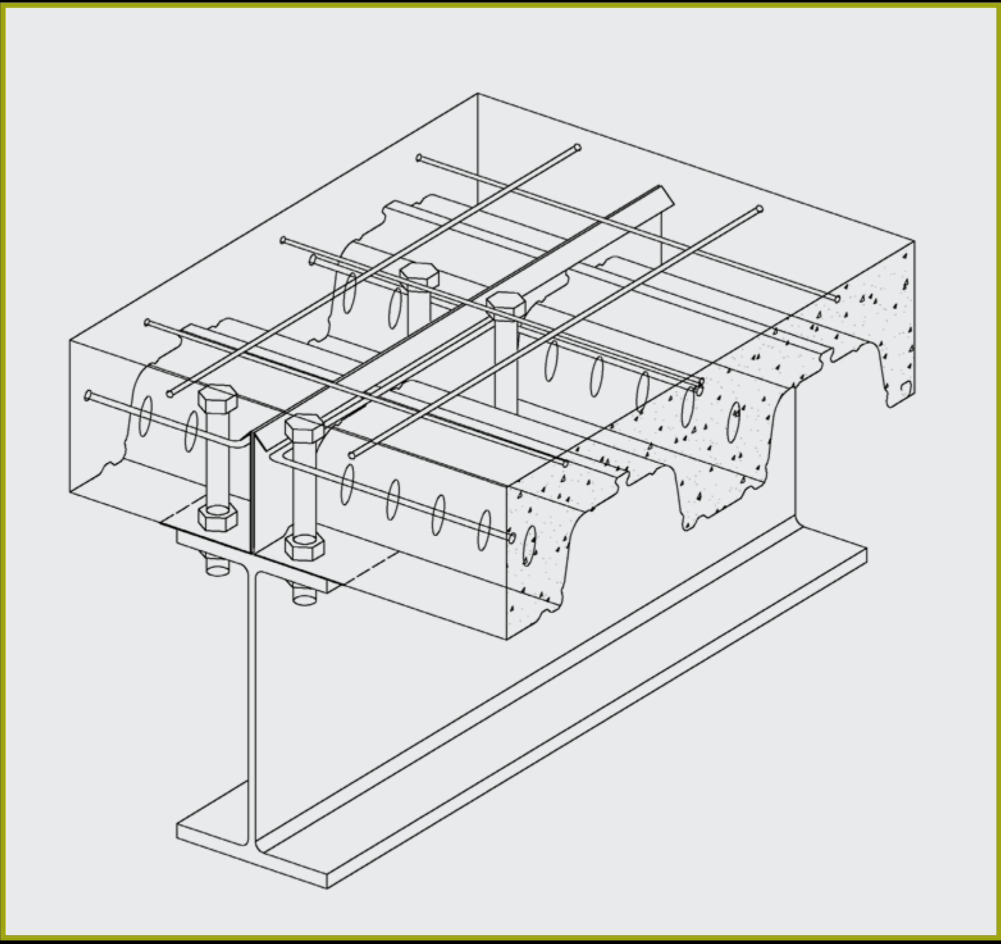
GUIDANCE ON DEMOUNTABLE COMPOSITE CONSTRUCTION SYSTEMS FOR UK PRACTICE



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GUIDANCE ON DEMOUNTABLE COMPOSITE CONSTRUCTION SYSTEMS FOR UK PRACTICE

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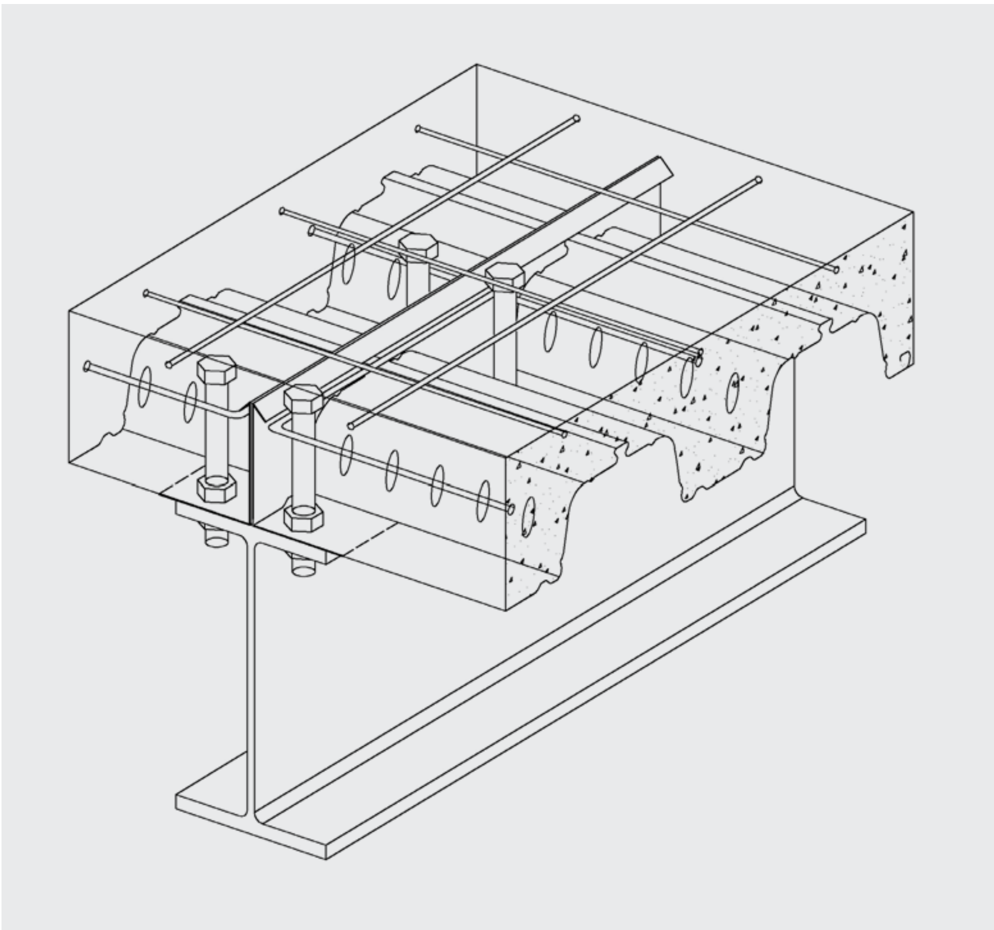


FOREWORD

A circular economy model based on the hierarchy of *reduce*, *reuse* and *recycle* is increasingly important in construction, which is the most resource-intensive sector in the UK. Composite construction is very efficient structurally but is difficult to deconstruct and hence to reuse. This guide presents suitable shear connection technologies and design data by which steel beams and potentially floor slabs may be reused in composite construction. In this way, the benefits of composite construction in the first and subsequent cycles of use are retained.

In this design guide, an approach for designing reusable steel-concrete composite structures is presented based on the results of an EU-funded project called *REuse and Demountability Using steel structures and the Circular Economy, REDUCE*. The *REDUCE* partners were: SCI (coordinator), University of Bradford, University of Luxembourg, Technical University of Delft, Bouwen Met Staal, Tata Steel (NL), Astron and AEC3.

This guide was written by Ana M. Girão Coelho of BCSA (formerly SCI), and Mark Lawson of the SCI and Professor of Construction Systems at the University of Surrey, with additional contributions from Dennis Lam and Jie Yang from the University of Bradford (Section 2), and Christoph Odenbreit and Andras Kozma from the University of Luxembourg (Sections 4 and 5).

CONTENTS

SUMMARY

This design guide presents a design procedure and worked examples for composite beams using demountable shear connectors that is based on the principles of Eurocode 4 (BS EN 1994-1-1). The design methodology takes account of the different characteristics of the demountable shear connectors, in terms of their shear resistance, stiffness, and ductility. Design data on the performance of two types of demountable shear connector, using high-strength structural bolts and coupler systems, are presented.

Two approaches are proposed for the design of demountable composite beams at the ultimate limit state, (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 the 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 the subsequent cycles of use. In this respect, the end slip is controlled to a value that ensures that the shear connectors will not experience plastic deformations. New formulae are presented for the effective stiffness, elastic section modulus, and end slip of composite beams based on the shear connector stiffness and equivalent spacing.

Worked examples are presented for a 12 m span I-section beam and a 15 m span cellular beam using the proposed design methodology.

NOTATION

Lower case

a	Length
b	Width
b_0	Average rib width (or minimum width for re-entrant profile)
d	Diameter
d_s	Effective diameter of bolt
d_w	Clear depth of the steel web
f	Natural frequency of floor
f_c	Value of the cylinder compressive strength of concrete at 28 days
f_u	Tensile strength of steel
f_y	Yield strength of steel
g	Permanent loads
h	Height, depth
k	Stiffness of shear connector
k_{flex}	Reduction factor for resistance of non-ductile shear connectors
k_l	Reduction factor for resistance of a shear connector used with profiled steel sheeting parallel to the beam
k_t	Reduction factor for resistance of a shear connector used with profiled steel sheeting transverse to the beam
n	Modular ratio
n_0	Modular ratio for short-term loading
n_L	Modular ratio for long-term loading
n_r	Number of shear connectors placed in one rib
n_{sc}	Number of shear connectors to point of maximum moment

q	Variable loads
r	Root radius of a rolled section
s	Slip, longitudinal spacing
$\bar{s}$	End slip
t	Thickness
z	Depth of the neutral axis

Upper case

A	Cross-sectional area
A_{s}	Tensile stress area
E_{a}	Young's modulus of steel
E_{cm}	Secant modulus of elasticity of concrete
F	Force
I	Second moment of area
L	Span of beam
L_{e}	Equivalent span of beam
M	Bending moment
N	Axial load
P_{R}	Shear resistance of the shear connectors
R	Ductility
S_{k}	Parameter used for elastic section modulus
V	Shear force
W	Section modulus

Greek letters and symbols

α	Factor for shear connector resitance
χ	Reduction factor for buckling
δ	Deflection of beam
δ_{i}	Imposed load deflections
δ_{t}	Total load deflections

δ_{el}	Slip determined from push tests at a load of $0.7P_{Rk}$
δ_{sw}	Deflection due to the self-weight and superimposed loads plus a nominal 10% imposed load
δ_u	Maximum slip
$\emptyset$	Diameter of a bar
ε	Strain
η	Degree of shear connection
φ_t	Creep coefficient
γ_{M2}	Partial factor for resistance of cross-sections in tension to fracture
γ_v	Partial factor for design shear resistance of a welded stud
$\bar{\lambda}$	Non-dimensional slenderness
ψ_L	Creep multiplier

Subscripts

a	Structural steel section
b	Bolt
c	Concrete slab, compression
comp	Composite
d	Design value or clear depth of the steel web
Ed	Factored applied load or internal force
eff	Effective
el	Elastic
eq	Equivalent
exp	Experimental value
f	Flange
k	Characteristic value
m	Mean value
max	Maximum
min	Lowest value
o	Opening

p	Profiled steel decking
pl	Plastic
Rd	Design resistance value
red	Reduced
s	Slab
sc	Shear connector
serv	Service conditions, unfactored load or internal action
sw	Self-weight loads
T	Tee section (cellular beam)
U	Unpropped construction
w	Web
wp	Web-post

Abbreviations

NA	National Annex
SLS	Serviceability Limit State(s)
UF	Utilisation Factor
ULS	Ultimate Limit State(s)

Axes

x	Longitudinal axis along the member
y	Major axis (parallel to flanges)
z	Minor axis (parallel to web)

A 11.2m span composite cellular beam was constructed and tested at the University of Bradford as part of the RFCS project REDUCE. The cellular beam used bolted shear connectors that were placed either side of a pair of partial depth edge trims (Detail B in Fig 3.2) with mesh reinforcement placed over them.

INTRODUCTION

1.1 Background

Composite construction is used in many types of buildings, such as multi-storey office buildings, hospitals and schools, car parks, and heavily loaded floors in industrial buildings. Its main application is long-span construction for spans of 12 to 20 m where the steel weight is 30 to 50% less than in non-composite construction, and where integration of services ducts and pipes can be made through large web openings. The reduction in steel weight is important from both economic and embodied carbon viewpoints and so the use of composite construction has become dominant in many building sectors in the UK.

Composite beams are generally used in combination with composite floor slabs using 50 to 80 mm deep steel decking, in which the 3 to 4.5 m spacing between the beams is determined by the spanning capabilities of the decking in the construction stage. The slab depth is typically 130 to 150 mm, and the spacing of the deck ribs is 150 mm for re-entrant profiles to 300 mm for trapezoidal profiles. The self-weight of a composite beam and slab system is approximately 300 kg/m^2 floor area, which is less than 40% of the weight of an equivalent reinforced concrete structure.

The commonly used shear connection system between the steel beams and concrete slab is in the form of 19 mm diameter welded stud shear connectors that may be through-deck welded on site, or welded in the fabrication shop and placed in pre-punched holes in the decking. The number of connectors depends on the required degree of shear connection as a function of the beam span.

There are many types of buildings which might have a relatively short life span in their first cycle of use, but the client would wish to retain the economic and sustainability benefits of composite construction 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 types of shear connector systems that can be disconnected from the beams. There are also possibilities to use these demountable shear connectors as an alternative to welded shear connectors in the following practical cases:

- Welding on-site is not permitted for various reasons,
- The beams are fully painted or galvanised so that site welding would require removal of this layer,
- Small projects or building extensions where the on-site cost of machinery for through-deck welding would be otherwise high,
- In projects where on-site welding may be difficult, or where weather conditions make site welding less reliable.

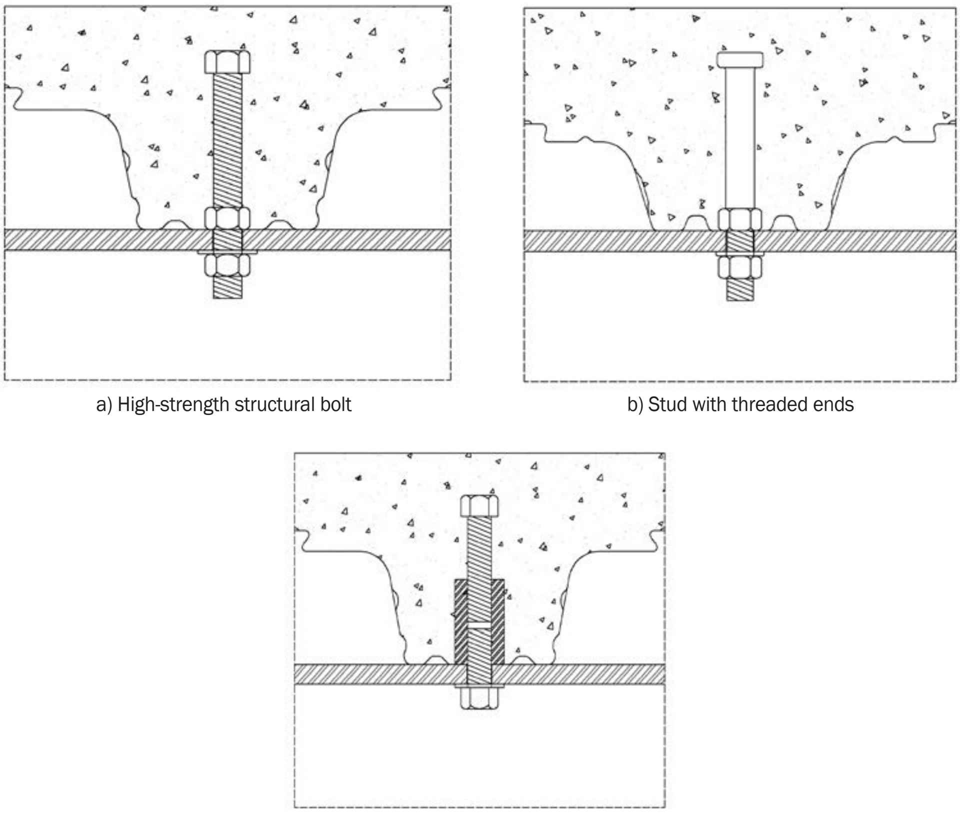
The different forms of shear connection systems that may be used in combination with composite floor slabs, are described as follows:

- 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 Fig. 1.1a,
- Stud shear connectors with threaded ends and with nuts above/below the beam flange, see Fig. 1.1b.

Other systems may be used for precast concrete slabs, as follows:

- Friction grip bolts placed in a tightly-fitting cylinder with a bolt below the flange and tightened from above, see Fig. 1.2a,
- Bolts placed from below the beam flange that are connected to couplers embedded in the slab, see Fig. 1.1c and Fig. 1.2b.

While there is 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. A relatively small additional initial construction cost is generally required to facilitate design for deconstruction but, whole life costing over two or more building cycles, demonstrates 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.

