DESIGN OF COMPOSITE BEAMS USING LIGHT STEEL SECTIONS

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PROYECTO DE VIGAS MIXTAS EMPLEANDO PERFILES DE ACERO LIGERO

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RESUMEN

La construcción mixta se viene empleando como un método de construcción desde hace décadas, sin embargo tradicionalmente se han utilizado secciones de acero laminado en caliente en lugar de perfiles ligeros (conformados en frío). Los principales componentes de la tradicional construcción mixta han sido las estructuras de acero laminado en caliente, las chapas de acero, los conectores y el hormigón in-situ con armadura pasiva. Las ventajas de este método de construcción son varias entre las que destacan la velocidad de la construcción gracias al rápido montaje de la estructura metálica, la economía en el uso de materiales, su resistencia última y su buen comportamiento en servicio. A estas ventajas pueden añadirse otras más si se emplean secciones de acero ligero en lugar de las tradicionales conformadas en caliente. Las mejoras que el acero ligero incorpora son básicamente su menor coste y peso debido al ahorro de material.

Las estructuras de acero ligero abarcan secciones C, Z o con otras formas similares, de acero galvanizado y conformado en frío, con unos espesores entre 1.2 y 3.2 mm. Trabajos previos han estudiado el comportamiento de estas secciones actuando como vigas o columnas bajo diferentes estados de cargas, pero la presencia conjunta de este tipo de secciones y del hormigón in-situ es un campo que aún no se conoce en profundidad.

En el caso particular de las vigas mixtas de acero ligero se emplean secciones doble C en lugar de perfiles I de acero laminado en caliente, pero la forma general de construcción es similar a la llevada a cabo en la construcción mixta convencional. Es importante resaltar el hecho de que no pueden ser empleados conectadores soldados debido al relativamente pequeño espesor de la sección de acero conformado en frío, y por tanto ha sido necesario desarrollar otras alternativas para los conectadores. Dichos conectadores constan de elementos de acero perfilados, fijados mediante espigas que pueden ser conducidas neumáticamente. Para conocer el comportamiento de estos conectadores se han llevado a cabo algunos ensayos que ofrecen una serie de valores de sus resistencias de cálculo.

Con el objetivo de ofrecer una guía para el dimensionamiento de vigas mixtas empleando perfiles ligeros, se ha llevado a cabo un cuidadoso estudio, desarrollándose ejemplos de cálculo, los cuales se pueden adaptar con facilidad a una condiciones de proyecto determinadas, y tablas para facilitar la rápida selección de un perfil adecuado, en función de la carga, el grado de acero empleado, y la luz a salvar.

Para alcanzar este objetivo global, en primer lugar se ha realizado un análisis de las formas genéricas de la construcción mixta a través de documentación existente, lo cual permite conocer las posibilidades de las losas y vigas mixtas así como las propiedades que son requeridas en los materiales involucrados. Posteriormente se ha centrado el estudio en el caso particular de la construcción mixta usando secciones de acero ligero.

Una vez se conocen la resistencia de los materiales y las dimensiones de los elementos, se estudia el comportamiento de las losas mixtas y de las vigas mixtas. Empleando perfiles ligeros conformados en frío (esbeltos), debido a su pequeño espesor, hay algunas diferencias en el modo de cálculo de la resistencia de la viga mixta. Estas diferencias han sido tenidas en cuenta, y para clarificar como se ha de llevar a cabo el dimensionamiento y la verificación de una viga mixta, dos ejemplos de cálculo han sido desarrollados paso a paso. Uno muestra el caso en el que la viga carece de apoyos provisionales durante la etapa de construcción y otro en el que si los tiene. Siguiendo el mismo procedimiento de los ejemplos, se han realizado diferentes tablas para los casos de empleo de apoyos provisionales y de ausencia de los mismos, variando el grado del acero empleado y la carga variable aplicada. En las tablas resulta complejo obtener una idea general de cuales son las relaciones entre las distintas variables, por eso se presentan unos gráficos que las muestran con claridad. Estas tablas y gráficos pretenden ser útiles en la etapa de diseño facilitando al ingeniero la elección del perfil ligero adecuado en base a las cargas existentes y a la luz requerida.

ABSTRACT

Composite construction is well established for some decades as a construction method but it has traditionally used hot rolled steel sections rather than light steel (cold formed) sections. The main components of traditional composite construction have been hot rolled steel framework, steel decking, shear connectors and in-situ concrete with mesh reinforcing steel. The benefits of this construction method are several and the most important of them are speed of construction due to the rapid erection of the steel framework, economy in use of materials, robustness to damage and good performance in service. To these benefits some more can be added if light steel sections are used instead of the traditional hot rolled. The advantage that light steel gives are basically two: that cold formed steel is cheaper that hot rolled and that is also lighter in weight.

Light steel framing comprises galvanized cold formed steel sections of C or Z or similar forms of 1.2 to 3.2 mm thickness. Previous works have studied how these sections behave as beams or columns under different loads cases, but the composite action of light steel sections with in-situ concrete is a field not yet fully explored.

