500	To suit cladding
	Imposed load + cladding	Span/360	To suit cladding
	Total load	Span/250	To suit cladding
Edge beams supporting cladding only	Cladding weight	Span/500	To suit cladding

Notes:
‡ Although not a serviceability criterion, this is to limit the additional load due to ponding of the concrete, consequent on beam

Where dead load deflections would be excessive, pre-cambering may be appropriate (this is normally only adopted for beams longer than 10 m). However, the pre-camber required may be difficult to determine accurately; for example, the stiffening effect of the end connections may be significant, so some pre-camber may remain after casting, and the depth of the slab may not be as intended at the critical point of mid-span. Therefore, a general rule of thumb is to design any pre-cambering to eliminate no more than two thirds of the dead load deflection. In some situations, large or varying amounts of pre-camber between different beams within a sheet length, may even hinder the laying of decking.

Further information on methods of calculating deflections are given in *Composite design of steel framed buildings* (P359)[59]. For the assessment of beams with web openings, additional guidance may be found in *Design of beams with large web openings for services*[69].

> The deflection limits used in design should be chosen to suit the building details.
>
> Pre-cambering can be used to limit beam deflections under dead load. Assessing the amount of pre-camber needed may prove difficult, and calculations should take into account the likely stiffening effect of the end connections.

Irreversible deformation

In BS EN 1994-1-1:2004 there is no specific requirement to limit stresses at the serviceability limit state (see clause 7.2.2). In BS 5950-3, the stresses in simply supported composite beams at the serviceability limit state, calculated using elastic principles, are limited to p_y in the bottom fibres of the steel section, and $0.5f_{cu}$ in the concrete slab. Full shear connection, with negligible slip, may be assumed when calculating these stresses. Any part of the concrete in tension should be neglected when calculating stresses (‘cracked’ section properties should be assumed – unlike the procedure for calculating deflections, when uncracked properties may be assumed). A similar limitation could be applied when designing to BS EN 1994-1-1:2004, whereby the steel stress would be limited to f_y and the concrete stress to $0.63f_{ck}$.

In unpropped construction, the stresses should be calculated first for the non-composite section subjected to the loading at the construction stage, and then those for the composite section should be added. In propped construction, the stresses due to the construction loading are often ignored.

Dynamic sensitivity

Traditionally, the parameter used to assess the dynamic sensitivity of a floor is its natural frequency. This allows a simple assessment of what is, in reality, very complex behaviour. A frequency of 4 Hz is a commonly accepted lower limit for the natural frequency of an individual composite floor beam, as this will generally mean that the frequency of the entire floor system is greater than 3 Hz, and therefore ensure that excitation activities do not occur at a frequency that coincides with that of the floor. A higher frequency limit may be appropriate for applications such as dance floors and gymnasia.

The natural frequency of a floor beam may be determined from the approximate formula $f = 18/\sqrt{\delta}$, where δ is the static deflection (in millimetres) resulting from the application of the self-weight of the floor, plus that of the ceiling and finishes, plus 10% of the imposed loading applied to the composite beam. Partitions tend to increase the damping and stiffness of the structure and are not included in the loading.

Floors are likely to be more 'lively' in situations where there is a grid of primary beams and secondary beams. In these cases, the cumulative deflection of the slab, secondary beams and primary beams (i.e., the total deflection in the middle of the slab) should be assessed, and a combined floor frequency calculated. A method for determining the combined frequency is set out in *Design of floors for vibration: A new approach*[71]. This publication also includes methods for determining the likely accelerations of a floor when subjected to vibration, in terms of Response Factors, and it is recommended that this more detailed analysis is used to assess the dynamic sensitivity of a floor if response is likely to be critical.

Long span applications, for which composite beams are often used on account of their excellent resistance and stiffness characteristics, often have a relatively low natural frequency. However, they also tend to have a high effective mass. The consequence of this is that the inertia of the floor relative to the impacting dynamic loads is large, so that floor accelerations (which are what dictate occupant comfort) remain acceptably low. This means that even if the natural frequency limit is not satisfied, a full calculation of the floor response may show it to be satisfactory. Further information may be found in Reference 70.

5.2.3 Design for fire resistance

Typically, composite beams are designed to achieve the required fire resistance by applying fire protection materials. Three methods of protection are commonly used: boards, sprays and intumescent coatings. Fire protection adds to the cost of the structural frame and has implications on the construction programme, as another trade has to be accommodated in the fabrication or construction programme. There are alternative methods of design available which limit the extent of fire protection required on composite floor plates, as described in SCI publication, P288[71].

It is also possible to take the fire protection operations for beams off-site. Using off-site intumescent coatings, unlike traditional forms of protection (which are applied on site), means the operation is not on the critical path and is not affected by the weather. Although more careful handling and storage of the sections is required with offsite coatings, any slight damage can be touched up easily. Thru-deck stud welding on site may have an effect on the intumescent coating but does not prohibit the use of offsite protection of composite beams. Part of the upper surface of the top flange must remain unprotected, and the minimum flange thickness required by BS EN 1994 may not be sufficient to prevent blistering of the intumescent on the underside (note that some blistering may be acceptable). Further information is available in *Code of Practice for Off-site Applied Thin Film Intumescent Coatings*[72].

For composite beams that are to be fire protected, a 'critical temperature' needs to be established in order to enable the required thickness of fire protection to be determined. It is worth noting that it may be cost effective to increase the size of the steel profile in order to reduce the thickness of protection that is needed (although this may have a negative impact on embodied energy). Methods of determining the failure temperatures are provided in BS EN 1994-1-2:2005 and BS 5950-8. The terminology used to describe these methods is different in each Standard but they both provide a calculation model for determining the relationship between beam failure temperature and the load applied in fire conditions. The thermal properties of proprietary fire protection systems are not readily available in the public domain. Table 5.2 provides an initial estimate for the critical temperature for composite beams subject to bending. More comprehensive information is given in BS EN 1994-1-2:2005 and in BS 5950-8. The load level at the fire limit state η_{fi} should be calculated as follows:

$$\eta_{\mathrm{fi}} = \frac{E_{\mathrm{fi,d,t}}}{R_{\mathrm{d}}}$$

where $E_{fi,d,t}$ is the design value of the effects of actions at the fire limit state and R_d is the design resistance at normal temperature.

For beams designed to the Eurocodes, it is recommended that the resistance of the composite beam is verified by calculation of the moment resistance $M_{fi,Rd}$ using the procedure given in section 4.3.4.2 of BS EN 1994-1-2:2005, which is described in more detail in *Fire resistance design of steel framed buildings* (P375). The temperature of the steel for this calculation should be based on the value of critical temperature obtained from Table 5.2, where values have been calculated on the basis of a uniform temperature for the steel section. There is no limitation on the depth of the section that may be designed using the temperatures from Table 5.2, and the temperatures are appropriate for all values of shear connection. For beams designed to BS 5950-8, the use of the limiting temperature method is recommended.

Beams with web openings present a particular problem as far as the specification of fire protection is concerned, as the critical mode of failure may be related to buckling of the web posts between the openings rather than global bending at the point of maximum applied moment. The performance of the fire protection material has also been found to influence the critical temperature. The critical temperatures given in Table 5.2 should not be applied to beams with web openings. Further guidance on fire protection of cellular beams is given in RT1356[73].

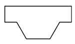
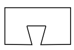
Table 5.2 Critical temperatures for composite downstand beams

Description of member	Critical Temperature (ºC) for a load level η_{fi} of:						
	0.7	0.6	0.5	0.4	0.3	0.2	0.1
Composite members in bending supporting concrete slabs or composite slabs	535	567	600	641	680	738	838

When the ribs of the profiled steel decking run across the steel beams, voids are created between the decking and the top flange of the steel (see Figure 5.4). Although additional heat enters into the steel beam via these voids, BS EN 1994-1-2:2005 recommends that the voids are ignored if at least 85% of the surface of the top flange is in contact with the slab. This means that for re-entrant decks the voids do not need to be filled. However, for trapezoidal decks the voids must be filled - or the effect of the voids on the beam temperature must be considered when determining the critical temperature of the section. This is beyond the scope of the simple thermal model given in BS EN 1994-1-2:2005. Therefore, if the voids are to be unfilled, the temperature of the beam must be determined from tests or advanced analysis. Using decking with preformed ('crushed') ends may alleviate this problem.

Fire tests in accordance with BS 476-21[74] have shown the effects of unfilled voids on structural performance[75]. As UK Building Regulations still recognise the BS 476 test methods, this guidance may still be used in the UK - although this situation may change in the future. Guidance is given in Table 5.3 to identify when special measures must be taken because of these voids, and what they should be. In some cases, it may be necessary to increase the thickness of fire protection to compensate for the adverse effect of the voids, or they may be filled.

Table 5.3 Recommendations for fire protection of voids between profiled steel decking and steel beams in composite floor construction

Trapezoidal deck 				
Beam Type	**Fire Protection on Beam**	**Fire Resistance (minutes)**		
		Up to 60	**90**	**Over 90**
Composite	Insulating sprays and boards (assessed at 550° C)	No increase in thickness	Increase thickness by 10 % or assess thickness using A/V increased by 15%*	Fill voids
Composite	Intumescent coatings (assessed at 620° C)	Increase thickness by 20% or assess thickness using A/V increased by 30%*	Increase thickness by 30% or assess thickness using A/V increased by 50%	Fill voids
Non-composite	All types	Fill voids		
Re-entrant deck 				
Beam Type	**Fire Protection on Beam**	**Fire Resistance (minutes)**		
		Up to 60	**90**	**Over 90**
Any	All Types	Voids may be left unfilled for all periods of fire resistance		

* The least onerous option may be used (A/V=heated surface area per unit volume of the steel section)

It should nevertheless be remembered that, for all beams forming part of a fire compartment wall, the voids should always be filled to avoid affecting the integrity of the compartment wall.

Where the voids have to be filled, it is not necessary to use the same material as that used to protect the beam; any non-combustible material will suffice.

For beams with decking orientated parallel to them, the edges of the top flange must be protected; when board protection is used, the boards should be taken past the edge of the flange to abut the underside of the decking.

> It is not always necessary to fire protect the voids between the steel flange and decking. Specifying such protection unnecessarily will lead to increased costs.

5.3 Shear connection

The longitudinal shear connection between the steel section and the concrete is provided by shear connectors, which normally take the form of studs welded to the top of the steel section. All connectors should be capable of resisting uplift forces caused by the tendency for the slab to separate from the beam as it bends. In the case of shear studs this is achieved by the head of the stud.

Although shear connectors fix the decking to the beam and can ensure composite action between the slab and the beam, they may not be needed simply to achieve this. They may also be used to tie the slab to edge beams when the floor acts as a diaphragm, and to address robustness requirements.

5.3.1 Connectors

The most common type of shear connector used in composite beams for buildings has a 19 mm diameter with a nominal length after welding through decking of either 95 mm or 120 mm (the same studs would be 5 mm longer if welded directly to a beam flange). For thru-deck welding (see below), this is the only stud diameter that can be used practically, because it is the only one for which suitable ferrules are readily available. Although other heights are available, they are not so easy to obtain.