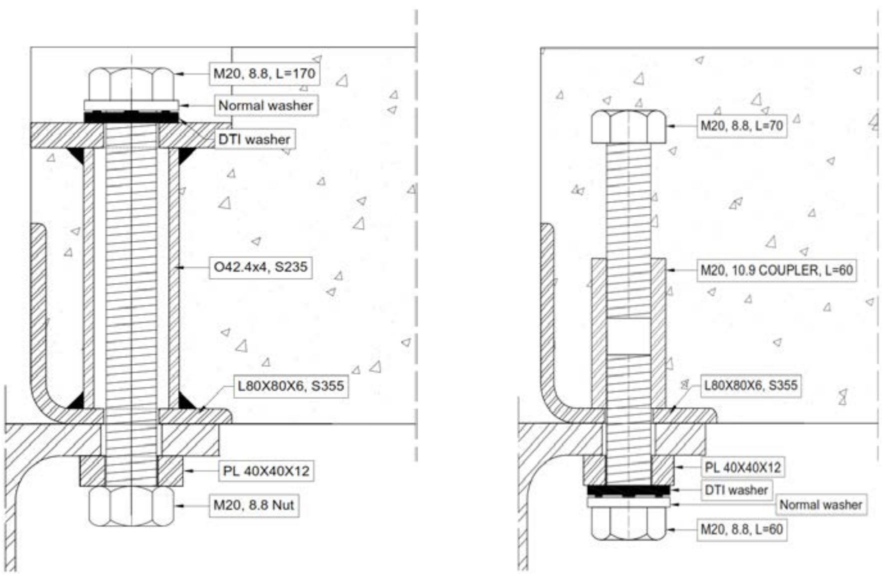


a) High-strength structural bolt

b) Stud with threaded ends

c) Coupler system

Fig. 1.1 – Alternative demountable shear connection systems for cast in-situ concrete in profiled slabs with ribs perpendicular to the beam



a) Friction grip bolts

b) Coupler system

Fig. 1.2 – Alternative demountable shear connection systems for pre-cast floors

1.2 General design considerations

This publication provides design guidance and criteria that can be used as a guide to the improvement of existing Eurocode-based procedures for the design of demountable composite steel and concrete beams.

The key design parameters that are required for composite design using demountable shear connectors are the:

- Characteristic shear resistance of the connectors, P_{Rk}, which is divided by a partial factor of 1.25 to obtain the design resistance, P_{Rd},
- Deformation (slip) capacity at the characteristic shear resistance, which should exceed 6 mm when using the current partial shear connection rules in BS EN 1994-1-1 [1],
- Stiffness of the shear connectors, k_{sc}, defined as a linear value to a load of $0.7P_{Rk}$, which is required for, e.g. serviceability calculations of deflections,
- Further reduction factor k_{flex} to determine the effective resistance in the case of non-ductile shear connectors (see definition in Section 1.5), which takes the value of 0.8 for uniform spacing and 0.85 for a non-uniform spacing where more shear connectors are placed towards to the ends of the beam.

Demountable shear connectors generally satisfy the requirements for plastic design of composite beams (with some modifications) but their stiffness is lower than that of welded stud shear connectors. Also, to minimise the number of shear connectors that are placed in the first cycle and which therefore have to be untightened and re-installed in the subsequent cycles of use, the minimum degree of shear connection requirement in BS EN 1994-1-1 is replaced with a limiting slip criterion at the Serviceability Limit State (SLS) in the design of demountable composite beams.

The design of simply supported long span composite beams is typically governed by serviceability criteria and therefore requires less composite action to achieve their bending resistance. The use of low degrees of shear connection and longer spans result in additional deformation demands on the shear connectors. Also, the flexibility of the shear connectors adds to the in-service deflections of the composite beam. It follows that the shear connector arrangement should be optimised to satisfy deflection criteria, as demonstrated in this guide.

A new design formula is presented for the composite stiffness that takes account of the flexibility of the shear connectors, which includes an equivalent uniform spacing for the shear connector arrangement. This same formula may also be used to predict the additional deflection when using welded stud shear connectors, albeit with a different stiffness.

For the second cycle of use, the same principles apply, but it is necessary to take account of practical aspects of the demounting and reconstruction process. Most composite beam design is based on unpropped construction for maximum construction efficiency. The requirements for composite stiffness using demountable shear connectors mean that it is important that the steel beam is also sufficiently stiff in the construction stage. Furthermore, for the greatest potential of reuse, the steel beam should be selected to allow it to be used as unpropped in the second cycle and so this guide is devoted to unpropped construction.

The conditions of the second use cycle are not necessarily predictable unless the structure is designed for a short design life. Therefore, some general design requirements are proposed to maximise the potential for reuse, as follows:

- The beam span and column spacing are based on a preferred planning grid of 1.5 m. The secondary beams are designed as composite and span the longer distance in an orthogonal grid. Sensible beam spans would be 12 m, 15 m and 18 m to maximise efficiency when designed with demountable shear connectors,
- The primary beams span 6 m, 7.5 m or 9 m and are designed as non-composite, so that the slab segments can be disconnected and connected easily in the second cycle of use. The primary steel beams are subject to point loads and may be slightly heavier than equivalent composite beams. The short span primary beams account for less than 25% of the overall steel weight so an increase in their size may amount to 3 to 5% in overall steel weight,
- The design imposed load is taken as 5 kN/m^2, which allows for a wide range of first and second cycles of use and it includes an allowance for superimposed dead loads and partitions and any additional loads in the second cycle. If the self-weight of the beam and slab is approximately 3 kN/m^2, the factored load is therefore 11.5 kN/m^2,
- The beams are designed as unpropped in construction. This means that the practical range of beam span to depth ratio is 22 to 26,
- The slab depth is taken as 150 mm using 80 mm deep decking for 3.75 m span and 130 mm using 60 mm deep decking for 3 m span,
- Holes are pre-drilled in pairs in the top flange of the beams at 300 mm longitudinal spacing and 100 mm transverse spacing. These holes should be close tolerance with a diameter of 21 mm for M20 bolts.

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 mesh 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.

Demountable construction systems may be considered for the types of buildings noted earlier, but ease of deconstruction and reuse may also be considered as a potentially important future requirement in other types of buildings. However, it is recognised that the designer should consider the requirements of design for deconstruction in the initial building concept which may extend to standardisation of floor grids, section sizes and details to facilitate ease of demounting and reuse, and avoidance of damage in this process.

The first cycle of use should follow the general principles of design of composite beams. Using the design procedure in this guide, it is necessary to prevent permanent deformation that may affect the subsequent cycles of use. The cost of any special features that are introduced to facilitate demounting and reuse should be minimised, when using the technologies presented in this guide.

1.3 Relevant code specifications

This guide is prepared in general structural engineering terms and refers to rules and principles given in the following standards:

- BS EN 1090-2:2018 [2], which sets all the technical requirements that should be taken into account for the execution of structural steelwork,
- BS EN 1990:2002+A1:2005 [3], which describes the principles and requirements for safety, serviceability and durability of structures, the basis for their design and verification and gives guidelines for related aspects of structural reliability,
- BS EN 1991-1-1:2002 [4], which gives best-practice design guidelines and actions for the structural design of buildings and civil engineering works,
- BS EN 1991-1-3:2002+A1:2015 [5], which provides guidance on determining the snow load to be used for the structural design of buildings,
- BS EN 1991-1-4:2002+A1:2010 [6], which is the European standard for wind actions on structures, and is used with the UK National Annex,
- BS EN 1992-1-1:2004+A1:2014 [7], which gives general rules for the design of concrete structures,
- BS EN 1993-1-1:2005+A1:2014 [8], which gives general rules for the design of steel structures,
- BS EN 1993-1-8:2005 [9], which gives rules for the design of steel joints,
- BS EN 1994-1-1:2004+A1:2014 [1], which describes the principles and requirements for safety, serviceability and durability of composite steel and concrete structures, together with specific provisions for buildings.

1.4 General notes on the guide

This publication discusses the design and technical issues related to the design and construction of demountable composite beams and floors in the first cycle of use with some information on the requirements for reuse. It focuses on conventional in-situ composite slab systems using steel decking for the first cycle of use, which follows the same design and construction principles as for composite construction with some modifications to facilitate reuse.

For the second cycle of use, the slab segments may be cut, separated and labelled, stored and reused as essentially precast concrete units. Alternatively, the slab may be discarded and only the steel structure reused.

Assessment of the environmental benefits of reusing composite construction [10] has shown that a greater benefit is achieved by reusing the floor slabs than by reusing just the steel beams. This is because the weight of the floor slabs is much greater than the beams and because the floor slabs are currently crushed and generally downcycled whereas the steel beams are already highly recycled at the end-of-life of buildings.

These guidelines deal with the design of demountable composite beam systems, together with any special features of their design, installation and reuse.

Section 2 presents the basic principles of demountable composite construction and recommended detailing and fabrication practices. Section 3 contains a review of the shear connection rules in BS EN 1994-1-1 and proposes a procedure for idealisation of the load-slip behaviour of flexible non-ductile shear connectors. The design checks for Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are presented, respectively, in Sections 4 and 5 and are formulated to provide economic designs with potential for reuse. Section 6 presents fully worked design examples for a 12 m span composite beam and for a 15 m span composite cellular beam.

1.5 Terms and definitions

For the purposes of this guide, the following terms and definitions apply, with specific reference to composite construction.

Deconstruction (or disassembly, or demounting)	Deconstruction is the process of taking a building apart into its component parts in such a way that they can be reused; it minimises the destructive and downcycling aspects of demolition and preserves the components and materials without generating waste
Deformation capacity (or slip capacity) of a connector	Maximum deformation that a shear connector can reach in a push test [1]

Design for Deconstruction	Designing for deconstruction involves consideration, at the design stage, of how a building can be demounted and potentially reused
Demounting or deconstruction	The ability to dismantle a structure safely and with minimum disruption and to salvage its components
Design life	The notional period for which the structure is designed to be used for its intended purpose with maintenance but without major repair being necessary; structures are often extended or modified over their design lives
Ductile shear connector	Shear connector that has sufficient deformation capacity to assume ideal plastic behaviour of the shear connection
End slip (composite beam)	Maximum slip reached by the end shear connectors
First cycle of use	The first use of the structure for which it is designed
Flexible connector	Shear connector that allows a certain amount of slip at the steel flange/concrete interface
In-situ reuse	The structure is reused in-situ, i.e. without being deconstructed; this can also be referred to as building renovation in which the structure is retained
Non-ductile shear connector	Shear connection that does not exhibit an ideal plastic plateau
Relocated reuse	Reuse of the structure on a new site or in a new location on the same site
Component reuse	Reuse of individual structural elements, e.g. a beam or a column member
Structure reuse	Reuse of the whole structure or sub-element(s) of a structure, e.g. a truss
Reuse	Use of old components with little or no reprocessing, largely in their original form; they may be reused for the original function (a conventional reuse scenario), or re-purposed
Second cycle of use	Second use of the demounted and re-assembled structure, or its components

Demountable, composite cellular beam test at the University of Bradford.

The bolted shear connectors were placed in pairs at 300mm spacing in the outer 1.2m of the span and singly in a staggered pattern either side of the edge trim in the rest of the beam span.

PRINCIPLES OF DEMOUNTABLE CONSTRUCTION

2.1 General requirements

Composite beams are widely used in multi-storey construction for their economy and function; this includes office and residential buildings, retail buildings and car park structures.

Demountable composite construction systems and their shear connectors should perform structurally in a similar manner to conventional composite beams, but the shear connectors should also be able to be disconnected from the beams in order that the beams, and potentially the floor slab, can be reused. The required performance characteristics of the shear connection system in demountable construction are:

1. Structural performance.
    a. The shear resistance of the shear connector should be similar to a welded stud of the same size (19 mm shank diameter × 120 mm length is taken as the standard size of a welded stud). The shear resistance is based on push tests and so is also dependent on the shape and height of the deck profile. Demountable shear connectors are placed in holes drilled in the top flange and therefore the holes are detailed in pairs either side of the web (at nominally 100 mm spacing).
    b. Shear stiffness influences the deflection of composite beams and it leads to end slip. Demountable shear connectors are more flexible than welded studs due to bolt slip and therefore the flexibility of the shear connectors should be taken into account in the design methodology.
    c. Deformation capacity is required so that the shear connectors can develop their shear resistance as a group in a beam. For *ductile* shear connectors, this is defined as a deformation of 6 mm at a load corresponding to the characteristic resistance [1].
    d. Loading: beams are essentially subjected to uniform loading, and concentrated loads are not significant, i.e. lower than 10% of the total load applied to the beam, so that the point of maximum moment is at or close to mid-span.
2. Installation. The installation process for the demountable shear connectors should be as efficient as is practical. As a starting point in the design process, the number of demountable shear connectors should ideally not exceed that required for welded shear connectors.

3. Demountability. The shear connectors should be accessible for safe unbolting. It may be necessary to saw along pre-defined lines to separate the slab elements from the beams. Deconstruction should ideally involve a small number of reasonably large components subject to limits on lifting.
4. Corrosion resistance. Where parts of the shear connectors are exposed, they should be corrosion protected, particularly in car parks and similar semi-exposed applications. In such cases, the bolts should be zinc coated to 600 g/m^2.
5. Aesthetics and damage over time. Where the top of the floor slab is covered by a raised access floor or by boards and battens, then the visual aspects of the demountable system are not so important. In industrial and car park applications, where the floor slab is exposed, then any components that are exposed should be protected where they may be subject to damage.
6. Safety. The procedure for demounting and reuse should be clearly explained and retained in the Building Information Modelling (BIM) system. Ideally, the lifting points for the components should be accessible from the top of the slab and the components should be of a size for ease of lifting and transportation.
7. Economy. The initial cost of the demountable structure should not be significantly more than conventional construction for the same application. Whole life costing including the costs of demounting, transport and re-assembly, etc., should be undertaken to compare a traditional construction against demountable options over a minimum of two building life cycles. The components should be robust so that they are not damaged during this process.
8. Adaptability. In addition to the potential for future reuse, the structure should be adaptable over time to different uses and internal layouts. Therefore, it is proposed that longer spans are used (typically 12 to 18 m) and the minimum imposed load is taken as 5 kN/m^2, which includes partitions and services, in order to be able to meet future uses for the space.
9. Standardisation. To facilitate the second cycle of use, it is proposed that a sensible level of standardisation is introduced into the design of the composite structure. For example, the beam span and spacing should be based on a standard floor grid. The floor arrangement should ideally be rectangular with columns on a regular grid and the longer spanning secondary beams and edge beams should be of the same size to facilitate their reuse.
10. Environmental impact. The environmental impact is calculated in accordance with the Life Cycle Assessment standards developed under CEN TC/350. This should include all life cycle phases or modules in particular, the impacts of demolition/deconstruction (Module C) and the potential future benefits arising from recycling and reuse (Module D) to reflect the benefits of design for deconstruction and reuse.

2.1.1 Materials and structural components

BS EN 1994-1-1 lists the materials that are structurally suitable for use in composite designs, which are relevant also to demountable composite construction. Based on cost, availability, and test data [10], the materials shown in Table 2.1 are recommended for composite beam design for reuse using demountable bolted shear connectors.

The structural system covered by this Guide is on-site concrete placed on steel decking (known as a composite slab) acting compositely with rolled or fabricated beams, as this is the most commonly used form of composite construction in the UK. The top flange width should be a minimum of 165 mm (therefore 305×165 UB or IPE 360 sections are the smallest recommended rolled sections) to comply with minimum bolt transverse spacing (gauge) and edge distance requirements. Other options include cellular beams or beams with large web openings conforming to the geometric limits in SCI-P355 [11]. The most common steel grade is S355.

Table 2.1 – Material properties for use of this guide

Component		Grade	Strength
Beam	IPE or UB (b_{min} = 165 mm)	S355	f_y = 355 N/mm^2
Shear connector	Bolt M20 (20 mm diameter)	8.8	f_u = 800 N/mm^2
Slab	In-situ concrete	C30/37	f_{ck} = 30 N/mm^2

Composite slabs are commonly manufactured with concrete strength class C25/30 or C30/37, using trapezoidal decking of 60 or 80 mm height and rib spacing of 300 mm, or re-entrant decking of up to 55 mm height and rib spacing of 150 mm and with a steel thickness of not less than 0.9 mm. The minimum slab topping over the decking is 70 mm. Therefore, the slab depth is typically 130 to 150 mm for 60/90 minute fire resistance. Slabs may be thicker for longer fire resistance periods.

To facilitate demounting and reuse of the composite slab, pairs of cold formed steel edge trim are placed along the centreline of the secondary beams to provide a pre-determined cut-line in the slab to facilitate deconstruction. These edge trims are similar to those used around the edge of the slab. Two forms of construction are proposed, see also Section 2.4: (a) full depth trims: the edge trims are equal to the slab depth and so form a discontinuity, and (b) partial depth trims: the edge trims are shallower than the slab depth so that mesh reinforcement can be placed over it 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 centreline of the beam to expose the edge trim and to be able to separate and reuse the slab segments. It was found from tests [10] 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.