Composite light steel beams use back to back double C sections rather than hot rolled steel I beams, but the general form of construction is similar to conventional composite construction. Importantly, welded shear connectors cannot be used for the relatively thin steel used in light steel construction, and therefore it has been necessary to develop alternative forms of shear connectors using powder actuated, or pneumatically driven pins. These shear connectors use profiled strip steel elements which are fixed by pins. To know the resistance and behaviour of these innovative shear connectors some tests have been carried out determine the design resistance of these connectors.

The aim of this project is to provide guidance on the design of composite beams using light steel sections, a carefully study has been carried out getting eventually design examples which illustrate the calculus method and can be adapted in a easy way to a design particular characteristic, and design tables to aid rapid selection of light steel sections, depending on the span, the loading and the steel grade used.

To be able to achieve the objectives a progressive work has been carried out. First a literature review on generic forms of composite construction such as composite slabs and beams as well as the types of shear connectors was carried out. The review also types of the materials involved and their properties. With the information already available from previous studies it was possible to get deeper knowledge of the specific shear connectors and beams sections used in composite light steel construction.

Once the materials resistance values and elements dimensions are known, the design of the composite slabs and composite beams was studied. However using cold formed steel sections (slender), the thin thickness of the steel, means that there are some differences in the calculation of the resistance capacity of the composite beam. These differences have been taken into account and to show how the design process of a composite beam works, two design examples have been developed step by step. One is for the case when the beam is not propped during the construction stage and the other one for when the beam is propped. Following this process design tables have been computed for the cases of propped and unpropped and using different steel grades and imposed load. With tables is quite difficult to get a general idea of which relations exist between the different variables, this is the reason why some graphs are presented which show the relations with clarity. The objective of these design tables and graphs are for use during the design stage, making it easier for engineer selection the light steel section base on loading and the span required.

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NOTATION

EC4 Definition

A_a cross-sectional area of the steel section
A_{cv} cross-sectional area of concrete per unit length in any shear plane
A_ε amount of the reinforcement crossing each shear plane
b   beams spacing
b   width of the steel section flange
b_{eff} effective breadth of slab
E   elastic modulus
f_{ck} characteristic strength of concrete or cylinder strength
f_{cu} cube strength of concrete
f_d design tensile strength of steel
f_{sk} yield strength of the reinforcement
f_y yield strength of structural steel
f_{yp} yield strength of the deck
F   action or force
G   permanent loads
h   height of the steel section
h_c height of concrete slab above deck profile
h_p deck profiled height
h_t slab depth
I   second moment of area
L   length of beam, beam span
M_{Rd} design value of moment resistance
M_{Sd} design value of applied moment
n   modular ratio of steel to concrete
N   number of shear connectors
N_f number of shear connectors for full shear connection
PNA   plastic neutral axis
P_{Rd} resistance of a shear connector
Q   variable loads
r   ratio of cross-sectional area of the steel section relative to the concrete section
R_c plastic axial compressive resistance of the slab
R_q longitudinal shear force transfer
R_s plastic axial tensile resistance of the steel section
t_f thickness of the steel section flange
t_w thickness of the steel section web
V   shear force
W   section modulus
γ_G partial safety factor on permanent loads
γ_Q partial safety factor on variable loads
γ partial safety factor on materials
δ deflections
ε $\sqrt{(235/f_y)}$
ρ dry density of concrete
ζ_{Rd} basic shear strength of concrete

The subscripts to the above symbols are as follows:
a   steel
c   concrete
s   reinforcement
p   steel deck
pl   plastic resistance of section
Rd   design value of resistance
Sd   design value of action or force

The member axes in all Eurocodes are:
x   axis along member
y   major axis bending
z   minor axis bending

1.   INTRODUCTION

1.1   BACKGROUND

Composite construction achieves important benefits by making steel and concrete work together, but these advantages can be improved if light cold formed steel sections instead of hot rolled sections are used. The advantages of the light steel composite construction are:

- to get a rapid erection of the steel framework.
- robustness to damage and good performance in service.
- weight and cost of materials are reduced.

In modern composite construction, the steel framing elements are erected first and provide a stable structure that is capable of supporting construction loads. The composite action that developed later with the concrete or other material serves to provide resistance to imposed loads, and importantly, to improve the stiffness of the construction. Often serviceability criteria dominate in modern design and therefore control of deflections and vibration response are as important as load resistance.

After several years of experience on composite construction as a construction method, light steel sections have been introduced in this type of construction. The general principles of composite design using light steel sections are the following:

- During construction light steel beams are designed ELASTICALLY to support the construction loads. Sometimes it is necessary to use single temporary props to control deflections at this stage.
- Once the steel and concrete are acting compositely, composite light steel beams are design PLASTICALLY to support the loads acting at the ultimate limit state. It is possible to carry out plastic design because the steel section acts entirely in tension.
- A minimum degree of shear connection is required. This must be compatible with the deformation capacity of the shear connectors.

Light steel sections introduce some particularities in composite construction. Composite light steel beams use double C sections rather than hot rolled I, and profiled strip steel connectors fixed by powder actuated pins instead of welded shear connectors.