There are a number of other forms of shear connector available, such as angles welded to the top flange. Rules have also been developed in recent years for so-called composite dowels, although to-date they have only found application in bridges (at the time of writing a CEN Technical Specification to supplement BS EN 1994-1-1 is being developed and will cover their use in buildings). However, most other forms of shear connection lack a practical application in composite beams, with the exception of shot-fired connectors. These should be considered for smaller projects, those where beams need to be galvanized or top flanges painted for reasons of durability, or indeed any project where the provision of power for stud welding is a problem. They may be particularly appropriate for refurbishment projects, where there is either limited access, increased risk of fire or no earthing facility. The most common shot-fired shear connector is that produced by Hilti, which is available in heights up to 140 mm. It should be noted that a shot-fired connector typically has less resistance than a welded stud. Design guidance and design values of the shear resistance should be obtained from the supplier.

> It should be noted that although a shot-fired connector has less resistance than a welded stud, they may be more suitable for certain applications. Design guidance should be obtained from the supplier.

Resistance

Design resistances of shear studs are given in BS EN 1994-1-1:2004 and BS 5950-3, based on standardised push-out tests on samples with 'standard' fabric reinforcement. However, tests have also been carried out on specimens with fibre reinforcement, and these results show they perform at least as well as those with fabric reinforcement. The design resistance is a function of the:

- Shape of the decking profile
- Decking orientation
- Size, strength and number of connectors per decking trough
- Concrete properties
- The sheet thickness (according to BS EN 1994-1-1:2004).

In BS EN 1994-1-1:2004, the resistance of a stud in a solid slab is calculated directly from formulae which include terms for the steel strength and concrete strength and modulus. The concrete modulus can be found in BS EN 1992-1-1:2004, where the value for normal concrete (E_{cm}) is given in Table 3.1, and the value for lightweight aggregate concrete (E_{lcm}) is given in Table 11.3.1. The presence of shallow cracking above a beam does not necessitate a reduction in the design resistance of the shear connectors, because of the presence of the transverse reinforcement or steel fabric reinforcement. When decking is present the solid slab resistance is reduced as a function of the geometry of the shear connection and decking profile, and the decking orientation. Although not clear in BS EN 1994-1-1:2004, it is recommended that the stud resistance values should be calculated using the nominal length after welding value[76].

In BS 5950-3, the design resistance of a stud in a solid slab in normal weight concrete is given in Table 5, according to concrete strength and stud dimensions. These values are reduced by 10% when lightweight concrete is used.

The reason why the efficiency of the shear connectors may be reduced when the decking is orientated with the ribs transverse to the beam is because the force transferred through the shear connector into the slab relies on a small, localised area of concrete immediately in front of the stud as can be seen in Figure 5.5. For this orientation of the decking, this area of concrete is limited in size by the presence of the profile, as shown in Figure 5.6. Limiting the volume of concrete ahead of the stud changes the concrete failure mode – in a solid slab the concrete is most highly stressed around the base of the stud, whereas with transverse decking a concrete surface passing over the stud is associated with failure. Reduction formulae are given in BS EN 1994-1-1:2004 and BS 5950-3 to allow for this by considering the relative geometry of the stud and the decking rib.

Figure 5.5 Local failure of the concrete surrounding a shear stud in trapezoidal decking

courtesy of University of Luxembourg

As well as the shape and thickness of the decking, the position of the stud in the trough is important; tests have shown that the integrity of this local area of concrete can break down if the stud is positioned close to the decking. The formulae given in BS EN 1994-1-1:2004 and BS 5950-3 assume that studs are located centrally in the troughs or are alternated between the 'favourable' and 'unfavourable' side of the trough. Recommended practice is to place the studs in the 'favourable' side (see Figure 5.6). This means that, for single studs on simply supported beams with symmetric loading, the position of the stud in a trough with a central stiffener must change at mid-span.

The number of studs placed transversely (across the width of the beam) in each trough also affects their resistance. A reduction factor should be applied to the design resistance when two studs are present. Note that the design resistances given in BS EN 1994-1-1:2004 explicitly do not cover more than two studs per trough.

It is anticipated that the Generation 2 EN 1994-1-1 will introduce revised values for stud resistances, alongside revised requirements for the length of embedment of the stud in the solid concrete above the deck. However, it will also include the possibility to demonstrate performance by specific testing, so it is not expected to have an impact on current UK practice. It may also include consideration of different grades of connector ductility, as defined by a load-slip curve.

For further information on resistance, reference should be made to the *Designer's guide to BS EN 1994-1-1, Composite Design of Steel Framed Buildings* (P359) and the *Commentary on BS 5950: Part 3: Section 3.1 Composite beams*.

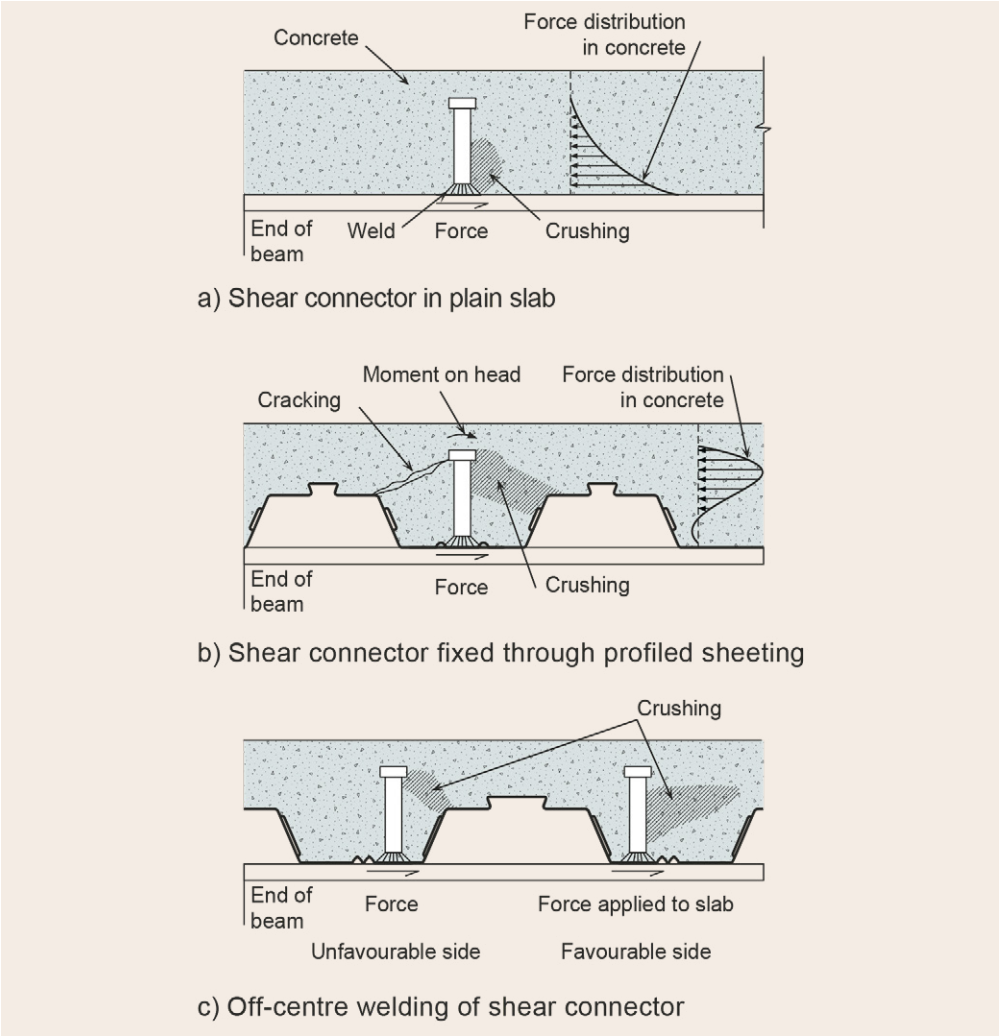


Figure 5.6 Shear connector forces in composite slabs

Attachment of studs

'Thru-deck' welding is generally used, particularly in the UK, to attach the shear studs to the steel beams. This process welds the stud, decking and steel section effectively together in a single operation. A typical run of thru-deck welded shear studs is shown in Figure 5.8. The PSD should recognise the following practical limitations before specifying thru-deck welding:

- The galvanized steel decking should not exceed 1.25 mm thick, and the total galvanizing thickness should normally not exceed 0.05 mm
- The thickness of the top flange of the steel section must not be less than 0.4 times the stud diameter (i.e., 7.6 mm for a 19 mm stud) to prevent localised bending of the flange at ultimate loading
- Small numbers of studs are uneconomic because of the amount and expense of the equipment needed
- A clear height above a beam of at least 600 mm is recommended to carry out the process of stud welding, i.e., to give room for the operative and equipment. (A typical example of where a problem can occur is when there is a change in the

floor level, as shown in Figure 5.7.) In these situations, it may also prove difficult to fix the slab edge trim. For shot-fired fixings, the clear height could be reduced

- The need to keep the top of the beam flange free of paint is generally not a problem (in an internal environment having an exposed, unprotected top of flange in the 'voids' is acceptable). Thru-deck welding may however blister any paint applied, and required, on the underside of the flange. Remedial measures may be required for aesthetic, if not corrosion protection, reasons. An intumescent coating on the underside of the flange might also be damaged but would not normally need remedial work other than for aesthetic reasons. Whilst it is not typically necessary to return paint to the top flange, where this is a specific project requirement the paint should extend no more than 10 mm from the edge of the flange. Paint thickness should be considered and intumescent coatings are not suitable for this region
- A minimum flange width is needed to provide sufficient bearing for the decking on both sides, end distance from the stud to the sheet when anchorage from the stud is required (for decking design and to enable the decking to be included as transverse reinforcement and shear reinforcement), and transverse distance between studs. Consequently, when the decking is perpendicular to the beam, flange widths less than 125 mm are not recommended (see below for advice when pre-welded studs are used).

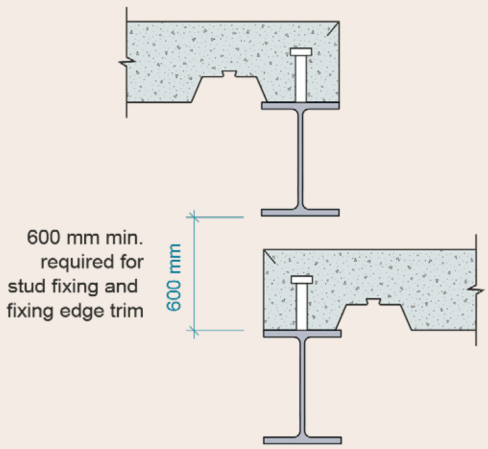


Figure 5.7 Minimum height clearance for stud welding and fixing edge trim

Thru-deck welding of shear studs is not recommended for galvanised decking with more than 350 g/m^2 of zinc coating because of the risk of substandard weld quality (typically metal decking has a total of 275 g/m^2). For galvanized steel beams, the same Thru-deck welding process cannot be performed because the quantity of zinc will be considerably greater than the quantity present on steel decking, further information can be found in AD 365.