Demountable shear connectors are typically 20 mm diameter grade 8.8 bolts, and have grade 8.8 nuts above and below the flange. The bolts are placed in close tolerance punched or drilled holes of 21 mm diameter in the beam top flange to minimise initial slip. The minimum transverse spacing of the holes is 100 mm to allow the shear connectors to be placed on either side of the beam centreline and 'staggered' along the beam. The bolts should have a minimum projecting height of 40 mm over the top of the decking, and U-bars of 10 mm diameter and 500 mm (= 50Ø) minimum length should be placed around pairs of shear connectors on each side of the edge trim to reinforce the slab locally and to allow it to act as transverse reinforcement in this pseudo-edge beam configuration, see Fig. 2.1.

Fig. 2.1 – Partial depth edge trims and U-bar details

2.1.2 Actions

Clause 6 of BS EN 1991-1-1 defines imposed loads on buildings as those which arise from occupancy, e.g. normal use by people, furniture, vehicles. Table NA.3 of the National Annex (NA) to BS EN 1991-1-1 specifies the characteristic imposed uniformly distributed load on floors for category B (office buildings) and category C1 (typically schools) of 3 kN/m^2. This value is greater than the prescribed minimum occupancy load for car parks, see Table NA.6 of the NA to BS EN 1991-1-1 for category F.

In designing for the second life, the self-weight of an additional concrete layer should also be considered. In order to cover the widest possible range of current and future applications, it is recommended to design for a minimum occupancy load of 3 kN/m^2 and 2 kN/m^2 for partitions, super-imposed dead loads and any additional self-weight that equals a total imposed load of 5 kN/m^2.

For the second cycle of use of the composite slab, it may be necessary to place an additional 50 to 70 mm layer of concrete with its mesh reinforcement over the slab segments to provide for diaphragm action. This additional load of 1.2 to 1.7 kN/m^2 should be included in the future assessment of the composite beam design in the second cycle of use and may be partly accommodated within the proposed 5 kN/m^2 initial design for imposed loads and superimposed dead loads. The additional 50 to 70 mm depth of the composite section, if acting effectively with the concrete segments, may add 10% to the beam stiffness and bending resistance which partly compensates for its additional load.

2.2 Grids and standardisation

The key design parameters for standardisation of the primary structure are the:

- Floor grid,
- Clear floor spans,

- Building size and regularity of shape,
- Floor-to-floor height,
- Integration of building services,
- Circulation and access space.

It is proposed that the structural grid of buildings for dimensional planning should be based on multiples of 1.5 m. This means that the optimum floor grid might be 15 m × 7.5 m (and with a 3.75 m floor span).

The most efficient use of a steel or composite beam occurs when the bending resistance and the critical serviceability criteria are close to their limiting values. This is expressed in terms of an optimum span to depth ratio for which the minimum weight solution is obtained.

The limiting span to depth ratio is also dependent on the design strength of the steel, the loading pattern, and the utilisation of the beam at the ULS, the Utilisation Factor (UF). It is recommended that UF ≤ 0.8 at the ULS is adopted, in plastic design, as a standard when combined with a design imposed load of 5 kN/m^2, which also takes into account permanent superimposed loads that are generally small. Therefore, the design of demountable composite beams should facilitate the maximum flexibility in current and future (re)use and adaptation.

Regarding the potential reuse of steel beams, the wide range of possible spans and beam sizes could be reduced considerably if designers work to a narrow range of span to depth ratios. For beams subject to uniform loading, this should be in the range of 18 to 20 for bare steel beams and 22 to 26 for composite beams. It follows that a 457×191 UB beam would span 8 to 9 m as a steel section and 10 to 12 m as a composite beam.

In a rectangular floor grid, it is proposed that the ratio between the spans in the two directions should be 2:1 unless other geometric criteria control. The long span secondary beams should be designed as composite, but the shorter span primary beams should be designed as non-composite to facilitate the demounting and reuse of the floor slabs. This layout means that the primary beams support one or possibly two incoming secondary beams and ideally their connections should be such that all the secondary beams are the same size and length. It is also proposed that edge beams should be the same size as internal beams to facilitate their reuse as internal beams in the second cycle of use.

Cellular beams have multiple circular openings along their length and may be manufactured from two rolled sections or from three steel plates. They are designed and manufactured for the particular loading and opening configurations and they are often asymmetric in shape. It is proposed that to facilitate reuse, they are designed as symmetric using the same rolled profile and that their span to depth ratio is 22 to 26 as for composite beams. The last cell at the ends of the beam should be infilled to facilitate some modification at the connections in the second cycle of use.

2.3 Possible reuse scenarios

The various possibilities for the potential reuse of composite beams and composite floor slabs depends on the economic and environmental requirements, and the practicalities of demounting, storage and reuse of the components. These scenarios are discussed below and are shown schematically in Fig. 2.2:

1. Steel reuse (Type 1 construction). The beam can be disconnected from the slab so that the beams can be demounted and reused, but the floor slabs are not reused. The beams are then available to be reused as part of an inventory of reclaimed sections.
2. General reuse (Type 2a construction). The beams can be disconnected from the slab and the slab elements can be demounted in addition to the beams, so that both can be potentially reused.
3. Specific reuse (Type 2b construction). The beams and slab can be demounted and re-assembled in another location but in the same configuration and geometrical arrangement as the first use.

The reuse of the floor slab has not traditionally been seen as part of the circular economy because of the difficulty in cutting and removing the slab segments without damage and also their size and weight for transport, storage and re-assembly. However, the environmental impact of not reusing the floor slabs is sufficient to provide motivation to consider the ways in which the floor slabs, in addition to the steel beams, may be reused.

The reuse of composite floor slabs is only a practical reality if the slab segments are standardised in the following ways:

- Slab span of 3 m or 3.75 m to be consistent with the recommended floor grid,
- Segment width of typically 1.5 m to 2.4 m to be able to be transported without restrictions, and comply with re-assembly tolerances,
- Slab depth of 130 mm for 2.7 to 3 m spans, and of 150 mm for 3.75 to 4 m spans,
- Cut-line along the centre of the beam and protection of the ends of the slab by a cold formed steel edge trim or steel angle to prevent damage.

A small number of variants permit the storage, handling and reuse of composite slab segments, and potentially a market for reuse of composite slabs may develop. The weight of a standard slab segment is 2 to 3 tonnes, and so 6 to 10 slab segments may be transported on one lorry.

A practical case of reusing composite slabs is when a building or a major part of it, is to be demounted, moved and re-assembled. Then the slab segments should be numbered as they are cut and salvaged so that they can be re-installed in the same configuration on the same beams.

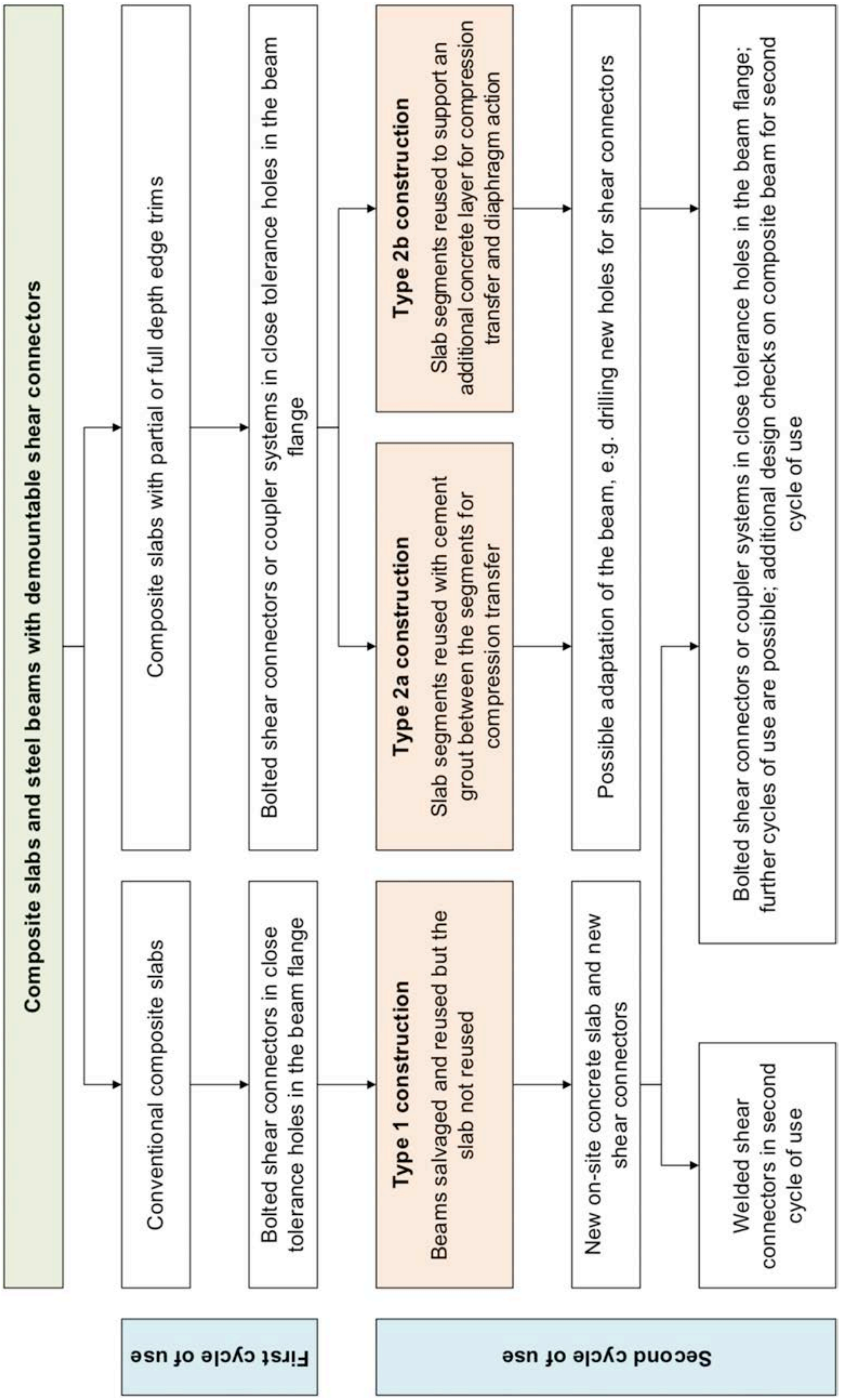


Fig. 2.2 – Composite beams and composite slabs: possible reuse scenarios

The cutting of the sab segments results in a loss of about 6 mm in their width which allows for some geometric deviation in position of the segments along the beam. The joint should be filled with non-shrinkable cement grout to provide for compression transfer. Where a high level of diaphragm action is required, a lightly reinforced concrete layer of 50 mm minimum thickness may be placed on the slabs but in that case, further reuse of the slabs is not practical.

Where holes have to be enlarged in the second cycle of use of the salvaged slabs, resin may be injected into the enlarged bolt holes depending on the alignment of existing holes and shear connectors in the slab segments. Epoxy resin injection has the advantage of increasing the shear connector stiffness and may be used in a small number of shear connectors at the ends of the span to have the maximum effect.

2.3.1 General requirements for the second cycle of use

The requirements for the second cycle of use of the floor may be summarised as follows:

- The beams should have a minimum width of 165 mm and should have holes for shear connectors in pairs at 100 mm transverse spacing and 300 mm longitudinal spacing which can be used to place the demountable shear connectors in the desired pattern for the commonly used deck profiles with 150 or 300 mm rib spacing. These holes can also be drilled or punched in the second cycle of use. The beam can also be reversed so that the bottom flange is available if the existing hole pattern is different from the desired pattern in the second cycle of use. For this reason, symmetric steel sections are recommended to facilitate reuse.
- The slab segments may be cut along the axis of the beams at the lines of the installed pairs of edge trims and transversely through the slab topping so that they can be reused either by grouting between the slab segments or by placing a thin additional concrete layer with mesh reinforcement. Depending on the quality of the slab, it may be necessary to remove any damaged or deteriorated concrete from the surface by scabbling before placing the new concrete layer.
- The slab segments may be used in the same arrangement if the building is moved and reused in its entirety or in large parts. This requires that the segments are numbered for relocation. It may not be practical to reuse the slabs in other applications unless the segments are highly standardised in dimensions and shear connector pattern.
- If the steel strength from the original design information is unknown, its yield strength should be determined by representative coupon tests that may be taken from low stressed areas (generally from the flange at the end of the beam). The mean yield strength of structural steel will generally be 10 to 15% higher than its design value and so it should be expected that measured values would be of similar magnitude. Guidance is given in the SCI publication P427 [12].
- It is proposed that the shorter span primary beams are not designed compositely and so they do not have demountable shear connectors, unless required for diaphragm action.

- It is considered that an imposed load of 5 kN/m^2 permits a sufficiently wide range of application in the first and second uses. In practice, most designs are controlled by the SLS limits for which the efficient use of demountable shear connectors is an important design requirement.
- For cellular beams, the same principles apply. Although cellular beams are often designed with different top and bottom chords, it is proposed that they are designed as symmetric in section to allow the beams to be reused more readily by the reversing the cellular beam if required.
- The beam end connections can be modified in the second cycle, or alternatively, adaptable connections such as the block connector, see [10], can be used to allow for small variations in member lengths.

2.3.2 Design of Type 1 construction in second cycle of use

The reclaimed steel beams may be designed compositely, either by using bolted shear connectors placed through the same drilled holes or by using welded shear connectors. The demountable shear connectors should have the same performance characteristics as the first cycle of use, so that multiple future cycles of use may be considered. Welded stud shear connectors may be used in the second cycle but no further reuse cycles would be possible.

2.3.3 Design of Type 2a construction in second cycle of use

The reclaimed steel beams and slab segments should be reused in the same configuration as in the original construction. The bolted shear connectors should have the same performance characteristics as in the original design provided they have not deteriorated by corrosion over time. The main requirement in Type 2a construction is to provide effective compression resistance through the slab segments by use of non-shrinkable cement grout.

The joint width is equal to the width of the saw cut in demounting (typically 6 mm). This allows for some deviations (within tolerance) in assembly in the second use cycle. The underside of the transverse joint is taped or sealed and the grout placed from above. The compression strength of the grout is typically equivalent to that of the parent concrete. Therefore, the load bearing capacity and stiffness of the composite beam is maintained for multiple cycles of use.

2.3.4 Design of Type 2b construction in second cycle of use

The slab segments are cut and retained as in Type 2a, but in the second cycle of use, they act as permanent formwork for a 50 minimum thickness concrete slab with additional mesh reinforcement. The self-weight is therefore increased by 1.2 kN/m^2 and this load is applied to the composite beam in the second cycle of construction.

Conversely, the bending stiffness of the composite section is increased because of the deeper slab and so in-service deflections will be less. Type 2b construction therefore requires further design checks for the second cycle of use but there is normally sufficient reserve in the beam design to allow this to be satisfied. It is necessary to clean and roughen the top of the retained slab and this preparation is generally sufficient to develop sufficient shear-bond strength with the new concrete layer for beams with uniform loading although a check on the shear transfer to the new concrete is required. The risk of potential contaminants affecting the concrete quality over time may influence whether the concrete strength has deteriorated and whether any additional shear connection system is required.

2.4 Alternative systems for cast in-situ composite slabs

Clause 6.6.1.1(12) of BS EN 1994-1-1:2004 allows the use of shear connectors other than welded studs. In that case, the behaviour assumed in the design should be based on tests and supported by a conceptual model.

Alternative demountable shear connection systems that were developed and tested with in-situ composite slabs are:

1. High-strength structural bolt (single embedded nut), see Fig. 1.1a. The high-strength structural bolt systems have an embedded nut above the top flange that is required to achieve sufficient stiffness of the shear connectors. The external nut or bolt underneath the top flange is used to tighten the shear connector. These systems can be placed in continuous (Detail A), partially-continuous (Detail B) or separable composite slabs (Detail C), see Fig. 2.3.
2. Coupler system (external bolt and single embedded long nut), see Fig. 1.1c, with the same slab details as in Fig. 2.3. The coupler system may also be used for precast slab segments, as shown in Fig.1.2 (although this scenario is not within the scope of this Guide).