The main objective is therefore to explore innovative composite construction technology where light steel sections act compositely with in-situ concrete. This will lead to increase speed of construction, longer spans, economy of materials and good performance in service, particularly in low and medium-rise buildings (which means buildings with no more than 6 levels).

1.2 OBJECTIVES

The purpose of this study is to find out the possibilities of light cold formed sections in composite construction to clarify in which designs can be considered and make use of all the benefits they incorporate.

To achieve this final objective requires a determinate process:

- Review the general forms of composite construction and the properties of the materials involved.
- Review various opportunities for composite construction using light steel frames and components.
- Knowledge of innovative shear connectors, based on strip steel and powder actuated pins, for use with light steel composite construction.
- Definition and behaviour of a composite slab.
- Design and analyse a typical light steel composite beam.
- Complete work example for light steel sections composite beams propped and unpropped during construction.
- Prepare design tables for light steel composite beams to be used by design engineers.

2. LITERATURE REVIEW ON FORMS OF COMPOSITE CONSTRUCTIONS

2.1 TYPES OF COMPOSITE SLABS

The classification of slabs depends basically on their geometry properties and the construction stage conditions. The common span of composite slabs is between 3m and 4 m, but these values are considering unpropped construction stage. If temporary propping is used it is possible to used longer spans. Considering a shallow decking, slab thicknesses are in the range 100 mm to 200 mm, and for deep decking between 280 mm and 320 mm.

The most common composite slabs consist on the combination of steel decking and in-situ concrete placed onto the steel decking. When the concrete has gained strength, it acts as a composite slab with the tensile strength of the decking. There are different types of in-situ composite slabs considering their dimensions and also the steel decking used.

In the actual building construction pre-cast concrete floors are widely used. There is an increase of composite frames and slim floors construction where the pre-cast slabs are design to interact structurally with the steel frame. In the same way as in-situ composite slabs, the composite action can be developed by welded shear connectors attached to the steel beams and by transverse reinforcement. The most common types of pre-cast slabs are:

- Hollow core units, with continuous circular or elongated openings along their length. The depth is between 150 mm and 260 mm (see Figure 2.1).
- Solid planks, which are designed to use with an in-situ concrete topping. The depth of the solid planks is between 75 mm and 100 mm, but without considering the concrete topping (see Figure 2.2).

There is a wide variety of composite slabs considering all the combinations of composite slabs with in-situ concrete and the wide range of pre-cast slab products available from various manufacturers.

There is a different type of composite slab construction called slim floor. The main difference of this construction is that the supporting beams are contained within the depth of the concrete slab. This is achieved by supporting the slab off the bottom flanges of the beams. The slim floor construction use hot rolled beams together with composite slabs using deep decking. Other forms of slim floor construction, using precast concrete planks to form the slab, are less effective in several aspects, such as ease of service integration (see Figure 2.3).
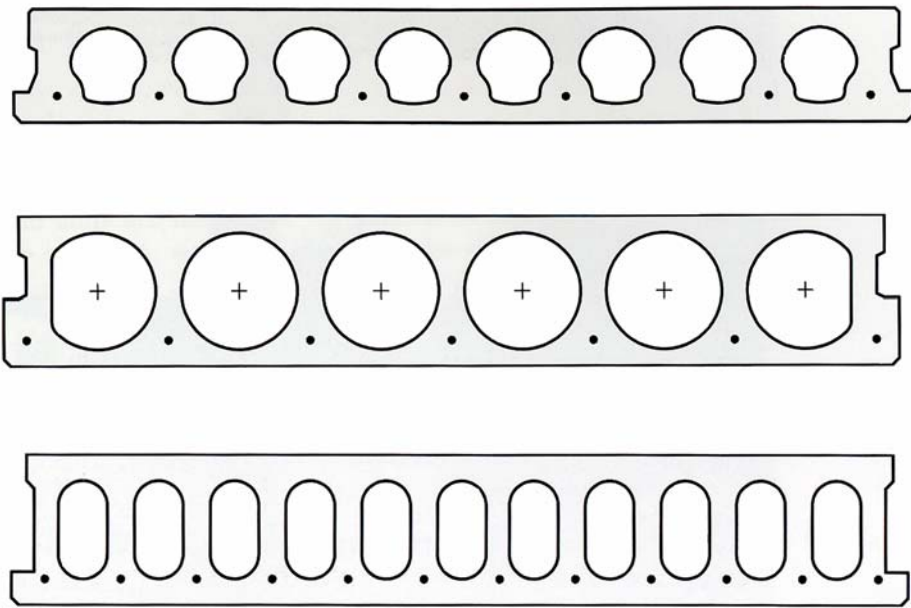
Figure 2.1 Examples of hollow core slabs
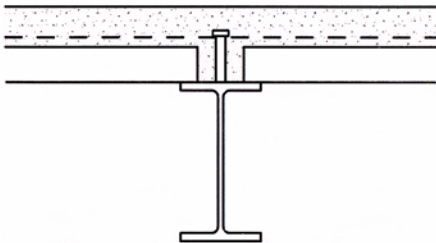
Figure 2.2 Example of solid planks slabs
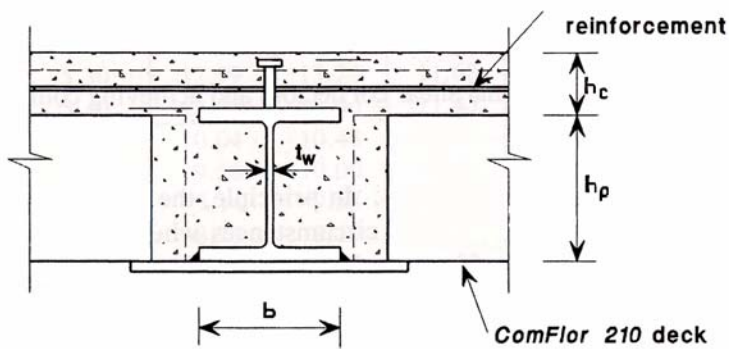
Figure 2.3 Basic components of the "Slimflor" system