Thru-deck welding is significantly more economical than the alternative of pre-welding the studs to the steel beams in the factory, although as noted above it is not always an option.

Problems associated with using pre-welded studs include:

- Deck installation can be more complicated and therefore slower
- Decking will normally need to be laid in single spans between the lines of studs, which requires beams with a sufficient flange width (≥140 mm, although this could be reduced to 133 mm provided deviations are tightly controlled on site) to provide the minimum safe bearing for the decking on each side of the beam, and the decking is less efficient as a single-spanning member
- Alignment of the troughs in decking perpendicular to the beam with pre-welded studs can be difficult. The flange width should be specified so that, should they not align, at least 50 mm concrete encasement is provided beyond the transverse spacing of the studs to the edges of the decking. The stud resistance may be reduced when studs do not align with the troughs, and it should be calculated using the reduction factor for ribs parallel to the beam using the width of encasement provided, rather than the reduction factor for ribs perpendicular to beam.

Holes may be cut in the decking to avoid these problems, but this leads to other complications trying to align the studs and holes. Decking placement will become more complicated because of the need to slot the studs through the holes, and so this method is not recommended.

Figure 5.8
A run of stud shear connectors

courtesy of Kingspan

Detailing rules

The following detailing rules apply to the positioning of stud shear connectors, and are illustrated in Figure 5.10:

- BS EN 1994-1-1:2004 requires that nominal height (before welding) of the shear studs should be at least 2d (where *d* is the stud diameter) above the top of the decking. The corresponding requirement in BS 5950-3 is 35 mm. (Note that the 'top of the decking' refers to the height of the shoulder, i.e., excluding any small stiffening ribs in the crest of the decking.) Studs that are longer than is necessary to meet these requirements will not have a greater resistance according to these codified rules, although work undertaken in the development of Generation 2 EN 1994 suggests greater embedment can result in higher resistance. A stud that is for example 105 mm long when manufactured would typically have length after welding of 100 mm when welded directly to a beam flange, or 95 mm when welded through decking. It would generally be described as a nominal 100 mm stud. A nominal 100 mm stud, of 19 mm diameter, may therefore be used with 60 mm decking. Further guidance on the shear stud length can be found in AD 380 and AD 424[77]

- To avoid damaging the decking, the studs should be located along pre-determined lines marked on it by the deck installers
- The distance between the edge of the shear connector and the edge of the steel flange should not be less than 20 mm
- The minimum longitudinal spacing of the studs should be 5*d*. BS EN 1994-1-1:2004 states that the maximum spacing should not exceed the lesser of 800 mm and six times the slab depth which P405 recognises as being too lenient (it requires studs to be placed in every trough with transverse decking). The limit given in BS 5950-3 is 450 mm, and this latter value is recommended in the light of recent test evidence[78]. It should be noted that studs are often required at larger spacings than these on non-composite beams for other purposes, such as restraint of the beam
- The transverse stud spacing should not be less than 2.5*d* in solid slabs and 4*d* in other cases
- Studs should normally be placed uniformly along the length of the beam. Any additional studs noted on the drawing that cannot be placed in equal numbers in all the troughs should be positioned symmetrically about the mid-span of the beam, working from the supports inward (assuming uniformly distributed loading)
- When the decking has a central stiffener in the trough (which makes it impossible to attach the stud centrally), adequate onsite quality control should be in place such that the studs should be attached on the favourable side of the trough (Figure 5.10 (c)). For symmetrically loaded beams, this will involve a changeover of position of the stud at mid-span. ('Favourable' or beneficial is defined as the side of the trough closer to the nearer support so that the zone of concrete in compression in front of the stud is larger than that behind the stud as shown in Figure 5.10 (a))
- It is assumed in BS EN1994-1-1 that a 'staggered' arrangement (see Figure 5.10 (b)), in which pairs of studs are welded on the 'favourable' and 'unfavourable' sides of the trough, would be equivalent to having two studs placed in the central position, so this arrangement may be adopted when it is not possible to place both in the 'favourable' position. However, in situations where trapezoidal decking with ribs transverse to the span of the beam is used, recent tests have shown that two studs will offer little benefit over a single stud

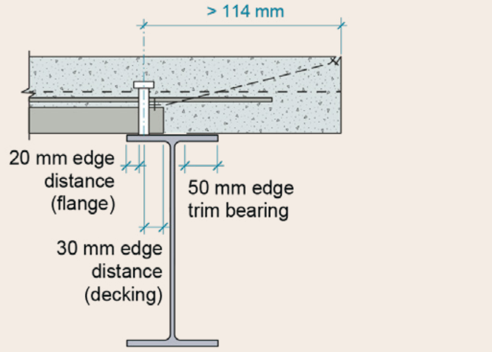


Figure 5.9 Shear connector detailing at an edge (for 19 mm diameter studs)

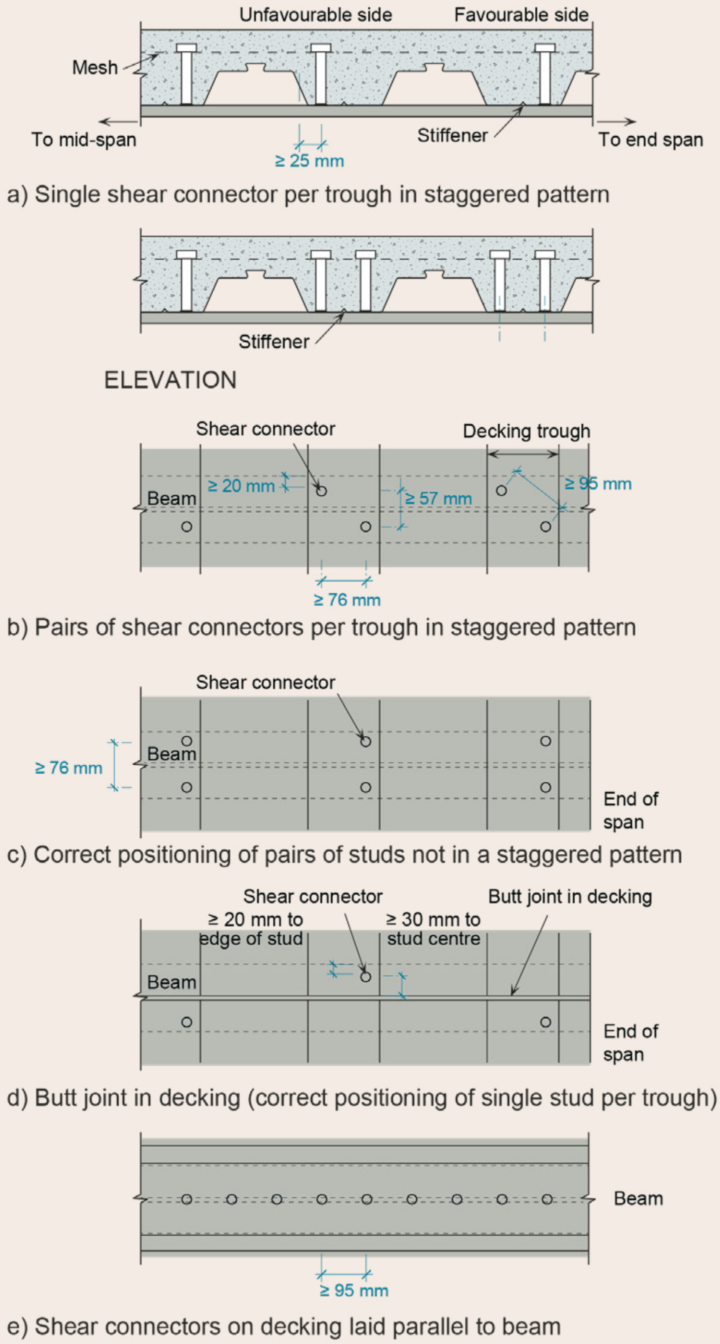


Figure 5.10 Detailing of shear connectors (19 mm diameter) welded through decking (note that a staggered position for pairs of studs should only be adopted when they cannot both be placed on the favourable side)

- At discontinuities in the decking, the studs should anchor both sheets. The minimum distance from the centre of the stud to the edge of each sheet should be 30 mm. Because of this, beams with flange widths less than 125 mm are not recommended – see notes on attachment of studs, above. [Note that studs should never be welded through two layers of decking. At joints, it is recommended the decking should be butted, and when studs are in single lines, they should be welded alternately on one sheet then the other (see Figure 5.10 (d)), and when in pairs they can be welded one on each.]
- Studs attached to edge beams should be placed no closer than 6*d* (from the stud centre-line) to the slab edge, as shown in Figure 5.9. Where the slab edge is less than 300 mm from the line of the studs, 'U' bars should be specified around the studs in accordance with BS EN 1994-1-1:2004 (or BS 5950-3) to prevent bursting of the concrete near the slab edge.

5.3.2 Longitudinal shear

Composite beams may be designed plastically if the shear connectors are sufficiently ductile. This enables a plastic shear connection resistance to be assumed, whereby the maximum resistance in each connector is assumed to occur simultaneously down the length of the beam. BS EN 1994-1-1:2004 states that 19 mm diameter studs with a 'length as welded' greater than 76 mm may be assumed to satisfy this requirement. The total longitudinal shear force that can be transferred across the steel-concrete interface is the sum of the resistances of the shear connectors positioned within the length between a support and the point of maximum bending moment. Where the loading is asymmetric, the lesser of the resistances totalled either side of the point of maximum bending should be used.

Transverse reinforcement

The longitudinal shear resistance of the concrete slab must be checked to ensure that the force from the shear connectors can be transferred into the slab without splitting the concrete. This requires the provision of transverse reinforcement (perpendicular to the beam centre-line), which may take the form of fibre reinforcement in the concrete. It is usually found that fabric or fibre reinforcement is sufficient for the design of secondary beams, where the decking ribs run perpendicular to the beam (as shown in Figure 5.11(a)). For beams where the ribs run parallel to the beam (Figure 5.11(b)) additional bar reinforcement is likely to be required. Potential shear planes through the slab lie on either side of the shear connectors (Figure 5.11). However, plane b-b need not be checked for composite slabs with single rows of studs as this failure will not occur in practice. The shear resistance per unit length of shear plane along the beam is a function of the concrete strength and the amount of reinforcement provided.

For edge beams, 'U' bars should be positioned as low as possible but with sufficient bottom cover for the aggregate to flow (BS 5950-3 states that the bars should be placed at least 15 mm below the head of the stud, although there is no such requirement in BS EN 1994-1-1:2004).