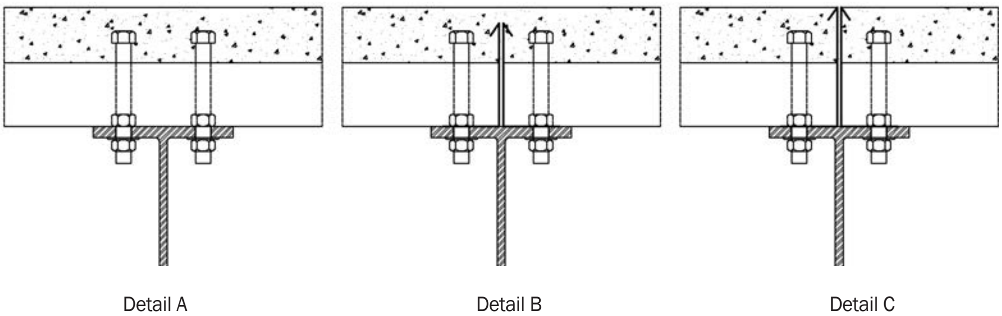


Fig. 2.3 – Demountable bolted shear connectors in Detail A; continuous, Detail B; partially-continuous (with edge trims) or Detail C; separable composite slabs (with edge trims)

Partially-continuous or separable composite slabs refer to their transverse spanning direction to the steel beam, see Fig. 2.3. Where partial-depth double edge trims are used between the pair of connectors along the longitudinal centreline of the beam, the composite slabs should have reinforcing mesh placed continuously over the edge trims. Where full-depth edge trims are used, the composite slab segments can be separated with only transverse cuts, but this system will reduce the diaphragm action of the floor slab unless the structure is braced regularly or is designed as a rigid frame. The edge trims are also found to play a beneficial role in reinforcing the composite slab locally to the shear connectors.

The use of double edge trims provides a cut-line to facilitate dismantling of composite beams and reuse of the composite slabs after their first cycle of use. A full-depth cut to a continuous slab along the centreline of the beam can be difficult to make accurately if both the beams and the composite slabs are intended to be reused. For the partial continuous slabs, a cut depth of approximately 30 mm will be needed along the centreline of the beam. No cut is needed for composite slabs with a full-depth edge trim although the edge trim will be exposed on the top surface and may be subject to some damage over time.

A bolt clearance of 1 mm can be used to place the bolted shear connectors in a reuse scenario where the slab segments are retained in Type 2 construction, see Fig. 2.2. This is shown to be adequate based on the practical dimensions of the retained slabs segments and from test evidence based on the measured stiffness of composite beams.

U-bars around the shear connectors should be provided in the systems according to standard push test results [10], see Section 3.2.

Advantages and disadvantages of the recommended demountable systems are summarised below for the Details A to C illustrated in Fig. 2.3:

- Bolted connectors Detail B: highest shear resistance, but an additional longitudinal cut in the slab topping at the beam centreline is required for reuse of the slabs.
- Bolted connectors Detail C: highest slip capacity, slab segments are cut only transversely to beam axis at the crest and are salvaged for reuse. This system may require protection to the upper surface of the slab, if subject to long term wear.
- Coupler system (Fig 1.1c): the external bolts are replaceable when the slab segments are retained, and the structural characteristics are similar to bolted shear connectors (although this system has only been tested with a full depth edge trim; Detail C).

2.5 Detailing issues using demountable shear connectors

The detailing requirements for demountable shear connectors may be summarised as:

- Hole spacing of 100 mm transverse to the axis of the beam,
- Minimum hole edge distance of 30 mm transverse to the load direction,
- Hole diameter of 1 mm more than the nominal bolt diameter so that slip is not excessive in service,

- The bolted shear connectors should use 20 or 24 mm nominal diameter grade 8.8 bolts and should have compatible nuts above and below the flange. The threaded length should be a minimum of the flange thickness plus 2 x diameters,
- The bolts should project a minimum of 120 mm above the beam flange for a 150 mm deep slab using 80 mm deep decking and 100 mm for a 130 mm deep slab using 60 mm deep decking,
- The edge trim height should be a minimum of 30 mm lower than the slab depth to allow the mesh reinforcement to be placed over it with a minimum cover of 15 mm. Full-depth edge trims may also be used but the push tests show that the demountable shear connectors are more flexible in this case, see [10]. The minimum thickness of the galvanised steel in the edge trim is 0.9 mm,
- The U-bars should be of 10 mm minimum diameter and should be placed around pairs of shear connectors, and below the head of the shear connectors. The width of the U bar should be 350 mm (rib spacing + 2.5d) and the length should be 500 mm (= 50Ø), minimum.

Push tests were carried out to determine the load-slip relationship for the bolted shear connectors

DEMOUNTABLE SHEAR CONNECTION SYSTEMS

3.1 Shear connection rules in BS EN 1994-1-1

3.1.1 Resistance

According to Clause 6.6.3 of BS EN 1994-1-1:2004, the design resistance of welded studs embedded in solid concrete should be obtained from the smaller of:

a) Shear failure of the stud:

$$P_{Rd} = \frac{0.8 f_u}{\gamma_v} \underbrace{\frac{\pi d^2}{4}}_{\text{area of the shank}} \tag{3.1}$$

and,

b) Failure of the concrete:

$$P_{Rd} = \frac{0.29 \alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_v} \tag{3.2}$$

with

$$\alpha = \begin{cases} 0.2\left(\frac{h_{sc}}{d} + 1\right) & \Leftarrow \quad 3 \leq \frac{h_{sc}}{d} \leq 4 \\ 1 & \Leftarrow \quad \frac{h_{sc}}{d} > 4 \end{cases} \tag{3.3}$$

where d is the diameter of the shank of the stud
h_{sc} is the nominal height of the stud
f_u is the ultimate tensile strength of the stud material
f_{ck} is the characteristic cylinder strength of the concrete
E_{cm} is the secant modulus of elasticity of the concrete
γ_v is the partial factor (= 1.25, according to the NA to BS EN 1994-1-1).

The efficiency of the shear connection between the concrete slab and the steel beam may be reduced as a result of (i) the use of trapezoidal sheeting, and (ii) the number of shear connectors placed in each rib, Clause 6.6.4.1 of BS EN 1994-1-1:2004 gives the following single reduction factor for the resistance of welded studs, singly or in pairs, in cases where the decking is orientated parallel to the beam:

$$k_l = 0.6 \frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1.0 \tag{3.4}$$

where b_0 is the average rib width (or minimum width for re-entrant profiles)
h_p is the profile height (to the shoulder of the profile)

Clause 6.6.4.2 of BS EN 1994-1-1:2004 combines both effects (i) and (ii) above to propose the following reduction factor for the resistance of shear connectors in decking with ribs transverse to the beam:

$$k_t = \frac{0.7}{\sqrt{n_r}}\frac{b_0}{h_p}\left(\frac{h_{sc}}{h_p} - 1\right) \le k_{t,max} \tag{3.5}$$

in which $k_{t,max}$ is given in Table 3.1. The reduction factors from Eqs. (3.4) and (3.5) are applied to both failure modes. However, the use of trapezoidal sheeting will only have an effect on the concrete failure mode.

Table 3.1 – Upper limits for the reduction factor k_t

Number of stud connectors per rib	Thickness of sheet (mm)	Welded studs (through profiled sheeting and $d \le 20$ mm)	Profiled sheeting with pre-drilled holes and studs with $d = 19$ mm or $d = 22$ mm
$n_r = 1$	≤ 1.0	0.85	0.75
	> 1.0	1.0	0.75
$n_r = 2$	≤ 1.0	0.7	0.6
	> 1.0	0.8	0.6

Note: Modified values are used in the UK for some situations, according to SCI P405 [14]

3.1.2 Stiffness of the shear connectors

The stiffness of the shear connector is defined in Clause A.3(3) of BS EN 1994-1-1:2004, see also Fig. 3.1, as follows:

$$k_{sc} = \frac{0.7P_{Rk}}{\delta_{el}} \tag{3.6}$$

where P_{Rk} is the characteristic resistance of the shear connector, see Clause B.2.5(1) [1]

δ_{el} is the slip determined from push tests at a load of $0.7P_{Rk}$

From BS EN 1994-1-1:2004; Clause B.2.5(1), the characteristic resistance per shear connector is given by:

$$P_{Rk} = 0.9P_{min} \tag{3.7}$$

Where P_m is the mean resistance of the three push tests

P_{min} is the lowest of the three resistances and the results are within 10% of P_m.

3.1.3 Ductility and deformation capacity of shear connectors

BS EN 1994-1-1:2004; Clause 6.6.1.1(4)P uses the term ‘ductile’ for shear connectors that have sufficient deformation capacity to assume ideal plastic behaviour of all shear connectors from the support to the point of maximum moment. The classification of a connector as ductile depends on its characteristic slip capacity, which is defined in Clause B.2.5(4). Clause 6.6.1.1(5) as a slip capacity of 6 mm [1]. In addition, the ductility, defined as the ratio between plastic deformation and elastic deformation at the design resistance has to satisfy:

$$R = \frac{\delta_u - 1}{1} \geq 5 \tag{3.8}$$

This is based on the generally accepted idealisation of the behaviour of welded studs, for which δ_u = 6 mm and δ_{el} = 0.7 mm, see Fig. 3.2.

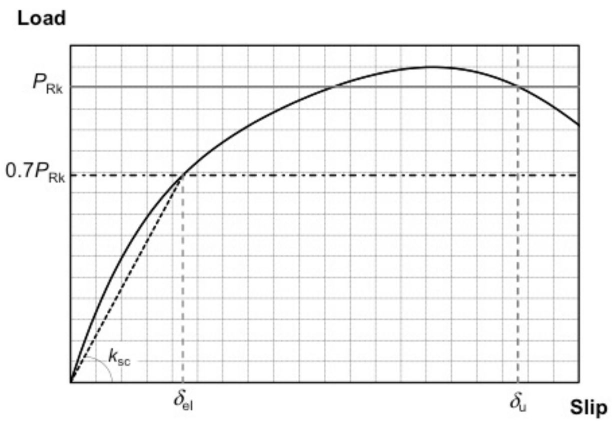


Fig. 3.1 – Characteristic load-slip curve from BS EN 1994-1-1

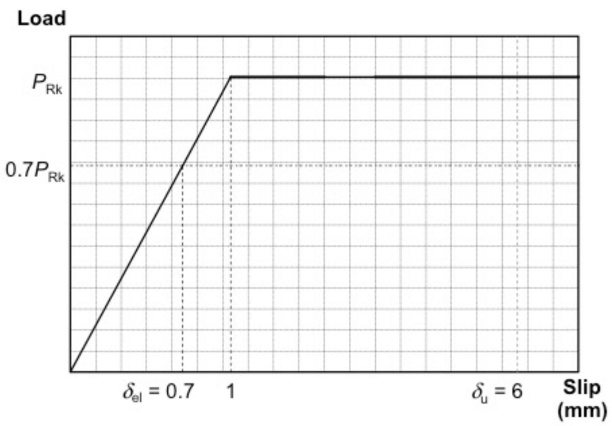


Fig. 3.2 – Idealised load-slip curve for welded studs

3.1.4 Minimum degree of partial shear connection

The degree of shear connection (η) is the ratio of the actual number of shear connectors to the number required for full shear connection over a length L_e between the points of zero and maximum moment. BS EN 1994-1-1; Clause 6.6.1.2(1) defines the minimum degree of shear connection for welded studs, which are considered to be ductile shear connectors. In the case of steel sections with equal flanges, the minimum degree of shear connection is defined as follows:

$$\begin{aligned} &\text{For } L_e \leq 25 \text{ m:} \quad \eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03L_e) \quad \text{and} \quad \eta \geq 0.4 \\ &\text{For } L_e > 25 \text{ m:} \quad \eta \geq 1 \end{aligned} \tag{3.9}$$

SCI-P359 [13] and SCI-P405 [14] have modified these limits for composite construction in the UK.

These limits enable the calculations of total end beam slip to be avoided, and are based on the assumption of a characteristic slip of 6 mm. This is an indirect approach for consideration of the ductility demands at the inferface of composite beams. For demountable connectors, the minimum degree of shear connection is replaced with a limit on the slip at SLS.

3.2 Idealisation of the load-slip behaviour of flexible, non-ductile shear connectors

3.2.1 General

The load-slip behaviour of the shear connectors influences the global behaviour of a composite beam. As an example, some typical load-slip characteristics for flexible connectors are given in Fig. 3.3: (i) ductile connectors (black curve), (ii) non-ductile shear connectors (grey curve), and (iii) non-ductile connectors of sufficient deformation capacity (orange curve).

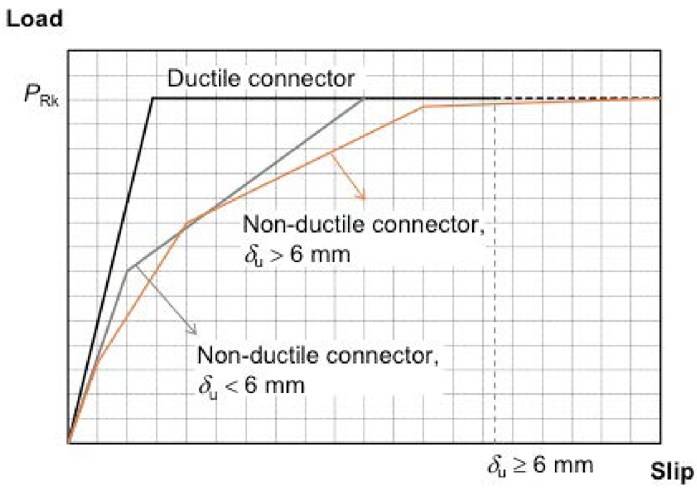
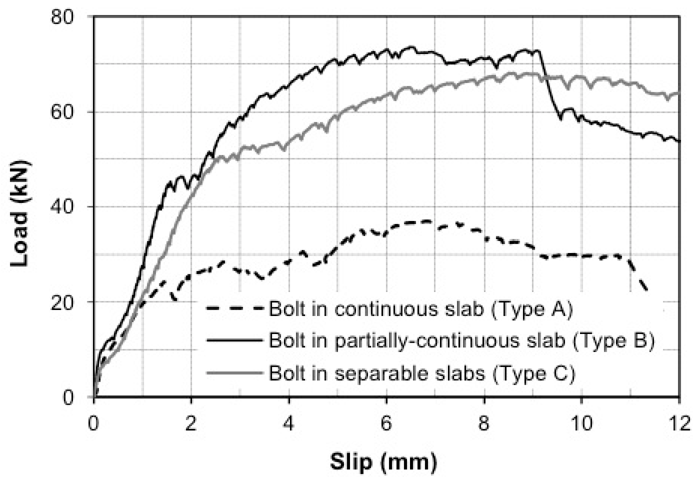


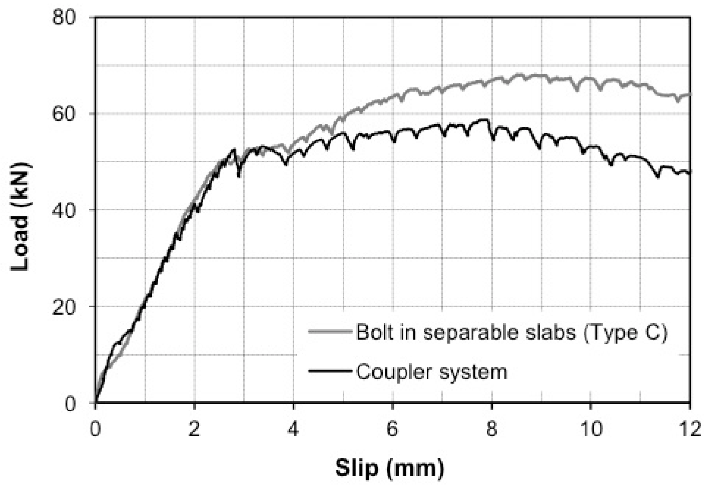
Fig. 3.3 – Typical load-slip curves for flexible connectors

The focus of this Guide is on non-ductile connectors that satisfy a characteristic slip capacity of 6 mm, according to push tests carried out at the University of Bradford [10]. Experimental results are shown in Fig. 3.4. In these tests, the important parameters are the stiffness in the elastic range and the shear resistance at high slip. The shear resistances of the bolted shear connectors with full or partial depth edge trim and the coupler system are similar to welded studs but are more flexible. This implies that designs in demountable construction will be controlled by the serviceability limit state for the same number of shear connectors unless the shear connector arrangement is made more efficient for elastic design, see Section 3.2.3.

The characteristic values and the design values of these shear connectors in accordance with the stiffness definition of BS EN 1994-1-1:2004; Clause A.3(3), are summarised in Table 3.2. (for details of the shear connection system refer to Fig. 1.1 and Fig. 2.3).



a) Bolted shear connectors



b) Bolted shear connectors and coupler system in separable slabs

Fig. 3.4 – Experimental load-slip curves for demountable shear connectors, see Section 2.1.1 (Depth of trapezoidal deck is 80 mm; failure mode governed by concrete crushing)

From the average experimental curves, and as an alternative to the stiffness definition of Clause A.3(3) of EN 1994-1-1:2004 [1], the shear connector stiffness may be obtained at an experimental slip of 1.2 mm corresponding to the serviceability limit, which leads to the values also given in Table 3.2.