2.2   TYPES OF COMPOSITE BEAMS

Composite beams typically consist of steel "I" sections acting structurally with a concrete slab by means of shear connectors attached to the top flange of the steel section. The beams are generally designed to be simply supported, and an effective part of the slab is taken as acting as a part of the composite section on either side of the centreline of the section. It is possible a significant saving in steel weight and/or structural floor depth due to the composite action of the steel beam and the concrete slab which increases the stiffness of the beam and the load capacity.

The size of the steel section used is often chosen by limitations on serviceability considerations. Composite beams tend to be used for long span applications, (in excess 9 m), which means that deflection and dynamic criteria may be critical.

There are two main groups of composite beams in composite construction, primary beams and secondary beams. Primary beams have decking spanning in a direction parallel to them, so the decking do not provide laterally restrain. Secondary beams are perpendicular to the decking spanning direction and the top flange of the steel beam is restrained laterally due to the decking and fixings. This is very important to consider at the design stage to choose the proper steel "I" section.
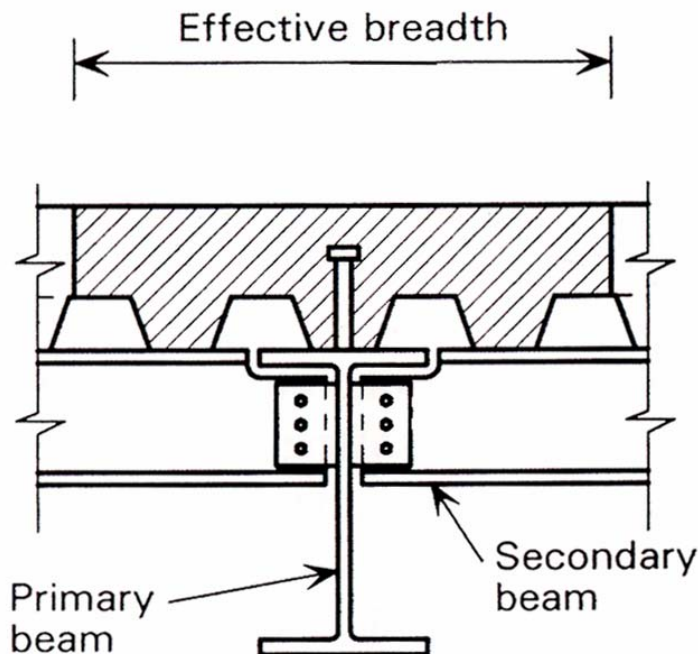
Figure 2.4 Primary and secondary beams

2.3   TYPES OF SHEAR CONNECTORS

There are quite different types of shear connectors, some welded and other nailed connectors. Welded connectors were commonly used in composite construction, but with the development of the use of thinner steel sheets, it has been necessary the use of nailed instead of welded. The choice of a specific type of connector is based on its ultimate resistance which depends not only on its own properties, also on the concrete grade used.

2.3.1 Headed studs

The standard dimensions of headed studs are Ø 19 mm and a length of 125 mm. The behaviour of the headed studs does not vary a lot when concrete properties are changed. Their load capacity is much lower than that of perfobondstrip and T-shape connectors, and it is always around the same value although fibre concrete, light weight concrete or higher strength concrete is used. Headed studs characteristic resistance is lower than that of perfobondstrip and T-shape connectors, and it depends on the number of studs used (see Figure 2.5)

2.3.2 Oscillating perfobondstrip

Oscillating perfobondstrip with a height of 100 mm, a thickness of 8 mm, 5 holes Ø 50 mm and bend in 1.5 wave with an amplitude of 110 mm. Its curve form will give a better force transfer between steel and concrete compared with a straight one.

The load capacity of this type of connector is larger than for example that of the headed studs and T-shape connectors, but in the case of ordinary strength and normal weight concrete, its performance is a bit disappointing due to a fast drop of the load capacity after the peak. But this behaviour disappears if light weight concrete (LC30/37), concrete with fibres or a high strength (C70/85) is used, and in these cases oscillating perfobondstrip connectors behave very well. Oscillating perfobondstrip connectors have also a higher characteristic resistance than others connectors as headed studs and T-shape connectors, and should be taken into account that the failure modes are different for lower and higher concrete strength (see Figure 2.6).