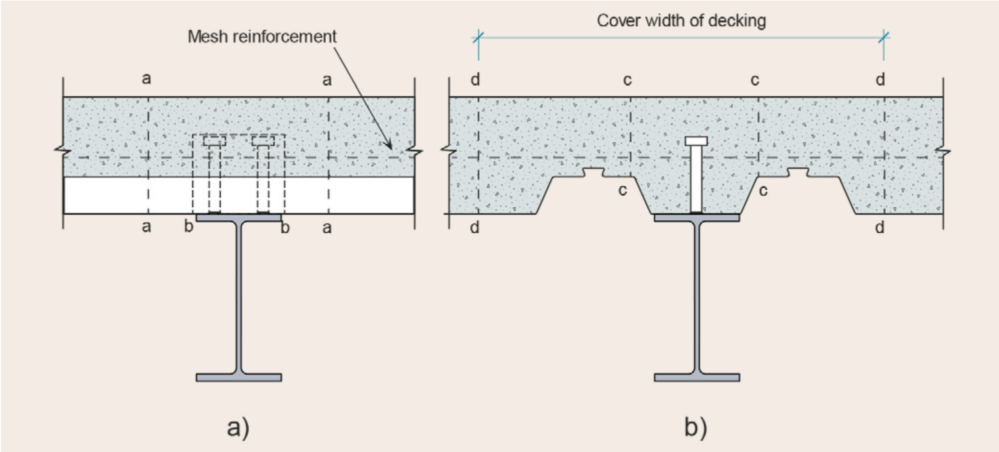


Figure 5.11 Potential failure planes through the slab in longitudinal shear (a) Decking perpendicular to beam (b) Decking parallel to beam

The decking may also act as part of the transverse reinforcement to contribute to the longitudinal shear resistance. The full resistance of the decking can be used when it is placed transverse to the beams and is continuous. In situations where the decking is discontinuous, the anchorage force that can be developed by the shear connectors limits this action, but the decking contribution may be included, provided that there is sufficient end distance of the decking beyond the centre line of the studs. Guidance on the anchorage resistance is given in BS EN 1994-1-1:2004 (clause 9.7.4(3)) and in BS 5950-4 (clause 6.4.3), but minimum values are quoted as 1.65 times stud diameter in the former and 1.7 times stud diameter in the latter.

The contribution of the decking should always be neglected where it is not properly anchored at discontinuities, or where the decking ribs run parallel to the beam.
In theory, when the decking is parallel to the beam and properly anchored, some contribution to the longitudinal shear resistance could be included. However, including this contribution is not recommended because the decking resistance is affected by the (unpredictable) presence of laps on site; this approach is consistent with BS EN 1994-1-1:2004. Studs fixed in a single line at a butt joint in the decking do not provide sufficient anchorage for the decking to contribute to the transverse reinforcement. Further guidance on transverse reinforcement can be found in AD 192[79], AD 266[80], AD 437[81] and AD 439[82].

Transverse reinforcement is always needed to ensure adequate performance of the shear connection. Fabric may be sufficient, but a check is always necessary, particularly for primary beams. Fabric is preferred because it minimises the need for steel fixers to work in a bent position. Fibre reinforcement may also be used if it provides sufficient resistance.

The contribution of the decking to the transverse reinforcement can only be included if it is properly anchored, and this depends on a number of factors - continuity of the decking, decking rib orientation, and laps in the decking. The contribution of the decking should always be neglected where the ribs run parallel to the beam.

Bending resistance envelope

The bending resistance of a composite beam depends on the shear transfer between the beam and the slab. Consequently, the resistance increases away from the supports; the resistance at a given point is a function of the resistance moment of the bare steel beam and the number of connectors between that point and the nearest support.

For a beam subject to uniformly distributed load, the maximum design moment is at mid span. It is only necessary for the PSD to check the moment resistance at this point and determine the total number of connectors needed to transfer the load into the slab (see Section 5.2.1 and Figure 5.3). The connectors should be evenly distributed between the support and mid-span.

For beams subject to point loads, it is necessary to check the resistance moment at intermediate points, not just the point of maximum design moment. For example, Figure 5.12 shows a beam with four-point loads, the applied bending moment diagram, and a resistance moment that is just sufficient at each of the four load positions. To achieve the required resistance at the intermediate point ‘A’, the number of connectors between that point and the nearest support must enable sufficient force to be transferred to the slab to achieve the required resistance M_A. With a heavy point load close to the support, it may not be possible to accommodate the required number of studs over that short length.

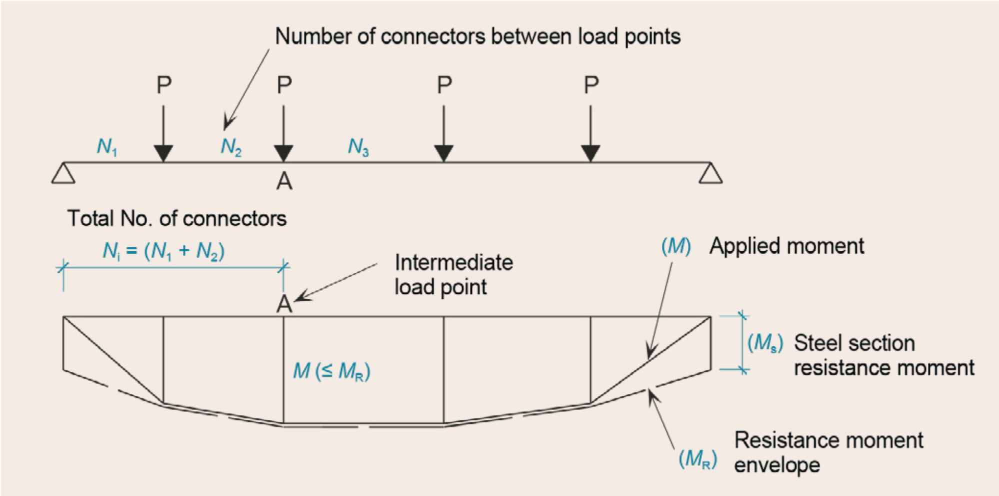


Figure 5.12 Shear connector distribution for beams subject to point loading

5.3.3 Degree of shear connection

The maximum longitudinal shear force that can be transferred from the steel to the concrete is the lesser of the compressive force to cause concrete crushing, and the force that would cause yielding of the steel section in tension. If sufficient shear connectors can be provided to transfer this force, the full plastic resistance moment of the composite section can be achieved. This is known as providing ‘full shear connection’.

‘Partial shear connection’ refers to a situation in which fewer shear connectors are provided. In this case, the stress block method used for calculating the resistance moment (see Section 5.2.1) must be modified to take into account the reduced longitudinal force that can be transferred. The use of partial connection often results in improved economy, and may be unavoidable when the number of shear connectors is limited by the spacing of the ribs in the decking.

Deformation of the shear connectors allows slip between the concrete and the steel section. This slip is zero at the point of maximum bending moment (often at mid-span) and increases towards the supports; the longer the beam span, the greater the slip at the supports. For partial shear connection, because there are fewer shear connectors, there will be more slip for a given load than with full shear connection. To avoid any adverse effects arising from excessive slip, a minimum limit to the degree of shear connection is specified in BS EN 1994-1-1:2004 and BS 5950-3, although the rules differ between them slightly.

BS EN 1994-1-1:2004 takes explicit account of three variables that affect minimum degree of shear connection – namely beam span, steel grade, and any asymmetry of the steel section. However, a number of other variables can also be considered, including the type and orientation of decking, whether or not the steel beam is unpropped during construction, the presence of regular large web openings, and the utilisation in bending of the composite beam. In many cases, consideration of this broader range of variables results in significantly less onerous requirements for the minimum degree, thereby enabling composite solutions that according to the code would be inadmissible. Further guidance on the degree of shear connection considering these variables can be found in P405 and AD 362[83]. It is expected that Generation 2 EN 1994-1-1 will include rules that are similar to those given in P405.

5.4 Further reading

The references given below relate particularly to beam design. For detailed information on authors and publishers, see Section 8, References.

Composite design of steel framed buildings (P359)[59]
This design guide provides a comprehensive design methodology for composite slabs on I section steel beams, for design in accordance with the Eurocodes. The guidance is complemented by a full numerical worked example.

Designer’s guide to BS EN 1994-1-1[46]
This guide provides the detailed background to the clauses to Eurocode 4. It includes many worked examples that show the application of the individual clauses. It is a standard reference for designers of composite beams to EC4.

Design of beams with large web openings for services (P355)[69]
This guide provides a design method for both composite and non-composite beams with web openings. A model for the complex force distribution around an opening is

explained, from which a simplified method for stiffened and un-stiffened openings is derived. Guidance on the positioning and size of openings is included.

Design of floors for vibration: A new approach (P354)[70]
This guide examines the theoretical aspects of vibrations in buildings caused by people walking. It shows how floors may be analysed for their dynamic sensitivity and suggests acceptance criteria for the floor response.

Fire protection for structural steel in buildings (4th Edition)[84]
This publication explains the basic aspects of fire protection and fire protection appraisal procedures. It gives application details for most of the products available, and protection thickness required for different beam and column exposure conditions. This is the standard design reference for fire protection of structural steel.

Code of Practice for Offsite Applied Thin Film Intumescent Coatings[72]
This guidance document describes the design and specification issues relating to off-site applied thin film intumescent coatings.

P178: Design for construction (P178)[85]
This guide highlights the effects of basic design decisions on the overall buildability and cost of a building. It is aimed at engineers primarily but has a relevance to all those having a design input.

Advisory Desk

The following 'Advisory Desk' items, published by the SCI in New Steel Construction and available on SteelBiz, provide further information relevant to this Section:

AD 174, *Shear connection along composite edge beams*[86]. This note outlines a method for checking the bending resistance of composite edge beams in existing buildings where the shear connectors have not been properly fixed, or the transverse reinforcement has been omitted.

AD 175, *Diaphragm action of steel decking during construction*[4]. A method is given in this note for checking the adequacy of the decking to stabilise the structure by providing diaphragm action. Fixing requirements are given.

AD 192, *Transverse reinforcement in composite T-beams*[79]. A detailed description of the principles of longitudinal shear, and the role of transverse reinforcement, is given in this note. The background to the relevant clauses in BS 5950-3 is explained.

AD 266, *Shear connection in composite beams*[80]. This note discusses the basis for effective breadth rules, the minimum degree of shear connection rules and transverse reinforcement calculations.

AD 362, October 2011, *Headed shear studs – Resistance and minimum degree of shear connection in composite beams with decking*. This AD highlights key changes to BS 5950-3.1:1990 in Amendment no 1, issued in January 2010, related to shear connectors. The Amendment reflects the findings of extensive experimental studies on the performance of “through-deck welded” headed shear studs in composite beams, as a result of which guidance on the design resistance and ductility of headed shear studs was revised.

AD 400, September 2016, T*he degree of shear connection in composite beams and SCI P405*. The stud resistances presented in both BS 5950-3 (as amended in 2010) and BS EN 1994-1-1 are lower than those given in the previous British Standard. This has resulted in many composite beam designs (that were previously satisfactory) becoming impossible to verify because the maximum number of studs that can be accommodated on a beam is often less than the number of studs needed to satisfy rules for minimum degree of shear connection.