Table 3.2 – Characteristic values and design values of the shear connectors from tests (concrete strength C30/37, with measured $f_{ck,test}$ = 38 N/mm²)

System	Stiffness of the connector, BS EN 1994-1-1; Clause A.3(3)			Stiffness at 1.2 mm slip	Shear resistance
	$0.7P_{Rk}$ (kN)	δ_{el} (mm)	k_{sc} (kN/mm)	$k_{sc,1.2}$ (kN/mm)	P_{Rd} (kN)
Bolt Detail A	21	1.4	15	16	24
Bolt Detail B	45	1.8	25	25	51
Bolt Detail C	39	2.3	17	20	44
Coupler	34	1.8	19	20	39

3.2.2 Modelling of shear connector resistance to longitudinal shear

Shear resistance

Table 3.4 of BS EN 1993-1-8:2005 [9] gives the design shear resistance of a bolt as follows:

$$P_{\mathrm{Rd}} = \frac{0.6 f_{\mathrm{ub}} A_{\mathrm{s}}}{\gamma_{\mathrm{M2}}} \tag{3.10}$$

where f_{ub} is the tensile strength of the bolt

A_{s} is the tensile stress area of the bolt

γ_{M2} is the partial factor (= 1.25, NA to BS EN 1993-1-8).

For M20 grade 8.8 bolts, f_{ub} is 800 N/mm^2 and A_{s} is 245 mm^2, and thus, from Eq. (3.10):

$$P_{\mathrm{Rd}} = \frac{0.6 \times 800 \times 245 \times 10^{-3}}{1.25} = 94 \text{ kN}$$

Resistance associated with concrete failure

The mode of failure in the push tests [10] was influenced by both the stiffness and the strength of the concrete, i.e. E_{cm} and f_{ck}, respectively. Therefore, for bolted shear connectors, it is assumed that Eq. (3.2) remains valid for the concrete crushing mode of failure.

Comparison with tests

For the particular bolt system Detail B in Fig 2.3, for which the observed failure mode was shear and separation due to the development of a concrete cone over the bolt:

n_{r} is taken as 1 as there is sufficient spacing between connectors in the ribs, and the presence of an edge trim and its confining effect

h_{p} is 80 mm (for ComFlor 95, which has a deck profile height of 80mm)

b_0 is 135 mm (for ComFlor 95)

h_{sc} is 120 mm.

From Eq. (3.5):

$$k_{\mathrm{t}} = \frac{0.7}{\sqrt{1}} \frac{135}{80} \left(\frac{120}{80} - 1 \right) = 0.59$$

This value is very close to the maximum value of k_{t} from Table 3.1. The values in this table recognise the beneficial role of decking in transferring shear into the composite slab. Therefore, this maximum value of k_{t}=0.6 is recommended for demountable shear connectors.

The bolts in demountable shear connector system are fully threaded. This means that instead of using the nominal bolt diameter, an effective diameter based on the bolt tensile stress area should be used. For a M20 bolt, the effective diameter, d_s, is:

$$d_s = \sqrt{\frac{4A_s}{\pi}} = \sqrt{\frac{4 \times 245}{\pi}} = 17.7 \text{ mm}$$

For concrete class C30/37, with $f_{ck,exp}$ of 38 N/mm², and with E_{cm} taken as 33 kN/mm², for C30/37, from Eq. (3.2) and using k_t = $k_{t,max}$ =0.6, with α = 1 as $h_{sc}/d \approx$ 120/20 > 4:

$$P_{Rd} = 0.6 \times \frac{0.29 \times 1 \times 17.7^2 \sqrt{38 \times 10^{-3} \times 33}}{1.25} = 49 \text{ kN}$$

This value compares well with the test design value of 51 kN, see Table 3.2.

3.2.3 Effective shear connector resistance

BS EN 1994-1-1:2004; Clause 6.6.1.1(3)P states that shear connectors shall have sufficient deformation capacity to justify any inelastic distribution of shear assumed in design.

Consider the following different shear connectors arrangements:

- Uniform distribution of pairs of shear connectors at 600 mm spacing along the beam, see Fig. 3.5a,
- Non-uniform distribution of connectors based on a 'pseudo-elastic' distribution in which more shear connectors are placed in the higher shear region, see Fig. 3.5b.

Because these demountable shear connectors do not exhibit an elastic-perfectly plastic behaviour, the shear connectors in the middle part of the span may have not developed their full shear resistance when the end connectors are at their maximum slip and their load resistance is their design resistence, P_{Rd}.

A reasonable assumption for the variation of slip along the beam is a cosine slip distribution in the connectors from maximum at the supports. In Fig. 3.6, the accuracy of the cosine function for the pseudo-elastic distribution is compared with finite element predictions for a beam subjected to uniformly distributed loads [10]. For each shear connector located at a distance, x , from the end of the beam, it is possible to calculate the corresponding load level from the load-slip curve. The averaged load of all shear connectors is $P_{Rd,eff}$.

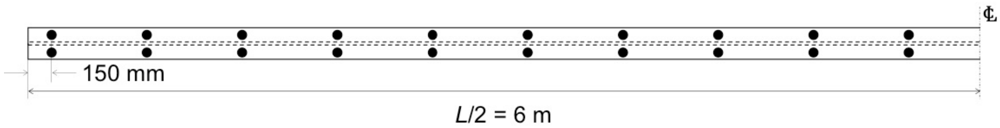
This is taken into account by means of a reduction factor, k_{flex}, that to obtain an "equivalent" idealised -plastic behaviour, as follows:

$$P_{Rd,eff} = k_{flex} P_{Rd} \tag{3.11}$$

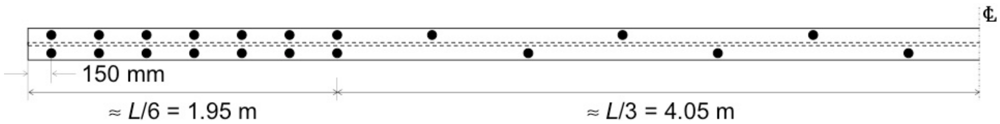
A comprehensive study has shown that the following values for bolted connectors and coupler systems are appropriate, see also Refs. [10] and [15].

$$k_{\text{flex}} = \begin{cases} 0.80 & \text{for} \quad \text{uniform spacing} \\ 0.85 & \text{for} \quad \text{pseudo-elastic distribution} \end{cases} \tag{3.12}$$

For welded studs, $k_{\text{flex}} = 1.0$.



a) Uniform distribution at 600 mm spacing, in pairs (40 connectors; this distribution is "equivalent" to single connectors at 300 mm)



b) Pairs at 300 mm spacing for the sixth span and singly at 600 mm spacing for the middle two-thirds span (40 connectors) – pseudo-elastic distribution

Fig. 3.5 – Different distributions of demountable shear connectors, example shown is for a beam of 12 m span

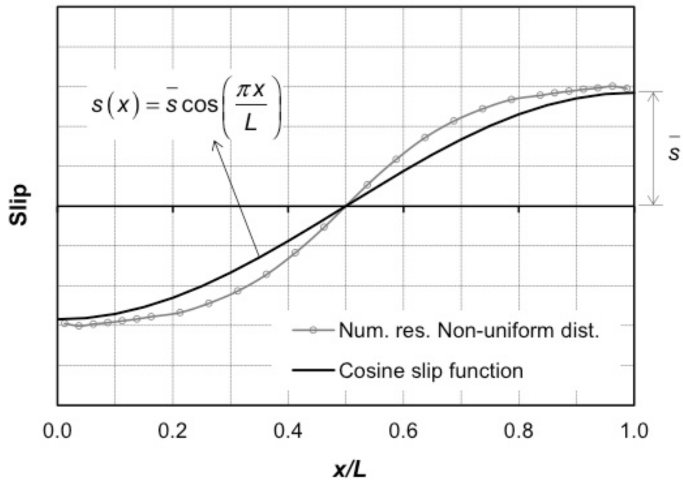


Fig. 3.6 – Cosine slip function along the beam compared to the slip obtained from finite element models for demountable shear connectors

The 11.2m span composite cellular beam with its demountable shear connectors was tested to failure at an equivalent uniform load of 24 kN/m² applied by 8 line loads.

ULTIMATE LIMIT STATE

4.1 General

The ideal plastic design model of BS EN 1994-1-1 can be adopted in design calculations if (i) the shear connector has a characteristic slip of at least 6 mm, and (ii) the degree of shear connection is greater than the minimum to Clauses 6.6.1.2 (this is given in Eq. (3.9), for a composite beam with equal flanges). If the shear connectors do not satisfy the requirements for ductile connectors, then plastic behaviour of the shear connection cannot be assumed. In this case, elastic theory should be used to determine bending resistance, see Clause 6.2.1.5 [1]

The design of composite beams using demountable and more flexible shear connectors may be performed using the current application rules to BS EN 1994-1-1 at the ULS and SLS. Elastic analysis can be used in all cases, and for shear connectors satisfying the 6 mm slip capacity, plastic analysis using the parameter, k_{flex} applied to the plastic resistance of the demountable shear connectors may be used. It is proposed to replace the requirement for minimum degree of shear connection with a slip limit at the SLS, which ensures the potential for reuse by avoiding inelastic behaviour. This design concept is shown diagrammatically in Fig. 4.1.

4.2 Plastic analysis of composite beams with partial shear connection

In plastic analysis, the bending resistance of a composite section $M_{\text{pl},\eta,\text{Rd}}$ is determined assuming that steel section has reached its yield strength f_{yd}, and the concrete has reached f_{cd} in compression over its effective width In the case of partial shear connection, the plastic neutral axis lies either in the steel web or top flange. The degree of shear connection, η, is obtained from:

$$\eta = \frac{k_{\text{flex}} P_{\text{Rd}} n_{\text{sc}}}{N_{\text{c,f}}} \tag{4.1}$$

where n_{sc} is the number of shear connectors between the points of zero and maximum moment

$N_{\text{c,f}}$ is the longitudinal force required for full shear connection, given by the smaller of the compression resistance of the concrete slab or the tensile resistance of the steel beam.

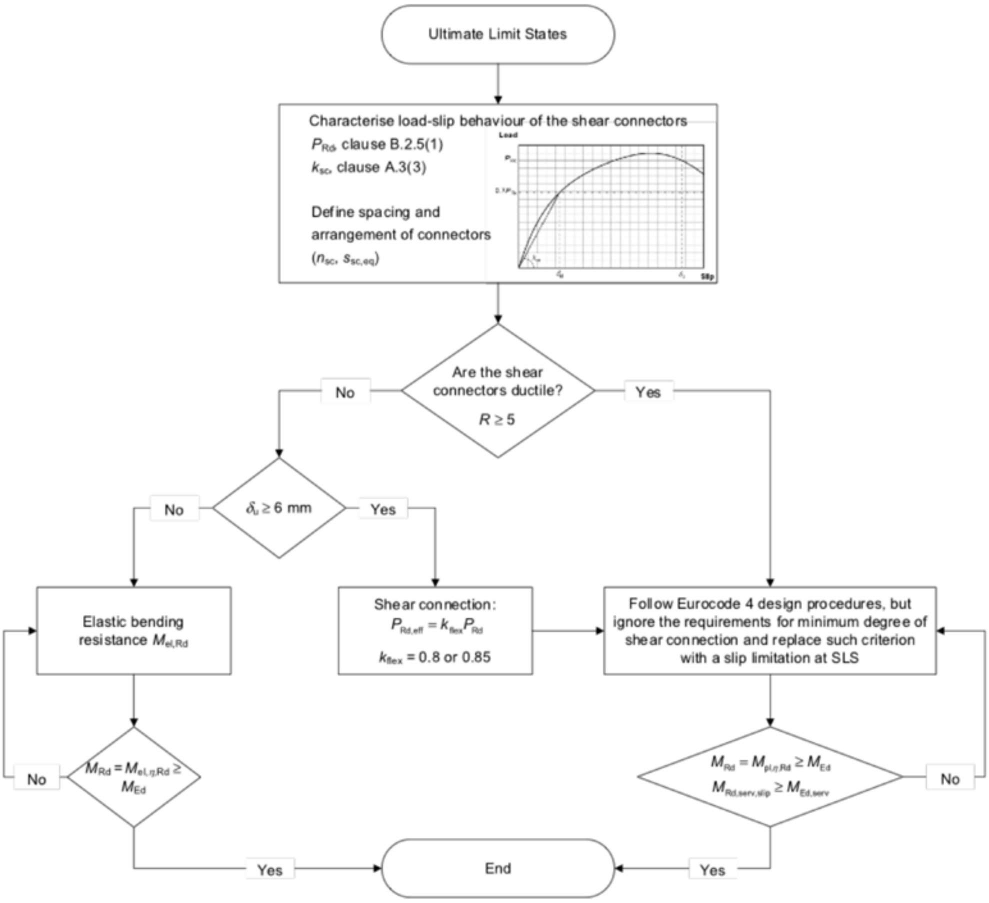


Fig. 4.1 – Overview of the design process depending on the characteristics of the shear connectors

For a given cross-section, the longitudinal force $N_{c,f}$ is obtained as:

$$N_{c,f} = \min\left(f_{yd}A_a \quad \text{and} \quad 0.85 f_{cd}A_c\right) \tag{4.2}$$

where f_{yd} is the design value of the yield strength of the structural steel

A_a is the cross-sectional area of the structural steel section

f_{cd} is the design strength of the concrete

A_c is the cross-sectional area of the concrete section

b_{eff} is the effective width of the slab (see Clause 5.4.1.2 of BS EN 1994-1-1:2004)

h_c is the depth of the concrete slab above the profiled decking

The two cases of the position of the plastic neutral axis are determined as follows:

4.2.1 Case 1: plastic neutral axis within the web ($\eta N_{c,f} < N_{pl,w}$)

The depth of the plastic neutral axis, measured from the extreme fibre of the concrete in compression, is given by:

$$z_{pl} = \frac{h_a}{2} + h_c + h_p - \frac{\eta N_{c,f}}{N_{pl,d}} \frac{d_w}{2} \tag{4.3}$$

where h_a is the depth of the structural steel section

h_p is the overall depth of the profiled steel sheeting excluding embossments

d_w is the clear depth of the steel web

$N_{pl,d}$ is the design value of the plastic resistance of the clear depth of the steel web to normal force ($= f_{yd} t_w d_w$)

The plastic bending resistance of the composite beam is determined from:

$$M_{pl,\eta,Rd} = M_{pl,a,Rd} + \eta N_{c,f}\left(\frac{h_a}{2} + h_c + h_p\right) - \left(\eta N_{c,f}\right)^2\left(\frac{h_c}{2N_{pl,c}} + \frac{d_w}{4N_{pl,d}}\right) \tag{4.4}$$

where $M_{pl,a,Rd}$ is the design value of the plastic resistance moment of the structural steel section

$N_{pl,c}$ is the design value of the compression resistance of the concrete flange ($= 0.85 f_{cd} b_{eff} h_c$)

4.2.2 Case 2: plastic neutral axis within the flange ($\eta N_{c,f} \geq N_{pl,w}$)

In the case where the plastic neutral axis lies in the top flange, the plastic bending resistance of the composite beam is determined from:

$$M_{pl,\eta,Rd} = N_{pl,a}\frac{h_a}{2} + \eta N_{c,f}\left(h_c + h_p\right) - \frac{\left(\eta N_{c,f}\right)^2}{N_{pl,c}}\frac{h_c}{2} - \frac{\left(N_{pl,a} - \eta N_{c,f}\right)^2}{N_{pl,f}}\frac{t_f}{4} \tag{4.5}$$

where $N_{pl,a}$ is the design value of the plastic resistance of the structural steel section to normal force

$N_{pl,f}$ is the design value of the plastic resistance of the steel flange to normal force ($= f_{yd} b_{tf}$).

The final term in t_f can normally be neglected as it represents the bending contribution of the top flange about its own axis.

4.3 Elastic analysis of composite beams with partial shear connection

The elastic bending resistance of the composite section $M_{el,\eta,Rd}$ is obtained from the superposition of elastic stresses for yielding of the steel section, in tension or compression, or concrete crushing. In unproped construction, the self weight loads are applied to the steel section.