2.3.3 Continuous perfobondstrip

Continuous perfobondstrip with a height of 100 mm, a thickness of 12 mm and 5 holes Ø 50 mm. The evolution in the behaviour of the continuous perfobondstrip, according to the change in concrete properties, is similar to oscillating perfobondstrip, but always archiving lower load capacity and characteristic resistance (see Figure 2.7).

2.3.4 Waveform strip

Waveform strip with a width of 50 mm, a thickness of 6 mm and bend in 2 waves with amplitude 110 mm. Welded with prop-welds Ø 25 mm. It had a very

disappointing behaviour in tests. Due to that its use is not recommended (see Figure 2.8).

2.3.5 T-shape connectors

A section with a length of 300 mm of a standard T-shape 120 welded to the beam. T-shape connectors performed very well compared to headed studs. T-shape connectors achieve the same load capacity as oscillating perfobondstrip, but have a much larger ductility. This type of connectors show an increase in their load capacity and ductility if the concrete used is with fibres, light weight concrete or a higher strength concrete. For the T-shape connectors, after LC62/75, the concrete is no longer decisive, but the strength of the connector itself.

Without considering the perfobondstrip, the T-shape connectors have the highest characteristic resistance and the mode of failure changes for different concrete strengths (see Figure 2.9).

2.3.6 Hilti HVB shear connectors

Hilti HVB shear connectors can be found between the nailed shear connectors which are presented as an alternative to welded headed studs connectors for composite beams. Hilti HVB shear connectors are cold formed angle shear connectors, fixed by two powder-actuated fasteners driven with a powder-actuated tool, placed on one leg of the angle. It is possible to use one, two or three connectors in each steel decking rib, depending on the requirements (see Figure 2.10).

2.3.7 Profiled shear connectors

Profiled shear connectors are a type of nailed strip shear connectors. There are two elements with the same importance in the nailed shear connector system, the strip shear connector itself and the powder-actuated fasteners. The strip is formed by folding a flat zinc-coated steel sheet with a thickness in the range of 1.0 to 2.0 mm. The distance between the two troughs of the trapezoidal strip connector has to be chosen to fit to the geometry of the decking, in composite beams. The legs of the connector have to be higher than the metal deck and they act as a diagonal reinforcement of the concrete rib. The strip connector is fixed to the beam flange by power-actuated fasteners. The connectors are designed to be used with automatic installation systems.

Push-out tests results indicate that optimised profiled strip shear connectors will achieve ultimate resistance in the range of 20 kN per fastener and also sufficient ductility to allow plastic beam design (see Figure 2.11).
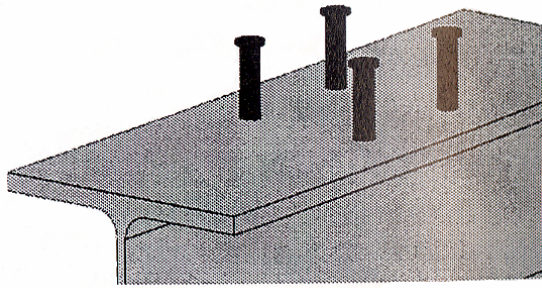
Figure 2.5 Headed studs
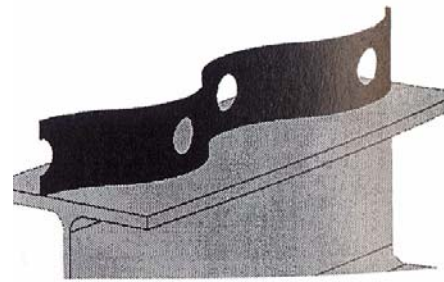
Figure 2.6 Oscillating perfobondstrip
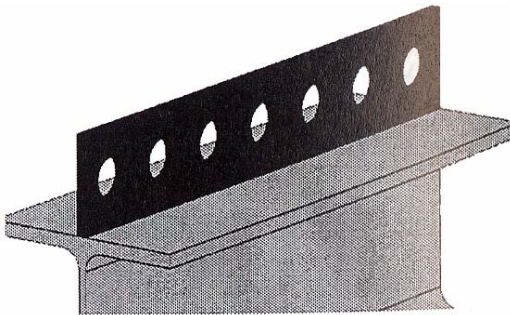
Figure 2.7 Continuous perfobondstrip
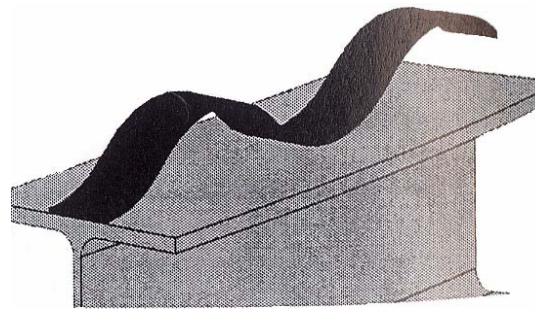
Figure 2.8 Waveform strip
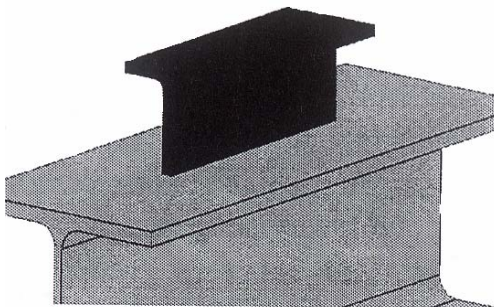
Figure 2.9 T-shape connector