AD 418, May 2018, *Web-post buckling in composite beams with rectangular and elongated web opening*s. The design of composite beams with large web openings is presented in SCI publication P355, which has been adopted in the development of software for the design of both hot rolled and fabricated steel sections with openings of various shapes and sizes. In P355, the method for addressing web buckling next to or between rectangular or elongated openings identifies two cases; closely spaced and widely spaced openings.

AD 419, June 2018, *Composite beams with different positions of web openings*. SCI publication P355 is widely used to design beams with large web openings. It is adopted in the development of software to design hot rolled and fabricated steel sections with openings of various shapes and sizes. The purpose of this Advisory Desk note is to address some common practical problems related to adjacent openings of different heights and positions.

AD 437, February 2020, *Curtailment of transverse bar reinforcement in composite beams with steel decking designed using Eurocodes*. The purpose of this Advisory Desk Note is to provide guidance on the curtailment of transverse bar reinforcement in slabs on composite beams with steel decking, designed to EN 1994-1 1. Such information was previously presented in AD 325, for design to BS 5950-3, but the provisions in EN 1994-1-1, and the clauses in EN 1992-1-1 to which it refers, give more explicit coverage of this topic than the BS rules. The approach to transverse bar curtailment is therefore different.

AD 439, April 2020, *Transverse reinforcement in composite beams*. This Advisory Desk note has been produced to reflect the publication in 2015 of P405 Minimum degree of shear connection rules for UK construction to Eurocode 4. As a result, AD 241: Transverse reinforcement in composite beams is redundant.

AD 441, May 2020, *Minimum degree of shear connection in composite beams*. Looks at the minimum degree of shear connection in composite beams, in particular why there is a need for such a limit.

AD 481, April 2022, *Composite beams with deep composite slabs*. This note deals with the design of composite beams with deep composite slabs and includes additional provisions for cases where the depth of the composite slab is greater than 180mm.

CONSTRUCTION PRACTICE – CONCRETE

This Section provides information concerning good practice in relation to the site activities associated with the procurement, placement and finishing of in-situ concrete. It is aimed at all personnel involved in the site activities. Guidance on concrete mix design is included, but guidance is not given on access requirements for concrete mixers, checking of concrete delivery notes etc. These issues are considered to be general site practice, and not appropriate for inclusion in a guide on composite construction.

6.1 Concrete supply

Concrete supply is normally the responsibility of the Principal Contractor, who should make sure that it is specified, supplied and assessed (normally) in accordance with BS 8500-1 to meet the strength grade specified by the PSD. Basic details for some typical concrete mixes are given in Table 6.1.

Aggregate Type	**Normal weight**					**Lightweight**		
Strength Class	**C25/30**	**C28/35**	**C32/40**	**C35/45**	**C40/50**	**LC25/28**	**LC28/31**	**LC32/35**
Max water cement ratio	0.65	0.60	0.55	0.50	0.45	0.65	0.60	0.55
Minumum cement content (kg/m^3)	260	280	300	320	340	260	280	300

Notes:

1. BS 8500-1 does not give universal relationships between strength class and water cement ratio and minimum cement content. The relationships depend upon the exposure class and cement types used. The relationships shown in Table 6.1 are those given in BS 8500-1 for XC exposure conditions. Other strength class/water cement ratio/cement content relationships are listed in BS 8500-1 for other exposure conditions
2. The minimum cement contents listed are for 20 mm aggregate. Generally minimum cement contents would need to be increased by 20 kg/m^3 for 14 mm aggregate and 40 kg/m^3 for 10 mm aggregate.

Table 6.1 Concrete specifications (extracted from BS 8500)

To ensure the quality control of the concrete mix, it should be obtained from a plant providing concrete in accordance with an approved quality assurance scheme.

Aggregate types and size

Most composite slabs are constructed with a normal aggregate, but lightweight aggregate is available.

When normal concrete is specified, the maximum size of the aggregate needs to be limited to ensure that the concrete may be placed easily into the decking ribs and between the reinforcing bars.

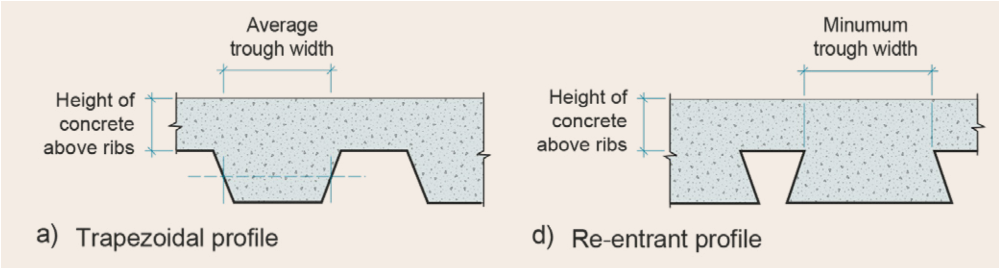
The nominal dimension of the largest aggregate, which has an angular nature, should not exceed the smallest of the following limits (see Figure 6.1).

- 40% of the concrete cover above the ribs
- The average width of the decking troughs (trapezoidal decking)
- One third of the minimum trough width (re-entrant decking).

It is recommended that 20 mm aggregate is used whenever possible. When smaller aggregates are used, the required cement content will increase, and the shrinkage performance of the concrete will be adversely affected.

With lightweight concrete, aggregate size is not a problem because of the small, rounded nature of the pellets.

Figure 6.1 Nominal cross-sectional dimensions used to determine maximum concrete aggregate size



Consistence

To ensure that the concrete is sufficiently workable to allow it to be pumped with the correct flow, and to achieve adequate compaction around the reinforcement, in the troughs of the decking and around the steel beams in shallow floors, a minimum consistence class of S3 should be specified in line with BS 8500-1.

Concrete mixes with low consistence should not be used as this can lead more readily to heaping of the concrete and overloading of the steel deck.

6.2 Placing concrete

6.2.1 Preparation

Prior to placing concrete on the decking, guard rails should be in position at all perimeters, internal edges and voids. The positions of any props (and back props) should be checked against the details shown on the decking layout drawings to ensure that the required support has been provided.

Cleaning the decking

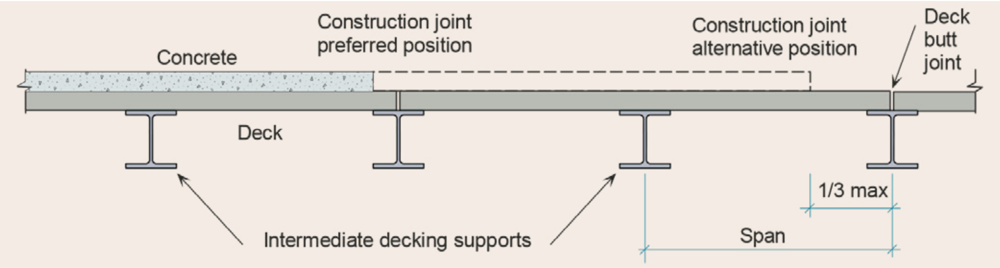
The surface of the decking should be reasonably free of dirt, oil, etc. prior to concreting. The slight surface grease that is present on the decking when it is delivered to site does not affect the assumed interaction between the concrete and steel, and therefore need not be removed. It is not essential to clean out all the broken ceramic ferrules that result from stud welding.

Construction joints

Typical pour sizes are up to 1000 m^2/day, although there is no technical limitation to the area that may be concreted. Where the limits of the pour do not coincide with permanent slab edges, construction joints (day joints) are used to define the extent of the pour.

Where possible, the construction joints should be located close to butt joints in the decking. For conventional composite beams, it is preferable to create the joint to one side of the line of the shear connectors, to ensure sound concrete around the studs. This does not affect the resistance of the shear connectors. If the construction joint cannot be made near a butt joint, it is suggested that no more than one-third of the decking span from a butt joint should be left unpoured, as shown in Figure 6.2. Concreting should not be stopped within a sheet length with more than this left unpoured because excessive deflections might occur when the loads on a continuous decking sheet are not balanced either side of the intermediate support beam.

Figure 6.2 Recommended positions for construction joints in the concrete slab



Stop ends, usually in the form of timber or plastic inserts, are used to create the construction joints. A number of proprietary products are available for this purpose, which may facilitate formation of the joint. As with all the joints and ends of the decking, they should be checked for potential grout loss.

A construction joint will form a discontinuity in the slab, so it is important that continuity reinforcement is provided across the joint. When fibre reinforcement is used, continuity reinforcement in the form of conventional reinforcing bars or a strip of steel fabric will be required at construction joints. The decking supplier should be consulted regarding the continuity reinforcement required in fibre reinforced slabs. The occurrence of cracking in the concrete adjacent to day joints is normal and does not affect the structural performance. If the size of the crack at the construction joint is important, e.g., when brittle finishes are being used, the reinforcement should be sized to control the crack width by the PSD. Alternatively, there are commercial systems for decoupling brittle finishes from the underlying composite concrete slab, and guidance on their use should be sought from the supplier.

Bar and fabric (mesh) reinforcement

All bar/fabric reinforcement should be properly supported so that it does not become displaced during concreting. Plastic stools, loops or preformed fabric may be used as ‘chairs’. When plastic channels are used care should be taken to ensure they do not induce cracking. Chairs should be robust, because operatives will need to use the floor as a working platform for themselves and their equipment. In particular, the handling and movement of concrete-filled pipes during pumping can cause significant local impacts on the fabric reinforcement. Although a slight depression (up to 15 mm) of the fabric may occur during concreting, the performance of the slab is not affected significantly by this. An example of a floor with the reinforcement in place and ready for casting is shown in Figure 6.3.

Figure 6.3 Decking and fabric reinforcement – ready for casting the concrete

courtesy of SMD

The reinforcement that has been fixed should be checked against the slab reinforcement drawing. Particular attention should be given to checking any additional bar reinforcement, such as may be needed around openings, across composite beams and U-bars for composite edge beams. When installing reinforcement, care must be taken at edges to ensure restraint straps to the edge trim are not damaged as this may adversely impact the deflection of the edge trim and the slab edge position achieved.

Alternatives to steel are available for use as reinforcement, for example glass fibre reinforced polymers may be formed into bars. When considering alternatives their behaviour should be carefully taken into account as part of the design process.

Fibre reinforcement

Steel fibres may be used as a structural alternative to reinforcing bars, in which case they are added to the concrete mix at the batching plant, or directly into the mixer on site. Fibre reinforced concrete can be pumped to elevated floors, as shown in Figure 6.4. When using a fibre reinforcement solution, it is still general practice to use U-bars on composite edge beams, bar reinforcement around openings in the slab, and fabric or bar reinforcement at construction joints, or where the composite slab cantilevers beyond a support. When preparing for concreting, the site team should ensure that any such bars/fabric are present.