4.3.1 Composite section properties

The second moment of area of an asymmetric composite section with flexible shear connectors of shear stiffness k_{sc}, at a uniform longitudinal spacing, $s_{sc,eq}$, is given by [16]:

$$I_{y,comp} = I_{y,a} + \frac{I_{y,c}}{n} + \frac{\left(h_p + 0.5h_c + z_a\right)^2}{\dfrac{A_c + nA_a}{A_c A_a} + \left(\dfrac{\pi}{L}\right)^2 \dfrac{E_a s_{sc,eq}}{k_{sc}}} \tag{4.6}$$

where $I_{y,a}$ is the second moment of area of the steel beam

$I_{y,c}$ is the second moment of area of the concrete slab $= \dfrac{b_{eff} h_c^3}{12}$

n is the modular ratio, see below

z_a is the depth of the elastic neutral axis of the steel beam from the top flange = $h_a/2$ for a symmetric section

$s_{sc,eq}$ is the equivalent uniform spacing

This formula also applies to welded shear connectors, which are placed at uniform spacing based on the centre spacing of the deck ribs (normally 300 mm). The RFCS project DISCCo [17] gave the following typical stiffness values for 19 mm diameter welded shear connectors in combination with profiled decking of 60 to 80 mm depth:

- Single shear connectors: k_{sc} = 70 kN/mm per deck rib,
- Pairs of shear connectors: k_{sc} = 100 kN/mm per deck rib.

The modular ratio is obtained from:

$$n = \frac{1}{3}n_L + \frac{2}{3}n_0 \tag{4.7}$$

The modular ratio n_L for long term loading should be calculated from:

$$n_L = n_0\left(1 + \psi_L \varphi_t\right) \tag{4.8}$$

and:

$$n_0 = \frac{E_a}{E_{cm}} \tag{4.9}$$

where ψ_L is the creep multiplier, taken as 1.1 for permanent loads

φ_t is the creep coefficient, taken as 1.5 for unpropped beams and 3.0 for propped beams

E_a is the modulus of elasticity of structural steel, given as 210 kN/mm^2 in BS EN 1993-1-1

E_{cm} is the secant modulus of elasticity for short term loading, for concrete, given in Table 3.1 of BS EN 1992-1-1.

4.3.2 Equivalent uniform spacing

For a particular distribution of shear connectors, the compression force $F_{c,s}$ that is developed in the slab is determined by integration of the shear connector forces over the half-span by considering a cosine slip function along the beam, see Fig. 3.6. Based on this slip distribution, the sum of the shear connector forces for a non-uniform spacing of shear connectors is given as follows:

$$F_{c,s} = \bar{s}\int_0^{L/2} \frac{k_{sc}}{s_{sc}(x)}\cos\left(\frac{\pi x}{L}\right)dx \tag{4.10}$$

where $\bar{s}$ is the end slip

k_{sc} is the stiffness of a shear connector

$s_{sc}(x)$ is the spacing of the shear connectors at position x from a support.

The compression force $F_{c,s}$ directly determines the stiffness of the composite section as influenced by the stiffness of the shear connectors. A non-uniform distribution of shear connectors may be included in the effective stiffness in Eq. (4.6) by integrating Eq. (4.10) for the particular pattern of shear connectors.

Fig. 3.5 shows the cases considered for the distribution of demountable shear connectors in order to determine the equivalent uniform spacing $s_{sc,eq}$ for use in Eq. (4.6). The cases are considered further in order to minimise the number of demountable shear connectors that are used, which improves the economy of demountable composite construction.

Uniform spacing of connectors

A uniform distribution at 600 mm in pairs is taken as equivalent to single connectors at 300 mm because it was found that the performance of demountable shear connectors is not affected when they are placed in pairs combined with the edge trim and the U-bars, unlike welded stud connectors. Therefore:

$$s_{sc,eq} = 300 \text{ mm} \tag{4.11}$$

The number of shear connectors, n_{sc}, in the beam length between points of zero and maximum moment for this particular distribution is obtained from

$$n_{sc} = \text{int}\left(\frac{0.5L}{0.3}\right) = \text{int}\left(\frac{L}{0.6}\right) \text{ (for span } L \text{ in [m])} \tag{4.12}$$

Equivalent uniform spacing for pseudo-elastic distribution of connectors

The spacing of the shear connectors in the pseudo-elastic distribution is at 150 mm over the outer part of the span (taken as L/6) and at a maximum of 600 mm in the middle part of the span (L/6 to L/2) one each half of the s=pan:

$$s_{sc}(x) = \begin{cases} s_{sc,1} = \dfrac{300}{2} = 150 \text{ mm} & \Leftarrow \quad 0 \le x \le \dfrac{L}{6} \\ s_{sc,2} = 600 \text{ mm} = 4s_{sc,1} & \Leftarrow \quad \dfrac{L}{6} < x \le \dfrac{L}{2} \end{cases} \tag{4.13}$$

The compression force that is developed in the slab at mid-span is obtained from Eq. (4.10), as follows:

$$\begin{aligned} F_{c,s} &= \bar{s}\int_0^{L/2} \frac{k_{sc}}{s_{sc}(x)}\cos\left(\frac{\pi x}{L}\right)dx \\ &= \bar{s}\int_0^{L/6} \frac{k_{sc}}{s_{sc,1}}\cos\left(\frac{\pi x}{L}\right)dx + \bar{s}\int_{L/6}^{L/2} \frac{k_{sc}}{4s_{sc,1}}\cos\left(\frac{\pi x}{L}\right)dx \\ &= 0.625\frac{\bar{s}L}{\pi}\frac{k_{sc}}{s_{sc,1}} \end{aligned} \tag{4.14}$$

The equivalent shear connector spacing for the same longitudinal force is therefore:

$$s_{sc,eq} = \frac{s_{sc,1}}{0.625} = 240 \text{ mm} \tag{4.15}$$

The number of shear connectors is given by Eq. (4.12).

4.3.3 Elastic bending resistance with partial interaction

In the case of partial interaction up to an elastic limit, the following assumptions are made [18]:

1. The end slip is directly proportional to the applied load.
2. The strain distribution in the slab is essentially linear.
3. The concrete does not crack in tension (this is reasonable given the likely position of the elastic neutral axis in the steel section).
4. The I-beam and the slab deflect equally without separation.

For flexible shear connectors, the rotations of the concrete slab and the steel beam produce differential movements between the underside of the concrete slab and the top of the steel beam, which leads to slip in the shear connectors.

As shown in Fig. 4.2, the slab and the steel section have their own neutral axes. The effective section modulus of the composite section is defined by the parameter S_k, which is a function of the shear connector stiffness, k_{sc}, and equivalent spacing, $s_{sc,eq}$ as follows :

$$S_k = \frac{h_p + 0.5h_c + z_a}{\dfrac{A_c + nA_a}{A_c A_a} + \left(\dfrac{\pi}{L}\right)^2 \dfrac{E_a s_{sc,eq}}{k_{sc}}} \tag{4.16}$$

The depth of the elastic neutral axis in the concrete can be obtained from, see also Ref. [15]:

$$z_{el,c} = \frac{h_c}{2} + \frac{nS_k}{A_c} \tag{4.17}$$

The depth of the elastic neutral axis in the steel section is:

$$z_{el,a} = z_a - \frac{S_k}{A_a} \tag{4.18}$$

For symmetric sections, the elastic neutral axis depth of the steel section is $z_a = 0.5h_a$.

The elastic bending resistance of the composite section is dependent on the stress limits in the concrete and steel flanges:

$$M_{el,\eta,Rd} = \min \begin{cases} \text{Top fibre: concrete in compression} & f_{cd} \dfrac{nI_{y,comp}}{z_{el,c}} \\ \text{Steel, top} & f_{yd} \dfrac{I_{y,comp}}{z_{el,a}} \\ \text{Bottom fibre: steel in tension} & f_{yd} \dfrac{I_{y,comp}}{h_a - z_{el,a}} \end{cases} \tag{4.19}$$

For unpropped construction, the elastic resistance to bending also depends on the proportion of the total load that is applied before the steel beam becomes composite.

Therefore, the elastic resistance for comparison with the design moment based on the limiting stress in the bottom flange is given by:

$$M_{el,\eta,Rd,U} = \left(1 - \frac{M_{sw,Ed}}{M_{el,a,Rd}}\right) M_{el,\eta,Rd} + M_{sw,Ed} \qquad (4.20)$$

where $M_{sw,Ed}$ is the self-weight of the concrete slab and steel beam
$M_{el,a,Rd}$ is the elastic resistance of the bare steel section (= $W_{el,y}f_{yd}$)
$M_{el,\eta,Rd}$ is the elastic bending resistance of the composite beam obtained from Eq. (4.19).

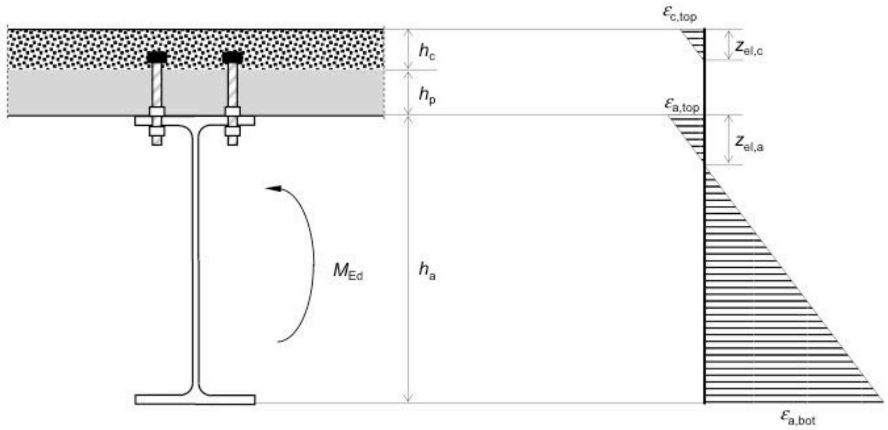


Fig. 4.2 –Cross-section and strain distribution for flexible shear connection

4.4 Additional requirements for cellular beams

The same approach may be applied to long-span composite cellular beams with multiple circular openings with the following modifications:

1. The cross-sectional area of the steel section at the centreline of the opening is used to calculate the bending resistance of the composite beam.
2. The second moment of area of the steel beam $I_{y,a}$ at the centreline of the opening is used to determine the composite stiffness and the end slip.
3. The web-post shear force is determined taking account of the number of shear connectors placed between the centreline of the openings, which may be affected by the non-uniform distribution of demountable shear connectors along the beam.

The design of the composite cellular beams may follow the procedure in SCI-P355 [11] using an equivalent rectangular opening width and depth of $0.45h_o \times 0.9h_o$ for *Vierendeel* bending. To facilitate the reuse of cellular beams, it is proposed that the following general design requirements are adopted:

- The cellular beams should be used for long span secondary beams subject to uniform loading with a minimum design imposed load of 5 kN/m². The recommended span range is 12 to 21 m based on a 1.5 m planning grid. The beam spacing should be in the range of 3 to 3.75 m depending on the slab depth.

- The cellular beam should ideally be of symmetric shape with a minimum flange width of 165 mm to be able to detail a pair of bolt holes at 100 mm transverse spacing. Symmetry allows the beam to be reversed if the hole pattern in the top flange is not suitable for the second cycle of use,
- The ratio of the opening diameter to the beam depth (h_o/h) should be in the range of 0.6 to 0.7 for efficient design for reuse of the composite cellular beam,
- The span to depth ratio should be in the range of 22 to 26 and the number of openings should be approximately equal to the numerical value of L/h. This ensures that the web-post width is not less than $0.5h_o$ so that web-post shear is not generally critical,
- Half or full cell infills should be detailed for the openings next to the supports to allow for different connections in the current and potential future uses,
- A single elongated opening may be detailed at mid-span. Therefore, the maximum length of an elongated opening is $2.6h_o$ for a web-post width of $0.6h_o$. The detailing of multiple elongated openings is not recommended if these beams are to be reused, as they are more affected by shear induced by non-uniform loading,
- If the shear connector arrangement follows the proposed pseudo-elastic distribution, which is efficient for flexural stiffness, composite design for the transfer of shear by *Vierendeel* bending may be considered for the outer openings at the ends of the span, but should be neglected for the openings in the middle two-thirds of the span. This will not normally be critical because the shear forces in this region are less than two-thirds of the maximum value at the ends of the span,
- The primary beams, if chosen as cellular beams, should be designed as non-composite.

The design of a 15 m span cellular beam with a spacing of 3.75 m is presented in detail in Section 6.3.

The cellular beam test to failure demonstrated that the bolted shear connectors could develop their full design resistance along the beam for plastic design. The deflection at failure was approximately span/100 and the end slip was over 8mm.

SERVICEABILITY LIMIT STATE

5.1 General

BS EN 1990:2002; Clauses 3.3 and 3.4 [3] define the performance requirements and loading at the ultimate (ULS) and serviceability limit states (SLS). ULS concern structural failure and collapse, rupture, safety of people, loss of equilibrium, etc. SLS correspond to functioning of the structure, comfort e.g. deflections, vibration. While criteria for ULS involve (geometrical and mechanical) parameters of the structure and appropriate actions only, the criteria for SLS are also dependent on the requirements of the client and users, and affect the effective use and reuse of the structure. Therefore, the reusability criteria require consideration of the serviceability performance of demountable composite beams in the normal conditions of use of the structure.

The serviceability requirements to be considered are summarised as follows, see also Table 5.1, and are further explained in the next sections:

- Deflections, which should be determined using the effective second moment of area that takes into account the flexibility and the spacing of the shear connectors, see Sections 4.3.1 and 4.3.2,
- Excessive vibration, which can cause discomfort to people and limit the function of the structure,
- The end-slip due to characteristic load combinations, which should not cause plastic deformations in the shear connector,
- The stresses due to characteristic load combination in the steel and in the concrete should not exceed their characteristic elastic limit.

Fatigue is not a design issue, and the bolted shear connectors should perform at least as well as a welded stud.

5.2 Deflections and vibrations

BS EN 1994-1-1 does not provide numerical serviceability criteria, but for office buildings, the following limiting values are generally accepted in the UK:

- Imposed load deflection ≤ span/360 (this also includes the super-imposed dead loads). This is less than the Eurocode 3 limit of span/300 in order to avoid damage to finishes,

- Total load deflection ≤ span/250. This limit is applied to avoid visible deflections and to be able to install partitions and ceilings, etc. A limit on the absolute deflection is not required, as this is limited in practice by a further limit on the minimum natural frequency of the beam.
- Natural frequency of the beams, f ≥ 4 Hz for single members (depending on the application) and ≥ 5 Hz for floor grillages where beam deflections are combined.

Table 5.1 presents the proposed serviceability requirements for composite beam design using demountable shear connectors.

Table 5.1 – SLS checks for demountable composite beams

SLS design check	Load combination	Consideration of reusability beyond current requirements
1. Beam deflection	Quasi-permanent	$I_{y,comp}$ including k_{sc} with consideration of the stiffness of demountable shear connectors
2. End-slip	Characteristic	Slip calculation at the SLS with $I_{y,comp}$ including k_{sc}
3. Stresses	Characteristic	Stress calculation for the steel section and concrete using $I_{y,comp}$ including k_{sc},
4. Natural frequency	Permanent loads plus 10% imposed loads	$I_{y,comp}$ using the fully composite stiffness

5.2.1 Imposed load deflection

The imposed load deflection δ_i of a uniformly loaded composite beam is given as follows:

$$\delta_i = \frac{5q_{serv}L^4}{384EI_{y,comp}} \leq \frac{L}{360} \tag{5.1}$$

where q_{serv} is the service load for quasi-permanent load combinations.

5.2.2 Total load deflection

The total load deflection of a composite beam arises from the:

- Deflection from the self-weight loads,
- Deflection associated with the imposed and superimposed loads.

This means that the calculation depends on the method of construction, as follows:

- For unpropped composite beams, the self-weight loads act on the steel beam, and the imposed and superimposed loads act on the composite beam,
- For propped composite beams, the self-weight loads act also on the composite beam which adds to the deflection of the composite section.

For unpropped construction, the total deflection of the composite beam is thus given by:

$$\delta_t = \frac{5q_{sw}L^4}{384EI_{y,a}} + \frac{5q_{serv}L^4}{384EI_{y,comp}} \leq \frac{L}{250} \tag{5.2}$$

5.2.3 Natural frequency

The natural frequency of the beam is calculated from a simple expression:

$$f = \frac{18}{\sqrt{\delta_{sw}}} \tag{5.3}$$

where δ_{sw} is the deflection due to the self-weight and superimposed loads plus a nominal 10% imposed load.

In this formula, δ_{sw} is in [mm]. For a composite beam, this deflection is calculated for loads applied to the fully composite section because of the small displacements involved at the low load levels due to rapid walking or occupant induced vibration.