Figure 2.10 Hilti HVB shear connector
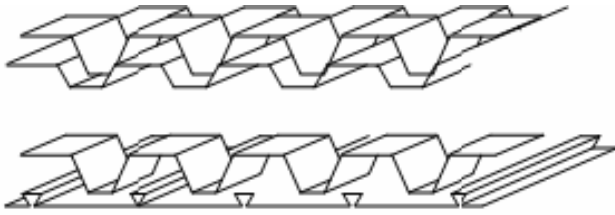
Figure 2.11 Profiled shear connector

2.3.8 Shear connectors' strength and ductility

The results of some push-out tests on shear connector devices for steel-concrete structures are presented by Galjaard and Walraven (2001). The tests are carried out in accordance with Eurocode 4 (1997) for standard push-out tests. Large differences in ductility and strength between the various connector devices and concrete types have been observed.

Table 2.1 Strength of different shear connectors

	Normal weight concrete Concrete grade 30/37		Light weight concrete Concrete grade 30/37	
	Strength P_{Rk} (kN)	Ductility δ_{uk} (mm)	Strength P_{Rk} (kN)	Ductility δ_{uk} (mm)
Headed studs	102	5.0	92	4.9
Continuous perfobondstrip	653	0.8	485	2.9
Oscillating perfobondstrip	974	1.9	858	2.7
Waveform strip	200	1.8	N/A	N/A
T-shape connector	633	16.1	657	38.8
Hilti HVB	28	N/A	25	N/A
Profiled Strip Shear	20	> 6	N/A	N/A

2.4   MATERIAL PROPERTIES

2.4.1 Structural steel

In composite construction design, the two grades of steel that can be used are: S275 and S355. This nomenclature means that minimum yield strength of 275 N/mm^2 and 355 N/mm^2 respectively, is guaranteed.

Table 2.2 Steel properties

Nominal steel grade	Nominal thickness of element, t (mm)			
	t ≤ 40 mm		40 mm < t ≤ 100 mm	
	f_y (N/mm^2)	f_u (N/mm^2)	f_y (N/mm^2)	f_u (N/mm^2)
S275	275	430	255	410
S355	355	510	335	490

2.4.2 Profiled steel decking

The way in which the grades of steel for profiled steel sheeting or decking are specified is in terms of the yield strength of the steel. Yield strength of 280 and 350 N/mm^2 are the common grades for sheet steel used (see 3.4.2).

2.4.3 Concrete (NWC and LWC)

The concrete grade in Eurocode is specified in terms of the cylinder strength, f_{ck}. Hence C30 concrete means that the concrete compression strength is 30 N/mm^2. If cubes were used instead of cylinders, then the resulting strength from testing (f_{cu}) would be different. The approximate conversion to become cylinder strength to cube strength is:

$$f_{ck} \approx 0,8 f_{cu} \tag{2.1}$$

The mean compression strength (f_{ck} and f_{cu}), tensile strength (f_{ct}) and elastic moduli of concrete (E_c) are presented in the following table for various concrete strengths:

Table 2.3 Concrete properties

Properties of concrete	Strength class of concrete						
	C20	C25	C30	C35	C40	C45	C50
f_{ck} (N/mm^2)	20	25	30	35	40	45	50
f_{cu} (N/mm^2)	25	30	37	45	50	55	60
f_{ct} (N/mm^2)	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_c (kN/mm^2)	29	30.5	32	33.5	35	36	37

For both, normal weight concrete (NWC) and light weight concrete (LWC), a concrete grade between C25 and C35 is normally chosen, in accordance with the design requirements. Light weight concrete is commonly used because of the obvious advantages of approximately 25% weight saving (dry densities: NWC density 2350 kg/m^3, LWC density 1850 kg/m^3), and also because LWC has better insulation qualities than NWC.

The concrete grade is normally chosen on the following basis:
- overall structural requirements
- exposure conditions
- flooring to be laid on the slab

As a minimum standard, grade C30 should be specified. In case of concrete used as a wearing surface, the minimum grade should be C35 (although C40 is preferred).

Because the concrete is only exposed on one surface of the composite floor, it can take a longer period of time to cure and dry than a traditional reinforced concrete slab. If moisture sensitive floorings or adhesives are to be applied, many months may be needed before the slab is sufficiently dry to accept them. If the time for drying allowed in the contract programme is inadequate, measures such as the specification of special "quick dry" concrete may need to be considered.