Grout loss

The decking joints should be closely butted and exposed ends should be 'stopped' with proprietary filler pieces to avoid excessive grout loss. Gaps greater than 5 mm should be sealed, but gaps smaller than this do not need any special provision normally. Small gaps may be spray-filled with expanding polyurethane foam. The decking is not designed to give a watertight seal, dirty water and some aggregate fines seepage is to be expected (unless special measures are taken due to specific requirements concerning a soffit, some grout loss is inevitable). Consideration should be made for the provision of power washing activities to the underside of supporting steelwork and decking following concrete pouring.

Figure 6.4 Pumping of fibre-reinforced concrete

courtesy of SMD

Controlling thickness

When tamping rails are used they should be placed close to the beam centre-lines, to avoid excessive deflection during concreting. The level of the concrete top surface will then tend to reflect the deflected shape of the beams.

'Dipping' is a more effective way to ensure a constant depth of concrete, with a top surface that also follows the deflected shape of the decking.

Mass flood technique

Where a 'mass flood' technique of casting (whereby the concrete is poured and the whole floor levelled to a fixed datum) is used, considerably more concrete will be needed and thicker slabs will result from deflections of the steelwork and decking. Levelling to a fixed datum should not be adopted without first confirming with the PSD that the extra weight of concrete 'ponding' has been allowed for in the design. The recommended method of pouring concrete is by constant depth using a depth gauge. For further information see AD 344 and AD 410.

6.2.2 Placement

The concrete should be well compacted, particularly near and around any shear connectors. This can be done using a vibrating beam, which will require adequate supports at both ends, or by an immersion poker vibrator. When the latter is used care must be taken to avoid damaging the decking. Hand tamping is not recommended as a way of compacting the concrete.

Concrete can be placed when the air temperature is 5 °C or above, some will supply if 3 °C and rising. In cold weather, it may be necessary to make provision to maintain an adequate temperature during at least part of the curing period (see below).

Concrete pumping

Pumping has for a long time been the normal way of placing concrete and can be adopted for both normal and lightweight aggregate mixes. Flow rates in the order of 0.5 to 1 m^3 of concrete per minute can be achieved, although, clearly, the longer the pump lines and the higher the concrete is to be pumped, the slower the operation. Some pumps can 'lift' the concrete up to 50 m. Secondary pumps, placed at intermediate levels, may be used to achieve higher lifts.

Pump lines are normally 100-150 mm in diameter and are assembled in segments. Because the force exerted at bends can be significant, straight line pumping is preferred. The lines should be supported on timber blocks at intervals of 2 to 3 m. Resetting of pump lines is required at frequent intervals as the pour progresses. This means that the outlet pipe should be moved frequently and carefully so that concrete heaping is minimised. A minimum of two operatives is necessary for this operation, one to hold and manoeuvre the outlet pipe, the other to shovel away excess concrete. No more than four workers should be present around the pipe outlet during pumping, because of the potential for overloading the decking. The concrete should not be dropped from the outlet pipe onto the decking from a height of more than about 1 m.

Any low quality concrete (the first part of each lorry load, or after flushing out pipeline blockages) should be discarded.

Skip and barrow

Whilst ideally concrete should be placed by pump, there will be occasions when small areas need to be concreted and placing by pump is not practical. Considerable care is needed if a skip and barrows are to be used, to ensure the decking is not overloaded. It would be preferable to discharge concrete into barrows on previously constructed areas, to avoid concrete being discharged directly from a skip onto decking. Placing concrete from a skip hung from a crane may be difficult because of obstructions from beams and decking at higher floor levels. However, despite being time consuming (progress rates rarely exceed 5 m^3 per hour), it is sometimes efficient to use the skip and barrow technique for small infill bays.

Skips should have a means of controlling the rate of discharge and should not be discharged from more than 0.5 metres above the decking or barrow. When discharging into a barrow, the barrow should be supported by thick (30 mm) boards covering a 2 m by 2 m area, or by a finished part of the slab. Either provision limits impact loads. Barrows should be run over thick boards placed on the fabric reinforcement, which should be supported locally.

Testing

The concrete should normally be tested in accordance with the requirements of BS EN 12350-1:2019[87]. Specifications for the required number of concrete cube tests vary, so the designer should be consulted if there is any lack of clarity. The cubes are crushed at 28 days, and the average of the cubes' strengths becomes the individual 28-day result for the batch sampled. Additional cubes may be taken for testing at 7 days, or other ages, for the determination of strength.

6.2.3 Finishing, curing and drying

The concrete surface finish is normally specified by the PSD (Section 4.2.1). If power-floating is to be carried out, this should be done within 2-3 hours of casting. This allows time for the concrete to sufficiently harden. As discussed in Section 4.1.2, it should be checked that this load does not exceed the allowable temporary construction loading of 1.5kN/m^2 over the 3 m × 3 m 'working area', see section 6.3.2 for further information.

Experience shows that there is a high risk that well trowelled surfaces will exhibit crazing[88]. Crazing is the term used to describe an irregular polygonal pattern of fine interconnected cracks which often occur on power trowelled concrete surfaces. Crazing should not be considered a defect and it generally has no adverse effect on the performance of the floor surface.

Although concrete normally gains strength relatively quickly, it is necessary to keep temperatures above 5°C for at least 3 days after pouring. When concreting during the winter months, loss of heat, such as by radiation from the lower surface of the decking at night, can be significant. It may then be necessary to use space-heaters to maintain the temperature. Some heat is generated during setting, or 'hydration', of the concrete (this raises the temperature by 3 to 5°C, typically).

The moisture in the concrete should not be allowed to evaporate too early, otherwise the surface may lose its integrity, forming dust and possibly cracking, and it will not have a good abrasion resistance. The slab therefore needs to be 'cured' either by spraying with a proprietary curing compound, or by covering the surface with polythene sheeting for 3 to 7 days, depending on the weather (this is particularly important in warm or windy weather).

Because the concrete is only exposed on one surface of a composite floor, it can take longer than a traditional reinforced concrete slab to dry out. The preferred method of checking the moisture content of the slab is the insulated hygrometer method given in BS 8203[89].

6.3 Loads on the slab during and after concreting

6.3.1 Loads during concreting

Loads during concreting arise mainly from the weight of the operatives, concrete, pump-lines and impact forces. Loads to be taken into account for design during concreting are specified in BS EN 1991-1-6:2005 and in BS 5950-3 and are outlined in Section 4.1.2. The self-weight of the finished slab (typically 2 to 3 kN/m^2) and local loading (caused by normal localised heaping of the concrete) are included. This is usually not critical because adjacent areas of decking are unloaded, or only partially loaded.

The following list describes the loads that usually arise during concreting, and that will normally have been allowed for by the SSD:

- A concrete gang consisting of five or six people (only four of whom are within 2 m of the pump outlet)
- Concrete that is poured from no higher than knee level above the decking (to avoid excessive impact loading)
- A 100-150 mm diameter pipeline full of concrete. The weight of this line should be adequately spread across the decking by using suitable timbers to avoid local damage to the deck
- A cone of heaped concrete of approximately 0.2 m height and 1 m base. It will have been assumed that the pump line outlet will be moved frequently to avoid excessive heaping (or, if a skip is used, the discharge will be carefully controlled).

Additional concrete may be placed because of deflections of both the decking and the steel frame, particularly if the slab is finished to ‘absolute’ (datum) levels or to achieve a certain flatness of the upper surface. The SSD must be consulted to confirm whether the resultant increased loads have been allowed for in the design. Levelling the top of slab to achieve a uniform thickness, rather than a ‘level’ top surface is recommended and will avoid this situation.

6.3.2 Construction loads after concreting

Construction loads are often applied to the slab soon after concreting. Examples of commonly occurring loads are power floats, bags of fire protection, skips of debris, pallets of blocks and other equipment. If these loads are no more than 1.5 kN/m^2 (over a 3 m by 3 m area), the construction load used in the design of the decking, then the slab is clearly not overloaded (provided there is no additional, unforeseen load due to ‘ponding’). For loads greater than this, the concrete strength will need to be relied upon. Props should not be removed, nor additional loads applied, until the concrete has reached 75% of its design strength, as indicated by ‘control’ concrete compression tests. If the slab is to be loaded before 28 days after concreting, its strength at the time of loading needs to be established (possibly by testing cubes or cylinders early), and an effective ‘design strength’ agreed with the PSD.

The following list gives some examples of typical construction loads. It may be necessary to make special provisions to guarantee that the items are placed on pallets, positioned directly over the support beams. Construction loads should not be applied to the slab soon after concreting:

- Concrete blocks: a 1 m high pallet of blocks applies a load up to 10 kN/m^2. Bricks: a 1 m High pallet of bricks can exert a load of over 15 kN/m^2
- Bags of fire protection: a bag of fire protection material normally weighs 25 kg. A 1 m high pallet of bags can be equivalent to a load of 2.5 kN/m^2
- Bags of cement: bags of cement weigh 25 kg each. A standard pallet of these weighs 1,400 kg (12 kN/m^2).

The application of very heavy construction loads should always be referred to the SSD. When considering the location of such loads, it is best to position them over the beams wherever possible. Examples of such loads are:

- Generators: welding generators can apply a load of 50 kN
- Fork lift trucks: fork lift trucks can exert a load up to 100 kN, not including their live load. In general, vehicles with axle weights above 3 tonnes should be used only if the slab has been designed/checked specifically for that purpose
- Crane counter weights: each counter weight is marked clearly with the value of its weight
- Mobile access platforms: The potential loading imposed by any mobile access platforms used to install services, finishes, etc should be checked.

BS EN 1994-1-1:2004 clause 9.4.3, entitled effective width of composite slabs under concentrated point and line loads has been the cause of much confusion, in particular the interpretation of load limits of 7.5 kN and 5.0 kN/m^2 quoted in its part 5. The guidance that ‘nominal transverse reinforcement may be used without calculation’ (i.e. assumed to be adequate) provided these limits are not exceeded has been widely misunderstood. AD 450 explains how the designer should consider concentrated loads in their design, and gives a simple way of determining the transverse bending moment that should be used to explicitly design the transverse reinforcement in the beam strip that passes under a load. Even when the load limits stated in BS EN 1994-1-1:2004 are not exceeded, explicit design of this reinforcement may be needed (for example when there is more than one concentrated load present , or when the transverse reinforcement is placed relatively high in the slab). AD 477 provides further guidance on the topic by proposing a more sophisticated approach, as a supplement rather than a replacement for the approach in AD 450. This more advanced approach is particularly relevant for the UK market, where typically we rely on a single layer of reinforcement with a minimal cover to the top surface of the slab, for which the simplified method proposed by AD 450 may result in an onerous requirement for the area of reinforcement required to resist transverse bending. Both ADs are considered in SCI's Tedds module which provides a way of optimising the slab design by iteration. This approach enables the width of the longitudinal strip to be reduced (provided it still passes the relevant verifications), so that the amount of transverse bending is reduced.

Care is needed if a composite floor is to be used in situations where there could be frequent vehicle movements. Designing such floors just for uniformly distributed loads may not be satisfactory. The fatigue effects of the repeated dynamic loading from vehicles on the slab and supporting beams must be considered by the PSD. The suitability of the floor design (beams and slab) for dynamic loading from vehicles should be checked.