5.3 End slip

The end slip of a uniformly loaded composite beam, $\bar{s}$, based on the mid-span moment M_{Ed}, see calculated for the characteristic load combination, is obtained from [15]:

$$\bar{s} = \frac{\left(h_p + 0.5h_c + z_a\right)M_{Ed}}{\frac{\pi}{L}E_a I_{y,comp}\left[1 + \frac{k_{sc}}{E_a s_{sc,eq}}\left(\frac{L}{\pi}\right)^2 \frac{A_c + nA_a}{A_c A_a}\right]} \tag{5.4}$$

The limit on the end slip is chosen so as not to cause plastic deformation in the shear connectors. This equation is derived based on the following assumptions:

1. The slip distribution along the beam follows a cosine function, see Fig. 3.6.
2. The compression force at the interface, at beam mid-span, is obtained from Eq. (4.10), for uniform shear connection using the equivalent uniform spacing, $s_{\mathrm{sc,eq}}$.
3. The rate of change of slip along the length of the beam is equal to the strain difference between the slab and beam at the interface. From assumption 4, in Section 4.3.3, this is obtained as:

$$\frac{ds}{dx} = \varepsilon_a - \varepsilon_c = F_{c,s}\left(\frac{1}{E_s A_s} + \frac{n}{E_s A_c}\right) - \frac{\left(h_p + 0.5h_c + z_a\right)M_{Ed}}{E_a I_{y,comp}} \tag{5.5}$$

Based on the available experimental data, see Table 3.2, it was found that the maximum end slip, when structural reuse is a design consideration, should be limited to 1.2 mm (< δ_{el}):

$$\bar{s}_{max} = 1.2 \text{ mm} \tag{5.6}$$

5.4 Stress checks

The stresses in the steel and in the concrete are calculated from the characteristic load combination and should not exceed their elastic limit stresses. This verification is only required when the beam is designed using plastic analysis at the ULS.

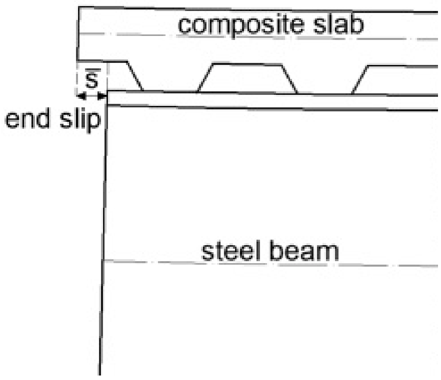


Fig. 5.1 – End slip between the slab and the steel section in a composite beam

5.5 Additional requirements for cellular beams

The additional deflections due to the openings in a cellular beam arise from the loss of flexural stiffness at the openings and from the additional shear deformation at the openings. For composite cellular beams, tests have shown that the additional shear deflection is small as a result of composite action of the slab locally, and this component can be neglected [10].

The flexural stiffness at the centreline of the web opening of a composite beam compared to the solid web section is reduced more than in a non-composite beam because the elastic neutral axis of the composite section is closer to the top flange. The effective length of a circular opening for deflection calculations may be taken as $0.7h_o$, where h_o is the opening diameter, and so the effective flexural stiffness of the composite beam also taking account of the flexibility of the shear connectors is given by:

$$I_{y,comp,red} = I_{y,comp} - \frac{0.7h_o}{s}\left(I_{y,comp} - I_{y,comp,o}\right) \qquad (5.7)$$

where $I_{y,comp}$ is the effective stiffness of the solid web composite beam taking account of the flexibility of the shear connectors, given by Eq. (4.6)

$I_{y,comp,o}$ is the effective stiffness of the composite beam at the centreline of the openings taking account of the flexibility of the shear connectors

s is the centre-to-centre spacing of the openings

h_o is the opening diameter

a) Pair of identical test beams with edge rims to facilitate demounting before the slabs were concreted

b) Pair of test beams-the right hand beam to be loaded by 2 point loads before demounting and re-assembling

c) Slab segments cut from composite beam having been tested to failure

d) Grouting of the sab segments before the beam was re-tested to failure

Tests at the University of Bradford (for EPSRC) on a pair of identical 6m span composite beams with demountable shear connectors: one beam to be load tested, the slab cut into segments, re-assembled and re-tested, and the other to be tested in its original form. The tests on the two beams showed that demounting and re-assembly did not adversely affect their stiffness and failure load.

APPLICATION TO OFFICE BUILDINGS

6.1 General

The two worked examples presented in the following sections are presented for a composite beam with a solid web, and for a cellular beam with regular circular openings in the web, in order to illustrate the guidance given in this publication.

6.2 Design example 1

Verify the adequacy of a 12 m span composite secondary beam with demountable bolted shear connectors. The beam supports a 130 mm deep composite slab and is subject to a recommended imposed load of 5 kN/m^2. The chosen beam size is 457×191×98 kg/m UB in S355 steel. The steel beam is unpropped during construction.

6.2.1 Dimensions and properties

General dimensions

Span of beam	L	=	12 m
Spacing of beams	b	=	3 m (for 12 m x 6 m grid)
Effective slab width	b_{eff}	=	L/4 = 3 m
Slab depth (composite)	h_s	=	130 mm
Deck depth	h_p	=	60 mm
Deck thickness	t	=	0.9 mm
Slab depth above profile	h_c	=	70 mm
Height of shear connectors	h_{sc}	=	100 mm

Section properties of the beam

Beam size 457×191×98 kg/m UB:

Depth	h	=	467 mm
Breadth	b	=	193 mm
Flange thickness	t_f	=	19.6 mm
Web thickness	t_w	=	11.4 mm
Root radius	r	=	10.2 mm
Clear depth of web	d_w	=	408 mm

Cross-sectional area	A_a	=	12500 mm²
Second moment of area	I_y	=	457 x 106 mm⁴
Elastic section modulus	$W_{el,y}$	=	1.96 x 106 mm³
Plastic section modulus	$W_{pl,y}$	=	2.23 x 106 mm³

Material properties

Steel grade S355	f_y	=	345 N/mm² (t_f > 16 mm)
	E_a	=	210 x 103 N/mm²
Concrete grade C30/37	f_{ck}	=	30 N/mm²
	E_{cm}	=	33 x 103 N/mm²
Design resistance of shear connectors	P_{Rd}	=	51 kN

6.2.2 Actions and design values of forces

Actions

Self-weight loads:

Self-weight of steel beam	g_a	=	$\frac{98 \times 9.8 \times 10^{-3}}{3} = 0.3$ kN/m²
Self-weight of concrete slab (catalogue)	g_c	=	2.3 kN/m²
Total self-weight (unfactored)	g_{sw}	=	2.6 kN/m²
Composite stage:			
Superimposed dead loads	g_d	=	0.5 k/Nm²
Total permanent loads	g_k	=	2.6 + 0.5 = 3.1 kN/m²
Imposed loads	q_k	=	5 kN/m²
Factored load	q_{Ed}	=	1.35 x 3.1 + 1.5 x 5
		=	11.7 kN/m²
Unfactored load Unpropped const.	q_{serv}	=	5 + 0.5 = 5.5 kN/m²

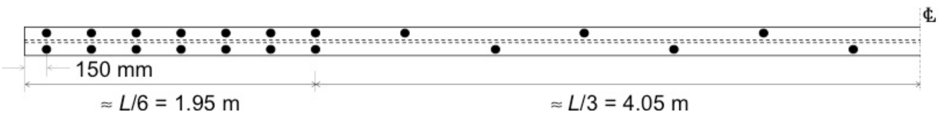
Design bending moments

At ULS	M_{Ed}	=	$11.7 \times 3 \times \frac{12^2}{8} = 632$ kNm
At SLS Unpropped const.	M_{serv}	=	$5.5 \times 3 \times \frac{12^2}{8} = 297$ kNm

6.2.3 Shear connection

The design of the composite beam is based on the bolted shear connector Detail B with bolts and nuts above/below the beam flange using the design shear resistance and stiffness obtained from push tests. The shear connectors are distributed non-uniformly based on a pseudo-elastic distribution (for which k_{flex} = 0.85 for plastic design).

Distribution of shear connectors in the half span



The shear connectors are non-ductile but satisfy the requirement of a slip capacity of 6 mm. Therefore, use $P_{\mathrm{Rd,eff}} = k_{\mathrm{flexPRd}}$ for plastic design.

Number of shear connectors to mid-span	n_{sc}	=	20
Equivalent uniform spacing	$s_{sc,eq}$	=	240 mm
Bolted connector Detail B	k_{sc}	=	25 kN/mm
	$P_{\mathrm{Rd,eff}}$	=	0.85 x 51 = 43 kN
Longitudinal force, full shear connection	$N_{c,f}$	=	$0.85 \times \frac{30}{1.5} \times 70 \times 3000 \times 10^{-3}$
		=	3570 kN
Compression force in the slab	$N_{sc,Ed}$	=	43 x 20 = 860 kN
Degree of shear connection	η	=	$\frac{860}{3570} = 0.24 \ (24\%)$

6.2.4 Design Checks at Ultimate Limit State

Plastic bending resistance

From Eq. (4.4), the plastic bending resistance with partial shear connection is calculated as follows:

Plastic resistance of the steel section	$M_{pl,a,Rd}$	=	2.23 x 10^6 x 345 x 10^{-6} = 769 kNm
Compression resistance of concrete flange	$N_{pl,c}$	=	3570 kN
Plastic resistance of clear depth of web	$N_{pl,d}$	=	355 x 11.4 x 408 x 10^{-3}
		=	1651 kN

Therefore, the plastic bending resistance for the case where the plastic neutral axis is in the web is given by:

$$M_{pl,\eta,Rd} = 769 + 0.24 \times 3570 \times \left(\frac{467}{2} + 70 + 60\right) \times 10^{-3}$$

$$- (0.2 \times 3570)^2 \times \left(\frac{70}{2 \times 3570} + \frac{408}{4 \times 1651}\right) \times 10^{-3}$$

$$= 1028 \text{ kNm}$$

This value is greater than the design moment of 632 kNm and so the utilisation factor (UF) in bending is 61%.

Elastic bending resistance

From the equations in Section 4.3, the elastic bending resistance for the flexible shear connectors is:

Area of concrete flange	A_c	=	3000 x 70 = 210 x 10^3 mm^2
Modular ratio Unpropped construction	n	=	$\frac{210}{3 \times 33}(1 + 1.1 \times 1.5 + 2)$ = 10

The second moment of area of the composite section taking account of the flexibility of the shear connectors is:

$$I_{y,comp} = 457\times10^6 + \frac{3000\times70^3}{10\times12} + \frac{\left(60+0.5\times70+0.5\times467\right)^2}{\dfrac{210\times10^3+10\times1.25\times10^4}{210\times10^3\times1.25\times10^4} + \left(\dfrac{\pi}{12000}\right)^2 \times \dfrac{210\times240}{25}}$$

$$= 872\times10^6 \text{ mm}^4$$

In comparison, the second moment of area of the fully composite section with rigid shear connectors is:

$$I_{y,comp} = 457\times10^6 + \frac{3000\times70^3}{10\times12} + \frac{\left(60+0.5\times70+0.5\times467\right)^2}{\dfrac{210\times10^3+10\times1.25\times10^4}{210\times10^3\times1.25\times10^4}} = 1315\times10^6 \text{mm}^4$$

The elastic section modulus, S_k is obtained from Eq. (4.16):

$$S_k = \frac{60+0.5\times70+0.5\times467}{\dfrac{2.1\times10^5+10\times1.25\times10^4}{2.1\times10^5\times1.25\times10^4} + \left(\dfrac{\pi}{12000}\right)^2 \times \dfrac{210\times240}{25}} = 1.24\times10^6 \text{ mm}^3$$

Depth of elastic neutral axis $\quad z_{el.c} = \dfrac{70}{2} + \dfrac{10\times1.24\times10^6}{2.1\times10^5}$ = 94 mm

Unpropped construction $\quad z_{el.a} = \dfrac{467}{2} - \dfrac{1.24\times10^6}{1.25\times10^4}$ = 134 mm

The elastic bending resistance of the beam is determined from:

(i) Elastic bending resistance of the composite section:

$$M^*_{el,\eta,Rd,U} = \min\begin{cases} \text{Top of slab} & \dfrac{30}{1.5}\times\dfrac{10\times872\times10^6}{94}\times10^{-6} = 1855 \text{ kNm} \\ \text{Top flange} & 345\times\dfrac{872\times10^6}{134}\times10^{-6} = 2245 \text{ kNm} \\ \text{Bottom flange} & 345\times\dfrac{872\times10^6}{467-134}\times10^{-6} = 903 \text{ kNm (*)} \end{cases}$$

(*) This value governs resistance.

(ii) Elastic resistance of the bare steel section:

$$M_{el,a,Rd} = 345x10^6 \times 1.96x10^{-6} = 676 \text{ kNm}$$

(iii) Self-weight of the concrete slab and steel beam:

$$M_{sw,Ed} = \frac{2.6\times3\times12^2}{8} = 140 \text{ kNm}$$

(iv) Elastic bending resistance taking account of the moment applied to the steel beam is:

$$M_{el,\eta,Rd,U} = \left(1-0.21\right)\times903+140 = 853 \text{ kNm}$$

The elastic bending resistance is greater than the design moment of 632 kNm acting on the composite section.

6.2.5 Design checks at Serviceability Limit States

End slip calculations

At the SLS for a maximum slip of 1.2 mm, and using the modular ratio for an un-propped beam, the serviceability bending moment is:

$$M_{\mathrm{Rd,slip,2}} = \frac{\dfrac{\pi\times210\times872\times10^{6}}{12000}\left[1+\dfrac{25}{210\times240}\left(\dfrac{12000}{\pi}\right)^{2}\dfrac{210\times10^{3}+10\times1.25\times10^{4}}{210\times10^{3}\times1.25\times10^{4}}\right]}{60+0.5\times70+0.5\times467}\times1.2\times10^{-3}$$

$$= 337 \text{ kNm}$$

This value is greater than the design moment of 297 kNm at the SLS and shows that the slip is 1.2x(297/337) = 1.06 mm.

Deflections

The total imposed load deflection is calculated for the service load and should not exceed span/360:

$$\delta_i = \frac{5\times5\times3\times12^4\times10^9}{384\times210\times872\times10^6} = 22 \text{ mm} \le \frac{12000}{360} = 33 \text{ mm}$$

The imposed load deflection is span/545.

The total load deflection of the beam should not be greater than span/250 in general applications. For unpropped construction, the total load deflection arises from (i) the deflection of the steel beam during construction, and (ii) the deflection of the composite beam due to the service load. The first term is:

$$\delta_{conc} = \frac{5\times2.6\times3\times12^4\times10^9}{384\times210\times457\times10^6} = 22 \text{ mm}$$

The total deflection is :

$$\delta = 22+22 = 44 \text{ mm} \le \frac{12000}{250} = 48 \text{ mm}$$

The total load deflection is span/270.

Vibration sensitivity

For a minimum limit of natural frequency of f = 4 Hz that is typical for the design of composite floors in offices, the limiting mid-span displacement of the composite beam when subjected to the permanent loads plus 10% of imposed load is obtained from the simplified formula for natural frequency:

$$f = \frac{18}{\sqrt{\delta_f}} \ge 4\text{Hz}$$

Therefore, the limiting deflection is $\delta_f = \left(\frac{18}{4}\right)^2 = 20 \text{ mm}$.

The deflection of the composite beam is determined using the fully composite stiffness for this load level, which is:

$$\delta_f = \frac{5 \times (3.1 + 0.1 \times 5) \times 3 \times 12^4 \times 10^9}{384 \times 210 \times 1.35 \times 10^6} = 10.3 \text{ mm}$$

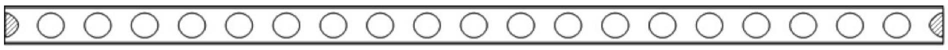
This corresponds to a beam natural frequency of:

$$f = \frac{18}{\sqrt{10.3}} = 5.6 \text{ Hz} > 4 \text{ Hz}$$

This shows that the total deflection is the limiting criterion.