2.4.4 Reinforcement bars

The reinforcement used in composite slabs construction is usually reinforcement which takes the form of a relatively light mesh, commonly supplemented by some bar reinforcement. The mesh reinforcement is required to perform a number of different functions:

- Provide bending resistance at the supports of the slab in the fire condition.
- Control and reduce cracking at the supports. This cracking occurs because of flexural tension and differential shrinkage effects.
- Distribute the effects of localised point loads and line loads along a greater area.
- Increase the strength at the edges of openings.
- Acts as transverse reinforcement to the composite beams

The most common mesh sizes are A142 and A193, where the numbers indicate the cross-sectional area (mm^2) of reinforcing bars per metre width. The mesh is normally manufactured in "sheets" that are 2.4 m wide and 4.8 m long. Mesh with layers of bars equally spaced in both directions is normally used. Mesh sizes less than A142 are not recommended because of their poor performance as fire reinforcement and inability to control shrinkage. But A142 is more common than A193.

Sometimes bar reinforcement may be used to supplement the mesh. With the use of these supplement bars it is possible to achieve some benefits as:

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

In shallow composite slabs, the reinforcement should be supported sufficiently high above the top of the deck to allow concrete placement around the bars. It is necessary to have a proper cover. In practice, for the "mild" exposure conditions that exist internally

in most buildings, this means that the mesh should be placed ideally in a zone between 20 mm and 40 mm from the upper surface of the slab.

2.4.5 Shear connectors

The properties and proportions of shear connectors used depend mainly on the thickness and shape of the steel sheet and the concrete grade used. Normally the steel used to fabric shear connectors is of 500 N/mm^2 ultimate tensile strength.
The characteristic resistances of the shear connector are tabulated in function of their dimensions and the characteristic strength of the concrete used.

2.5   COMPARISON BETWEEN BS5950 AND EC4 PART 1.1

There are quite differences between Eurocode (EC) 4 and British Standard (BS) 5950, not only about design equations, also about materials properties and partial safety factors.

2.5.1 Structural steel

The properties of structural steels are in accordance with Eurocode 4 and BS after the EN 10025 (which replaces BS 4360 for these grades). In this case both of them agree that the two grades of steel that can be used in composite construction like structural steel are S275 and S355.
Since 1993 the nomenclature for strength grades used are the same for both codes, when BS adopt the Eurocode.

2.5.2 Profiled steel decking

The profiled steel sheeting or decking are classified based on the grade of the steel used. Eurocode and BS agree that the yield strength of 280 and 350 N/mm^2 are the most appropriate. In BS these are known as Z28 and Z35 respectively.

2.5.3 Concrete

There is a significant difference in how the concrete grade is specified in Eurocode and in BS. The concrete grade is specified in terms of the cylinder strength, f_{ck} in Eurocodes, instead of the cube strength f_{cu} as in BS. The approximate conversion to be able to change from one of them to another one is:

$$f_{ck} \approx 0,8 f_{cu}$$
(2.2)

Hence C30 concrete based on cylinder strength is 37 N/mm^2 cube strength. The designation of concrete grade is therefore C30/37 defining the cylinder/cube strength.

2.5.4 Reinforcement bars

The most common tensile grade used is 500 N/mm^2. Reinforcement is normally used like mesh. Meshes of around 2.4m x 4.8m are fabric to use. The most common mesh sizes are A142 and A193, where the numbers indicate the cross-sectional area (mm^2) of reinforcing bars per metre width. These values are the same in Eurocode and BS.

2.5.5 Shear connectors

The dimensions and properties of the different shear connectors are defined in Eurocode and also in BS. The commonly ultimate tensile strength used in the fabrication of the connectors is 500 N/mm^2. This is common in both codes, but it has to be taken into account that many forms of shear connectors are permitted and they have different resistances.

2.5.6 Partial safety factors

Partial safety factors are applied to each of the materials involved in the composite construction. Also some factors are applied to the loads and they are different between the Eurocode 4 and BS.

Table 2.4 Partial safety factor according to Eurocode 4 ENV 1994 (1994) [*]

Partial safety factors on:		Limit State		
		Ultimate	Serviceability	Fire
Materials	Structural steel	1.05	1.0	0.9
	Concrete	1.5	1.3	1.0
	Shear connectors	1.25	1.0	1.0
	Shear bond	1.25	1.0	0
	Reinforcement	1.15	1.0	1.0

(*) Nowadays the ENV is in a changing process into the EN

Table 2.5 Loads factors according to Eurocode 4 and BS

Loads factors		Limit state	
		Ultimate	Serviceability
Eurocode 4	Imposed (variable) load	1.5	1.0
	Dead (permanent) load	1.35	1.0
BS	Imposed (variable) load	1.6	1.0
	Dead (permanent) load	1.4	1.0

To illustrate the differences between both codes the following table summaries the main points where EC4 Part 1.1 has a criteria and BS5950 a different one on the design of composite beams.