Where the concrete is to be used as a wearing surface, or where bonded finishes are specified, the concrete surface should be protected from oil spillage and damage from moving plant.

6.4 Further reading

The references below relate particularly to this Section. (For information on authors and publishers, see Section 8, References.)

Good construction practice for composite slabs[90]
This document covers much of the information included in the current guide but has a bias towards the European market.

Concrete Society Technical Report No. 34, 2018 - Concrete industrial ground floors a guide to design and construction.
This publication provides comprehensive guidance on the design, specification, construction and finishing of industrial concrete ground floors. The guidance on concrete quality requirements for durability, and on finishing procedures, is also relevant to suspended composite slabs.

Concrete Society Technical Report No. 63, 2007 - Guidance for the design of steel-fibre-reinforced concrete[40].
This publication summarises the range of applications for steel-fibre-reinforced concrete and includes practical aspects such as production and quality control.

Concrete Society Technical Report No. 75, 2016 - Composite concrete slabs using steel decking.
This publication provides guidance for the construction of composite floor slabs on steel decking and complements the design guidance given in P300.

Additional information concerning concrete may be found at The Concrete Society and The Concrete Centre.

Advisory Desk

AD 344, *Levelling techniques for composite floors*. This Note highlights the importance of considering potential levelling techniques of composite floor slabs in relation to achieving the specified tolerances and a safe design. Levelling methods are covered and the issues of pre-cambering, propping of decking, ponding, flatness and design approaches are discussed.

AD 347, *Saw cutting of composite slabs to control cracking*. This Note discusses the issues involved in the use of saw cutting to form crack inducing joints in composite slabs and to emphasise the risks involved and the care needed in practice.

AD 367, *Construction loading for composite slabs - update to P364*. This Advisory Desk Note provides clarification about the different design approaches adopted in worked examples in SCI publications P359 and P364.

DEMOUNTABLE COMPOSITE CONSTRUCTION SYSTEMS

Many types of building might have a relatively short life span in their first cycle of use, but despite this the economic and sustainability benefits of composite construction can be retained by being able to demount and rebuild the structure with no damage or loss of performance. Furthermore, the requirements of the circular economy may, in the future, extend to buildings and their primary structure, so that all buildings should possess the ability to be adapted and modified over their lives, and potentially their components should be capable of being reused.

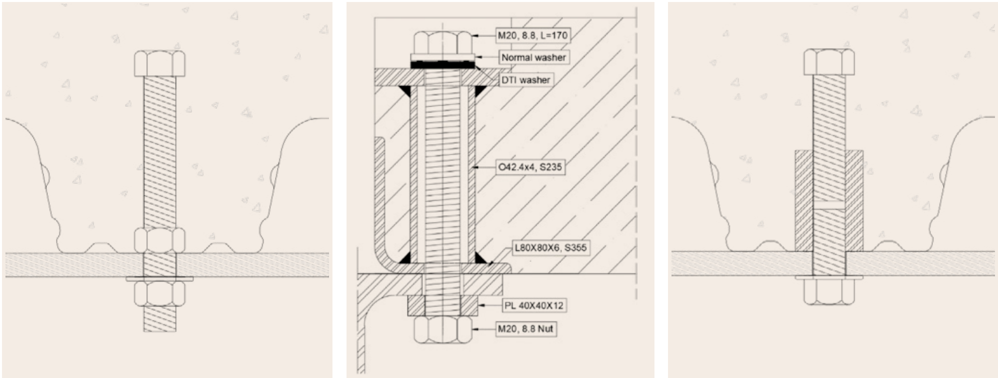
Practical application for the use of demountable construction can be found in:

- Multi-storey car parks, which may need to be demounted and rebuilt depending on future uses of the site or in re-planning of the road system, particularly in city centres,
- Schools and educational buildings that need to respond to changing educational demands, often by being moved and rebuilt on the same site
- Out of town business parks whose office buildings are often standardised in plan form and which also have to respond to changing market needs, whilst retaining the asset value of the buildings
- ‘Pop-up’ type developments on sites of short-term planning use, particularly in urban areas. These developments are often on previously used or brownfield sites and so the new development has to minimise its intervention in terms of foundations and services
- Industrial buildings subject to high loading on the production and storage areas, which should respond to changes in production requirements over time, for example by the construction or demounting of intermediate floors. Examples of the requirements for more flexible spatial uses are in the chemical and process industries.

Composite beams may be designed to be demountable by using various shear connector systems that can be disconnected from the beams. Different systems, that may be used in combination with composite floor slabs, are described as follows:

- Bolts with double nuts above/below the beam flange. In this case, the bolts are embedded in the concrete and are connected and disconnected from the underside of the beam flange, see Figure 7.1a
- Stud shear connectors with threaded ends and with nuts above/below the beam flange

- Friction grip bolts placed in a tightly-fitting cylinder with a bolt below the flange and tightened from above, see Figure 7.1b
- Bolts placed from below the beam flange that are connected to couplers embedded in the slab, see Figure 7.1c.



a) High-strength structural bolt b) Friction grip bolt c) Coupler system

Figure 7.1 Alternative demountable shear connection systems

While there is a general desire to move towards adaptable and reusable buildings, as part of a circular economy, there are currently few drivers or requirements to encourage this change in design approach. Design for deconstruction will inevitably cost more initially, but whole life costing over two or more building cycles may demonstrate the longer-term economic advantage of this new approach to design. Ideally, the beam and slab are reused in the same configuration, for example, if the building is demounted and rebuilt as a structural entity.

Various techniques are proposed by which the beam and slab components in a composite structure may be reused easily:

- The slab is cut into single span segments of 1.5 m to 2.4 m width and the segments can be reused with the same beams by placing the shear connectors in the pre-existing holes and then grouting the joints between the segments. The 4 to 6 mm joint between the segments is sufficient for the grout to be effective in transferring compression forces for composite design in the second cycle. Diaphragm action is reduced by the loss of the fully continuous slab and so it may be necessary to introduce a form of shear key between the segments. Further cycles of use are possible
- The slab is cut into segments and the segments can be reused with the same beams by placing the shear connectors in the pre-existing holes. A 50 mm concrete topping with light fabric reinforcement is placed on segments to provide a new slab. The composite beam does not rely on compression transfer through the existing segments, unless they are grouted. The self-weight of the slab is increased by 1.2 kN/m^2 but the bending resistance and stiffness of the composite section are also increased because the overall depth is increased (by the additional concrete layer). This system has the advantage that diaphragm action is maintained but further cycles of use of the floor slab may not be possible
- The steel beams can be reused with a new composite slab.

To facilitate demounting and reuse of the composite slab, pairs of cold formed, steel edge trims are placed along the centreline of the secondary beams to provide a pre-determined cut-line in the slab. These edge trims are similar to those used around the edge of the slab. Two forms of construction are proposed: (i) full depth trims in which the edge trims are equal to the slab depth and so form a discontinuity, and (ii) partial depth trims (see Figure 7.2) where the edge trims are shallower than the slab depth so that fabric reinforcement can be placed over them and the slab is continuous (a minimum of 30 mm top cover is recommended). In this approach, the slab has to be cut along the centre-line of the beam to expose the edge trim and to be able to separate and reuse the slab segments. It was found from tests that the full depth trim may lead to a slight reduction in the characteristic resistance of the shear connectors. Also, it could be subject to damage due to wear if the slab surface is exposed such as in a car park or a factory floor.

Figure 7.2 Partial depth edge trims and U-bar details

Some adjustments to the beam design process are needed when using demountable shear connectors. For the ultimate limit state two options are proposed in P428[91], (i) a modified plastic method using a factor k_{flex} that takes account of the load-slip relationship and the spacing of the shear connectors, and (ii) an elastic method in which the bending resistance is limited by the design strength of the steel and concrete, and the resistance of the shear connectors. The elastic method takes account of a non-uniform arrangement of shear connectors by use of an equivalent spacing, which means that the shear connector distribution can be optimised for beams with low degrees of shear connection in unpropped construction.

It is also recognised that design for the first use should satisfy strict serviceability limits to avoid any permanent deformation that may affect deconstruction and the subsequent cycles of use. In this respect, the end slip should be controlled to a value that ensures that the shear connectors will not experience plastic deformations. New formulae are presented in P428 defining the effective stiffness, elastic section modulus, and end slip of composite beams based on the shear connector stiffness and equivalent spacing.

Further information on sustainability of composite flooring systems is given in *Composite flooring systems: sustainable construction solutions*[92].

REFERENCES

[1] Construction (Design and Management) Regulations 2015 (CDM 2015)

[2] MANN, A.
Safe erection of steel structures,
The Structural Engineer, Vol. 100 (9), September 2022

[3] Code of Practice for Metal Decking and Stud Welding
BCSA, 2014

[4] AD 175: Diaphragm action of steel decking during construction.
Advisory Desk, in New Steel Construction, Vol 3 (4), Aug 1995

[5] Temporary fixing of metal decking.
Technical, in New Steel Construction, Vol 30 (3), March 2022

[6] BS EN 10346:2015 Continuously hot-dip coated steel flat products for cold forming - technical delivery conditions
BSI

[7] AD 365: Welding of shear studs to galvanized steel beams.
Advisory Desk, in New Steel Construction, Vol 20 (2), Feb 2012

[8] AD 247: Use of composite construction in an aggressive environment.
Advisory Desk, in New Steel Construction, Vol 9(2), March 2001

[9] BS EN 1991-1-6:2005 Eurocode 1. Actions on structures. General actions. Actions during execution
BSI

[10] AD 371: Design of cold-formed steel trapezoidal sheeting.
Technical, in New Steel Construction, Vol 20 (9), Nov/Dec 2012

[11] BS 5950: Structural use of Steelwork in Building:

BS 5950-1:2000 Structural use of steelwork in building. Code of practice for design. Rolled and welded sections
BS 5950-3-1:1990 Design in composite construction. Code of practice for design of simple and continuous composite beams
BS 5950-4:1994 Structural use of steelwork in building. Code of practice for design of composite slabs with profiled steel sheeting

BS 5950-6:1995 Structural use of steelwork in building. Code of practice for design of light gauge profiled steel sheeting
BS 5950-8:2003 Code of practice for fire resistant design
BSI

[12] BS EN 1991-1-1:2002 Eurocode 1. Actions on structures. General actions. Densities, self-weight, imposed loads for buildings
BSI

[13] AD 346: Design actions during concreting for beams and decking in composite floors
Advisory Desk, in New Steel Construction, Vol 18 (6), July 2011

[14] AD 367: Construction loading for composite slabs – update to P364
Advisory Desk, in New Steel Construction, Vol 20 (5), May 2012

[15] BS EN 1993-1-3:2006 Eurocode 3. Design of steel structures. General rules. Supplementary rules for cold-formed members and sheeting
BSI

[16] BS EN 1994-1-1:2004 Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings
BSI

[17] BS EN 1090-4: 2018 Execution of steel structures and aluminium structures. Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications
BSI

[18] BS EN 1992-1-1:2004 Eurocode 2: Design of concrete structures. General rules and rules for buildings BSI

[19] BS EN 206:2013 Concrete - Specification, performance, production and conformity
BSI

[20] BS 8500-1:2015 Concrete - Complementary British Standard to BS EN 206. Part 1: Method of specifying and guidance for the specifier
BSI.