6.3 Design example 2: Cellular beam

Verify the adequacy of a 15 m span composite cellular secondary beam in accordance with the guidance given in this publication. The beam is subjected to a uniform imposed load of 5 kN/m^2. The method of construction is unpropped. Cells are infilled next to the supports, as shown below:



6.3.1 Dimensions and properties

General dimensions

Span of beam	L	=	15 m
Spacing of beams	b	=	3.75 m (for 15 x 7.5 m^2 grid)
Effective slab width	b_{eff}	=	L/4 = 3.75 m
Slab depth (composite)	h_{s}	=	150 mm
Deck depth	h_{p}	=	80 mm
Deck thickness	t	=	1.2 mm
Slab depth above profile	h_{c}	=	70 mm

Section properties of the beam

Beam size 533×210×109 kg/m UB

Depth	h	=	665 mm
Breadth	b	=	211 mm
Flange thickness	t_{f}	=	18.8 mm
Web thickness	t_{w}	=	11.6 mm
Root radius	r	=	12.7 mm
Diameter of openings	h_{o}	=	425 mm
Spacing between openings	s	=	750 mm
Depth of Tees	h_{T}	=	120 mm
Area of Tees (neglecting radius)	A_{T}	=	$(120 - 18.8) \times 11.6 +$ $+ 211 \times 18.8 = 5141 \text{ mm}^2$

Depth of centroid of Tee from flange	z_{el}	=	$\frac{1}{5141}\left(101\times11.6\times69.3 + +211\times\frac{18.8^2}{2}\right) = 22$ mm

Material properties

Steel grade S355	f_y	=	345 N/mm² (t_f > 16 mm)
	E_a	=	210 x 103 N/mm²
Concrete grade C30/37	f_{ck}	=	30 N/mm²
	E_{cm}	=	33 x 103 N/mm²
Design resistance of shear connectors	P_{Rd}	=	51 kN

6.3.2 Actions and design values of forces

Actions

Self-weight loads

Self-weight of steel beam	g_a	=	$\frac{109\times9.8\times10^{-3}}{3.75} = 0.3$ kN/m²
Self-weight of concrete slab (catalogue)	g_c	=	2.5 kN/m²
Total self-weight (unfactored)	g_{sw}	=	2.8 kN/m²

Composite stage

Total permanent loads	g_k	=	5 kN/m²
Imposed loads	q_k	=	1.35 x 2.8 + 1.5x5
Factored load	q_{Ed}	=	11.3 kN/m²
Unfactored load Unpropped const.	q_{serv}	=	5 kN/m²

Design bending moments

At ULS	M_{Ed}	=	$11.3\times3.75\times\frac{15^2}{8} = 1192$ kNm
At SLS Unpropped construction	M_{serv}	=	$5\times3.75\times\frac{15^2}{8} = 527$ kNm

Design shear forces

At ULS	V_{Ed}	=	$11.3\times3.75\times\frac{15}{2} = 318$ kN

Use infills for the last cell. The shear force at the second cell is given by:

$$V_{Ed,cell\#2} = 11.3\times3.75\times\left(\frac{15}{2} - 0.75\right) = 286 \text{ kN}$$

6.3.3 Shear connection

The design of the composite beam is based on the bolted shear connector Type B with bolts and nuts above/below the beam flange using the design shear resistance and stiffness obtained from push tests. The shear connectors are distributed non-uniformly, i.e. they paced in a pseudo-elastic distribution, as above (for which k_{flex} = 0.85).

Number of shear connectors to mid-span	n_{sc}	=	26
Equivalent uniform spacing	$s_{sc,eq}$	=	240 mm
Bolted connector Detail B	k_{sc}	=	25 kN/mm
	$P_{Rd,eff}$	=	0.85 x 51 = 43 kN
Longitudinal force, full shear connection	$N_{c,f}$	=	$0.85\times3750\times70\times\dfrac{30\times10^{-3}}{1.5}$
		=	4248 kN
Compression force in the slab	$N_{s,Ed}$	=	43 x 26 = 1118 kN
Degree of shear connection	η	=	$\dfrac{1118}{4248}=0.26\ (26\%)$

6.3.4 Design checks at Ultimate Limit State

Plastic bending resistance at the centreline of the opening

Compression/tension resistance of Tees	$N_{T,Rd}$	=	$\dfrac{5141\times345\times10^{-3}}{1.0}=1774$ kN
Effective depth between centroids of Tees	h_{eff}	=	665-2 x 22 = 621 mm
Depth of concrete in compression	z_c	=	$70\times\dfrac{1118}{4248}=18$ mm

Because $N_{s,Ed} < N_{bT,Rd}$, then the plastic neutral axis (p.n.a.) lies in the flange of the top Tee. Assume the p.n.a. located at half of the flange thickness. The plastic bending resistance is therefore given by:

$$M_{c,\eta,Rd} = 1774\times621\times10^{-3}+1118\times(22+150-0.5\times18)\times10^{-3}=1284 \text{ kNm}$$

Shear resistance of perforated composite beam section

The design shear resistance is the sum of the resistances of the top and bottom Tees and the concrete slab, calculated below:

Shear area of Tees (neglecting radius)	$A_{v,T}$	=	$5141-211\times18.8+0.5\times11.6\times18.8$
		=	1283 mm^2
Shear resistance of Tee	$V_{T,Rd}$	=	$\dfrac{1283\times345\times10^{-3}}{\sqrt{3}\times1.0}=256$ kN
Estimated shear resistance of the slab	$V_{c,Rd}$	=	35 kN

The total shear resistance at the opening is therefore:

$$V_{Rd} = 35+2\times255=545 \text{ kN}$$

This exceeds the design shear force at opening 2, $V_{Ed,cell\#2}$ = 286 kN.

Bending resistance of Tees

Section classification

The top flange is class 2 due to its attachment to the slab. Classification of the web outstand of the top Tee in *Vierendeel* bending (ignoring axial compression):

- Effective length of the opening for local buckling classification:

$$a_{o,eff} = 0.7 \times 425 = 297.5 \text{ mm}$$

- For the web to be class 2, independent of its depth:

$$a_{o,eff} < 32\varepsilon t_w < 32 \times \sqrt{\frac{235}{345}} \times 11.6 = 306 \text{ mm}$$

Therefore, the Tee is class 2 for its *Vierendeel* bending resistance.

Assumed distribution of force

Initially, assume that 50% of the shear force V_{Ed} is resisted in each Tee. Since the shear force in each Tee (286/2 = 143 kN) is greater than $0.5V_{T,Rd}$, the web thickness should be reduced when determining the plastic bending resistance and axial resistance of the Tees.

If the shear force in the bottom Tee is limited by *Vierendeel* bending resistance across the Tee, the shear forces may need to be redistributed.

Plastic bending resistance

For a class 2 cross section, the plastic bending resistance of an unstiffened Tee, in the absence of axial force, and in the presence of high shear is determined below.

Utilisation of the cross section in shear $\mu = \dfrac{0.5 \times 286}{256} = 0.56$

Effective web thickness $t_{w,eff} = 11.6 \times \left[1 - (2 \times 0.56 - 1)^2\right] = 11.4 \text{ mm}$

Depth of p.n.a from outer flange $z_{pl} = \dfrac{211 \times 18.8 + 101 \times 11.6}{2 \times 211} = 12 \text{ mm}$

Therefore, the plastic resistance of a Tee is:

$$M_{T,pl,Rd} = \frac{101 \times 11.4 \times 345 \times 10^{-3}}{1.0}(0.5 \times 101 + 18.8 - 12) \times 10^{-3} + \frac{211 \times 18.8 \times 345 \times 10^{-3}}{1.0}\left(0.5 \times 18.8 - 12 + \frac{12^2}{18.8}\right) \times 10^{-3}$$

$$= 29.7 \text{ kNm}$$

The plastic bending resistance of the bottom Tee has to be further reduced for axial tension.

Axial force in bottom tee $N_{bT,Ed} = \dfrac{1192 \times 10^3}{621 + 22 + 150 - 0.5 \times 70}$

$$= 1572 \text{ kN}$$

Reduced bending resistance $M_{bT,N,Rd} = 29.7 \times \left[1 - \left(\frac{1572}{1774}\right)^2\right]$

$= 6.4$ kNm

Composite bending resistance

If the shear connector arrangement follows the pseudo-elastic arrangement, a pair of shear connectors acts over the first three openings, but in the middle two -thirds of the beam, the shear connectors are placed more widely and do not align with the openings. Therefore, in the worst case, assume that there are no shear connectors over the openings, and thus no local composite action is developed. Therefore, the component of *Vierendeel* bending resistance due to composite action, $M_{vc,Rd}$, is neglected.

Verification of resistance to Vierendeel bending

The criterion for adequacy of *Vierendeel* bending resistance is:

$$2M_{bT,NV,Rd} + 2M_{tT,NV,Rd} + M_{vc,Rd} \geq V_{Ed}a_e$$

The length of the equivalent rectangular opening for *Vierendeel* bending is:

$$a_e = 0.45 \times 425 = 191 \text{ mm}$$

Using the above values for the Tees, the *Vierendeel* bending criterion is:

$$2 \times 6.4 + 2 \times 29.7 + 0 = 72 \geq 286 \times 191 \times 10^{-3} = 55 \text{ kN} \quad \text{OK}$$

Web-post verifications

Web-post bending resistance

The elastic bending resistance of the web-post, at mid-height between circular openings, is given by:

$$M_{wp,Rd} = \frac{(750 - 425)^2 \times 11.6 \times 345 \times 10^{-6}}{6} = 70.5 \text{ kNm}$$

Since the *Vierendeel* bending resistance was verified above for equal shear force in each of the Tees, with no shear force in the slab, the web-post moment at mid-height is zero. Hence, this verification is not required.

Web-post shear resistance

The horizontal shear force in the web-post assuming no composite action is:

$$V_{wp,Ed} = \frac{286 \times 750}{621 + 22 + 150 - 0.5 \times 70} = 283 \text{ kN} \quad \text{at the first opening}$$

$$V_{wp,Ed} = \frac{\frac{2}{3} \times 286 \times 750}{621 - 2 \times 22} = 248 \text{ k} \quad \text{at the fourth opening}$$

The longitudinal shear resistance of the web-post is given by:

$$V_{wp,Rd} = \frac{(750-425)\times 11.6\times 345\times 10^{-3}}{\sqrt{3}\times 1.0} = 751 \text{ kN} > 283 \text{ kN}$$

Web-post buckling resistance

Since the openings are placed centrally in the web depth and the web-post moment required at mid-height is zero, the web-post buckling is checked for $N_{wp,Rd} = V_{wp,Ed}$.

The web-post buckling resistance is computed from the simple strut model.

Non-dimensional slenderness ratio $\bar{\lambda}_{wp} = \dfrac{1.75\sqrt{325^2+425^2}}{11.6\times 93.9}\times\sqrt{\dfrac{355}{235}} = 1.06$

Imperfection factor (curve a) $\alpha = 0.21$

Factor $\Phi = \dfrac{1+0.21\times(1.06-0.2)+1.06^2}{2} = 1.15$

Reduction factor for buckling $\chi_{wp} = \dfrac{1}{1.15+\sqrt{1.15^2-1.06^2}} \leq 1.0 = 0.63$

The web-post buckling resistance is:

$$N_{wp,Rd} = 0.63\times\frac{(750-425)\times 11.6\times 345\times 10^{-3}}{1.0} = 815 \text{ kN} > 283 \text{ kN}$$

6.3.5 Design checks at Serviceability Limit States

End slip calculations

From the equations in Section 4.3, the composite stiffness with flexible shear connectors is obtained for the following elastic beam properties:

Area of solid web steel section	A_a	=	15.2 x 10^3 mm^2
Area of steel section at opening	$A_{a,o}$	=	10.3 x 10^3 mm^2
Second moment of area of solid web steel section	$I_{y,a}$	=	1066 x 10^6 mm^4
Second moment of area of steel section at opening	$I_{y,a,o}$	=	993 x 10^6 mm^4
Area of concrete flange	A_c	=	3750 x 70 = 263 x 10^3 mm^2
Modular ratio for unpropped construction	n	=	10

The second moment of area of the solid web composite beam is given by:

$$I_{y,comp} = 1066\times 10^6 + \frac{\frac{3750\times 70^3}{12}}{10} + \frac{(80+0.5\times 70+0.5\times 665)^2}{\frac{263\times 10^3+10\times 15.2\times 10^3}{263\times 10^3\times 15.2\times 10^3}+\left(\frac{\pi}{15000}\right)^2\times\frac{210\times 240}{25}}$$

$$= 2.12\times 10^9 \text{ mm}^4$$

The second moment of area of the solid web composite beam at the openings is given by:

$$I_{y,comp,o} = 993\times10^6 + \frac{\frac{3750\times70^3}{12}}{10} + \\ + \frac{(80+0.5\times70+0.5\times665)^2}{\frac{263\times10^3+10\times10.3\times10^3}{263\times10^3\times10.3\times10^3} + \left(\frac{\pi}{15000}\right)^2 \times \frac{210\times240}{25}} \\ = 1.9\times10^9 \text{ mm}^4$$

It follows that the composite stiffness is reduced by 10% at the opening positions.

Consider the effective opening length of $0.7h_o$ = 297 mm and s = 750 mm. The effective composite stiffness taking account of the proportionate length of the openings is obtained from Eq. (5.7):

$$I_{y,comp,red} = 2.12\times10^9 - \left(2.12\times10^9 - 1.9\times10^9\right)\times\frac{297}{750} = 2.03\times10^9 \text{ mm}^4$$

This may be compared to the fully composite stiffness for rigid shear connectors:

$$I_{y,comp,red,full} = 3\times10^9 - \left(3\times10^9 - 2.48\times10^9\right)\times\frac{297}{750} = 2.79\times10^9 \text{ mm}^4$$

Therefore, the flexible shear connectors add 37% to the composite beam deflection.

At SLS for a maximum slip of 1.2 mm, and using the relevant modular ratio, the serviceability bending moment is obtained as follows using the cross-sectional area at the centre of the opening:

$$M_{Rd,slip} = \frac{\frac{\pi\times210\times2.03\times10^9}{15000}}{80+0.5\times70+0.5\times665}\times \\ \times\left[1+\frac{25}{210\times240}\left(\frac{15000}{\pi}\right)^2\frac{263\times10^3+10\times10.3\times10^3}{263\times10^3\times10.3\times10^3}\right]\times1.2\times10^{-3} \\ = 605 \text{ kNm}$$

This value is greater than the design value of 527 kNm and shows that the end slip is $\bar{s} = 1.2\text{x}\frac{527}{605}$ = 1.05 mm at the serviceability moment.

Deflections

The imposed load deflection is given by:

$$\delta_i = \frac{5\times5\times3.75\times15^4\times10^9}{384\times210\times2030\times10^6} = 29.0 \text{ mm}$$

This should not exceed the limiting imposed load deflection of span/360

$$\delta_i = 32.5 \text{ mm} \leq \frac{15000}{360} = 42 \text{ mm} \text{ OK}$$

The imposed load deflection is equal to span/515, which is sufficiently stiff.

The effective stiffness of the steel beam taking account of the proportionate length of the openings is:

$$I_{y,comp,red} \quad = \quad 1.07\times10^{9}-\left(1.07\times10^{9}-0.93\times10^{9}\right)\times\frac{297}{750}=1.01\times10^{9}\ \text{mm}^{4}$$

The deflection of the steel beam after construction is obtained from:

$$\delta_{sw}=\frac{5\times2.8\times3.75\times15^{4}\times10^{9}}{384\times210\times1013\times10^{6}}=32.5\ \text{mm}$$

The total deflection of the cellular beam is:

$$\delta_{t}=32.5+29.0=61.5\ \text{mm}>\frac{15000}{250}=60\ \text{mm limit just exceeded}$$

This shows that the total deflection of the cellular beam will control in practice. If this total deflection is considered not to be acceptable, the beam may be pre-cambered by 30 mm to offset the deflection due to the permanent loads. However, this may affect the second cycle of use if the beam is reversed and so it is recommended that the total deflection of 61.5mm is considered to be acceptable when taking account of the nominal 10% fixity of the end connections.

Vibration sensitivity

Check the natural frequency of the composite cellular beam under all permanent loads plus 10% of imposed load. The beam deflection for this load uses the full composite stiffness of the cellular beam:

$$\delta_{f}=\frac{5\times\left(2.8+0.1\times5\right)\times3.75\times15^{4}\times10^{9}}{384\times210\times2790\times10^{6}}=13.9\ \text{mm}$$

This corresponds to a natural frequency of:

$$f=\frac{18}{\sqrt{13.9}}=4.8\ \text{Hz}>4\ \text{Hz}$$

This shows that the vibration sensitivity is acceptable.

Test on a 6 m span composite beam using friction grip bolt shear connectors, as shown in Figure 1.2a, conducted at the University of Luxembourg.

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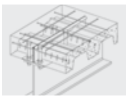

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Demountable composite beam using friction grip bolt connectors

GUIDANCE ON DEMOUNTABLE COMPOSITE CONSTRUCTION SYSTEMS FOR UK PRACTICE

Composite construction is structurally efficient but is difficult to deconstruct and hence reuse the components.

This publication presents guidance and design data on demountable shear connection systems which can enable the steel beams and the potentially floor slabs, to be reused in composite construction. In this way, the benefits of composite construction in the first and subsequent cycles of use are retained.

The guide presents a design procedure and worked examples for composite beams using demountable shear connectors that is based on the principles of Eurocode 4 (BS EN 1994-1-1). The design methodology takes account of the different characteristics of the demountable shear connectors, in terms of their shear resistance, stiffness, and ductility. Design data on the performance of two types of demountable shear connectors, using high-strength structural bolts and coupler systems, are presented.

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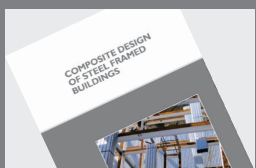

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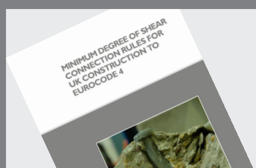

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