[21] Low carbon concrete – what you need to know.
Technical, in New Steel Construction, Vol 29 (10), Nov/Dec 2021

[22] BS 8204-2:2003 Screeds, bases and in situ floorings. Concrete wearing surfaces. Code of practice
BSI

[23] Concrete industrial ground floors - A guide to their design and construction. Third Edition (TR34)
The Concrete Society, 2013

[24] AD 163: Provision for water vapour release in composite slabs
Advisory Desk, in New Steel Construction, Vol 2 (6), December 1994

[25] BS EN 13670:2009 Execution of concrete structures
BSI

[26] National Structural Concrete Specification for Building Construction (4th edition)
CONSTRUCT, 2010

[27] AD 344: Levelling techniques for composite floors
Advisory Desk, in New Steel Construction, Vol 18 (4), April 2010

[28] AD 410: Pouring concrete to a constant thickness or to a constant plane
Advisory Desk, in New Steel Construction, Vol 25 (8), September 2017

[29] Composite concrete slabs using steel decking: guidance on construction and associated design considerations (TR75)
The Concrete Society, 2016

[30] Surface regularity – levelness and flatness (CAS65)
The Concrete Society, 2022

[31] BS 4483:2005 Steel fabric for the reinforcement of concrete. Specification.
BSI

[32] BS 4449:2005 Steel for the reinforcement of concrete. Weldable reinforcing steel. Bar, coil and decoiled product. Specification.
BSI

[33] BS 8110-1:1997 Structural use of concrete. Code of practice for design and construction BSI

[34] BS 8666:2020 Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete. Specification.
BSI

[35] AD 461: Anchorage of bars in the troughs of composite slabs.
Advisory Desk, in New Steel Construction, Vol 29(5), May 2021

[36] BOND, A.J., et al.
How to Design Concrete Structures using Eurocode 2. Second edition
The Concrete Centre, 2018

[37] Guidance on the use of macro-synthetic fibre reinforced concrete (TR65)
The Concrete Society, 2007

[38] BS EN 14889 Fibres for concrete:.
BS EN 14889-1:2006 Steel fibres. Definitions, specifications and conformity.
BS EN 14889-2:2006 Polymer fibres. Definitions, specifications and conformity
BSI

[39] Use of fibre-reinforced concrete in composite slabs.
Technical, in New Steel Construction, Vol 31 (2), Feb 2023

[40] Guidance for the design of steel-fibre-reinforced concrete (TR63)
The Concrete Society, 2007

[41] AD 450, Resistance of composite slabs to concentrated loads.
Advisory Desk, in New Steel Construction, Vol 28(9), Oct 2020

[42] AD 477, Transverse bending of composite slabs subjected to point loads.
Advisory Desk, in New Steel Construction, Vol 30(2), Feb 2022

[43] Vertical resistance of composite slabs.
Eurocode Nugget 02, vol. 2, Jan 2023
Available from: https://portal.steel-sci.com/documents.html.

[44] Cracking in composite/corrugated metal decking floor slabs (CAS13)
The Concrete Society, 2016

[45] AD 347, Saw cutting of composite slabs to control cracking.
Advisory Desk, in New Steel Construction, Vol 18(7), July/Aug 2010

[46] JOHNSON, R.P.
Designers' Guide to Eurocode 4: Design of Composite Steel and Concrete Structures, Second edition
The Institute of Civil Engineers, 2014

[47] BS EN 1994-1-2:2005 Eurocode 4. Design of composite steel and concrete structures. General rules. Structural fire design
BSI

[48] NA to BS EN 1994-1-2:2005 UK National Annex to Eurocode 4. Design of composite steel and concrete structures. General rules. Structural fire. Design
BSI

[49] PN005c-GB.
Fire resistance design of composite slabs.
Available from https://portal.steel-sci.com/documents.html.

[50] SIMMS, W.I.
Fire resistance design of steel framed buildings (P375)
SCI, 2012.

[51] AD 447: Openings in composite slabs
Advisory Desk, in New Steel Construction, Vol 28(7), July 2020

[52] Backpropping Flat Slabs by Eur Ing Pallett Bsc Eng FICE FCS – PFP/136 (23/08/04): Pallett TemporaryWorks Ltd

[53] McKENNA, P.D. and LAWSON, R.M.
Interfaces: Design of steel framed buildings for service integration (P166)
SCI, 1997

[54] PD 6697: 2019 Recommendations for the design of masonry structures to BS EN 1996-1-1 and BS EN 1996-2
BSI

[55] OGDEN, R.G.
Interfaces: Curtain wall connections to steel frames (P101)
SCI, 1992

[56] WAY, A.G. and COUCHMAN, G.H.
Acoustic Detailing for Steel Construction (P372)
SCI, 2008

[57] Approved Document E: Resistance to the passage of sound
The Building Regulations 2010.
UK Government, 2015

[58] Part E Robust Details Handbook 4th edition
Robust Details Ltd

[59] SIMMS, W.I. and HUGHS, A.F.
Composite design of steel framed buildings (P359)
SCI, 2011

[60] AD 150, Composite floors - wheel loads from forklift trucks
Advisory Desk, in New Steel Construction, Vol 1(7), Dec 1993

[61] AD 450, Resistance of composite slabs to concentrated loads.
Advisory Desk, in New Steel Construction, Vol 28(9), Oct 2020

[62] PN001a-GB.
Resistance of headed stud shear connectors in transverse sheeting.
Available from: https://portal.steel-sci.com/documents.html.
SCI, 2010.

[63] COUCHMAN, G.
Minimum degree of shear connection rules for UK construction to Eurocode 4 (P405)
SCI, 2015.

[64] AD 343, Position of reinforcing mesh relative to stud shear connectors in composite slabs.
Advisory Desk, in New Steel Construction, Vol 18(3), March 2010

[65] LAWSON, R.M.
Commentary on BS 5950: Part 3: Section 3.1 ‘Composite Beams’ (P078)
SCI, 1990

[66] BS EN 1993-1-1:2005 Eurocode 3. Design of steel structures. General rules and rules for buildings
BSI

[67] NETHERCOT, D.A. and LAWSON, R.M.
Lateral stability of steel beams and columns (P093)
SCI, 1992

[68] BS EN 1990:2002 Eurocode. Basis of structural design
BSI

[69] LAWSON, R.M. and HICKS, S.J.
Design of beams with large web openings for services (P355)
SCI, 2011.

[70] SMITH, A.L., HICKS, S.J. and DEVINE, P.J.
Design of floors for vibration: A new approach (P354)
SCI, 2007

[71] NEWMAN, G.M., ROBINSON, J.T. and BAILEY, C.G.
A new approach to multi-storey steel-framed buildings. Second Edition (P288)
SCI, 2006.

[72] Code of Practice for Offsite Applied Thin Film Intumescent Coatings
ASFP, 2010

[73] RT1356: Fire Design of Composite Beams With Rectangular and Circular Web Openings
SCI, 2013.

[74] BS 476-21:1987 Fire tests on building materials and structures. Methods for determination of the fire resistance of loadbearing elements of construction
BSI

[75] NEWMAN, G.M. and LAWSON, R.M.
Fire resistance of composite beams (P109)
SCI, 1991

[76] AD 380, What height of shear stud should be used in Eurocode 4?.
Advisory Desk, in New Steel Construction, Vol 22 (2), Mar/Apr 2014

[77] AD 424, Shear stud length.
Advisory Desk, in New Steel Construction, Vol 26 (10), Nov/Dec 2018

[78] HICKS, S.J.
Strength and ductility of headed stud connectors welded in modern profiled steel sheeting,
The Structural Engineer, Vol. 85(10), May 2007, pp 32-38

[79] AD 192, Transverse reinforcement in composite T beams.
Advisory Desk, in New Steel Construction, Vol 5 (1), Feb/Mar 1997

[80] AD 266, Shear connection in composite beams.

Advisory Desk, in New Steel Construction, Vol 11 (4), July 2003

[81] AD 437, Curtailment of transverse bar reinforcement in composite beams with steel decking designed using Eurocodes
Advisory Desk, in New Steel Construction, Vol 28 (2), February 2020

[82] AD 439, Transverse reinforcement in composite beams
Advisory Desk, in New Steel Construction, Vol 28 (4), April 2020

[83] AD 362, Headed shear studs - Resistance and minimum degree of shear connection in composite beams with decking
Advisory Desk, in New Steel Construction, Vol 19 (9), October 2011

[84] Fire protection for structural steel in buildings (5th edition)
ASFP, 2014.

[85] CIMSTEEL
Design for Construction (P178)
SCI, 1997

[86] AD 174, Shear connection along composite edge beams.
Advisory Desk, in New Steel Construction, Vol 3 (3), June 1995

[87] BS EN 12350-1:2019 Testing fresh concrete. Sampling and common apparatus
BSI

[88] Crazing: power trowelled concrete for floor slabs (CAS8)
The Concrete Society, 2003

[89] BS 8203:2017 Installation of resilient floor coverings – code of practice
BSI

[90] Good construction practice for composite slabs
Publication No. 73, European Convention for Constructional Steelwork
Technical Committee 7 - Working Group 7.6
ECCS, 1993

[91] Guidance on Demountable composite construction systems for UK practice (P428)
SCI, 2020.

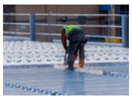
[92] Composite flooring systems: sustainable construction solutions
MCRMA, 2014

CREDITS

Front Cover
Kingspan

ii Composite Profiles UK

iv Pro-deck Fixing



vi Raised Floor Solutions

viii Composite Profiles UK

6 SMD

16 Severfield

24 Raised Floor Solutions

70 SMD

96 Severfield

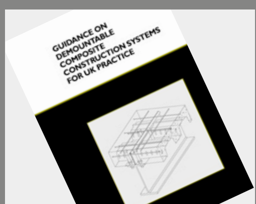
108 Severfield

112 Severfield

COMPOSITE SLABS AND BEAMS USING STEEL DECKING: GOOD PRACTICE FOR DESIGN AND CONSTRUCTION

This revised edition reflects the latest guidance for good practice and gives information on design to the Eurocodes as well as design to BS 5950, including comments on changes that will be incorporated in the second generation of Eurocodes. Information on the specification and use of low carbon concrete has been added as well as a new chapter on demountable composite construction systems.

Complementary titles



P428 | Guidance on demountable composite construction systems for UK practice

P359 | Composite design of steel framed buildings

SCI Ref: P300, 3rd Edition

ISBN 978-1-85942-252-6

9 781859 422526

SCI
Silwood Park, Ascot, Berkshire. SL5 7QN UK
T: +44 (0)1344 636525
F: +44 (0)1344 636570
E: reception@steel-sci.com
www.steel-sci.com