Table 2.6 Summary of code designs of composite beams

Code/Method	Load factors	Effective breadth	Stress blocks		Headed stud shear connector design resistance (kN)		Minimum shear connection	Serviceability limits	
			Steel	Concrete	NWC[(*)]	LWC[(**)]		Steel	Concrete
BS 5950 Part 3	1.6Q + 1.4G	Span/4 but < 0.8b primary beams	f_y	$0.57f_{ck}$	80	72	0.4 for L < 10 m	f_y	$0.5f_{cu}$
Eurocode 4	1.5Q + 1.35G	Span/4	$0.95f_y$	$0.57f_{ck}$	73	66	0.4 for L < 5 m	None	None

(*) Design value for 19 mm diameter headed studs (100 mm high) in Grade 25 (cylinder) normal weight concrete
(**) Design value for lightweight concrete (density = 1800 kg/m^3)

3. LITERATURE REVIEW ON FORMS OF COMPOSITE CONSTRUCTION USING LIGHT STEEL SECTIONS

3.1 GENERIC FORMS

Composite construction using hot rolled steel elements has been carefully studied and is properly established. But light steel construction comprises cold formed steel sections instead of hot rolled steel sections, and there are quite big differences in their behaviours.

It is important to understand the design principles and practical considerations taken into account, because if not the design of cold rolled sections may appear to be more complicated than the design of hot rolled sections.

One of the most important aspects that have to be considered carefully working with these thin cold formed elements are the methods of cutting, joining and attachment of other members and materials

Nowadays cold formed steel sections are widely used in building applications and decking is used in composite floors and also in flat roofs. Some of the most important advantages related with the used of cold formed elements are:

- For a given section depth, a high load resistance may be achieved
- Long span capability
- Good dimensional accuracy
- Good durability response in internal environments
- Freedom for long term creep and shrinkage
- Lighter constructions

Composite construction using light steel frames and components has different applications:

- Composite beams using double-C sections, also called back to back, with steel decking and in-situ concrete working composite due to strip shear connectors attached by powder actuated pins.
- Composite frames using C and Z sections with steel decking and in-situ concrete in which the framework acts as permanent formwork.
- Vertically orientated steel decking acting with in-situ concrete to form a double skin composite wall.
- Heavy duty flooring acting compositely with light steel floor joists to improve the stiffness of the floor.
- Heavy duty walling acting compositely with light steel wall panels to improve the diaphragm action of the wall.

3.2   TYPES OF LIGHT STEEL COMPOSITE FRAMES, FLOORS, BEAMS AND WALLS

3.2.1   Light steel composite frames

In different places around the world is possible to find cold-formed sections or decking used to provide the formwork to in-situ concrete frames. In France a system of slim floor construction is used in which the Z sections used as secondary beams span between the primary slim floor beams. The secondary beams dimensions might be chosen to match with the slab requirements of the construction (see Figure 3.1).

A wide ribbed U section which acts compositely with in-situ concrete, has been developed in Finland by a steel company. The U section is propped during the construction stage and incorporates composite concrete-filled hollow sections with saddles on which the U section beam is supported. If additional bar reinforcement is placed, no fire protection is required, as in a common T beam (see Figure 3.2).

Other different composite frame which has some disadvantages has been developed in Australia. A profiled decking is used to provide permanent formwork to the sides and soffit of deep beams and large columns. But secondary frames are required to support the decking and to resist concrete pressures, which may be considered a disadvantage (see Figure 3.3).

Composite frames consisting in C and Z sections combined forming primary and secondary beams, and complete frameworks which support in-situ concrete have been developed in New Zealand and Canada. This system achieves excellent seismic resistance by confining the in-situ concrete (see Figure 3.4).
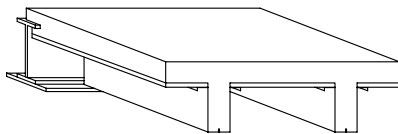
Figure 3.1 Slim floor beam using composite Z sections as secondary beams

Figure 3.2 Finnish system of light steel composite construction
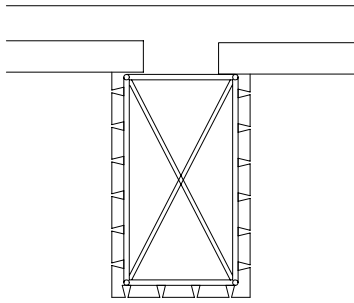
Figure 3.3 Profiled sheeting used as permanent formwork to deep beams and columns
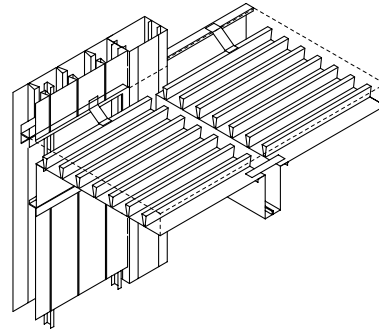
Figure 3.4 Light steel framework as permanent formwork to in-situ concrete

3.2.2 Light steel composite floors

In housing and in low-rise building construction is very common to use light steel floor joints. They comprise C sections which are placed 400 to 600 mm apart. Floor boarding attached to the joints improved their stiffness considerably. Layers of board, plasterboard and suspended ceilings are often used to increase the acoustic insulation of the floor.

A system which provides a very stiff construction comprise profiled decking as part of a composite flooring system in which boarding is fixed by frequent pins and screws. The high stiffness of this construction improves the lateral transfer of loads across the decking (see Figure 3.5)
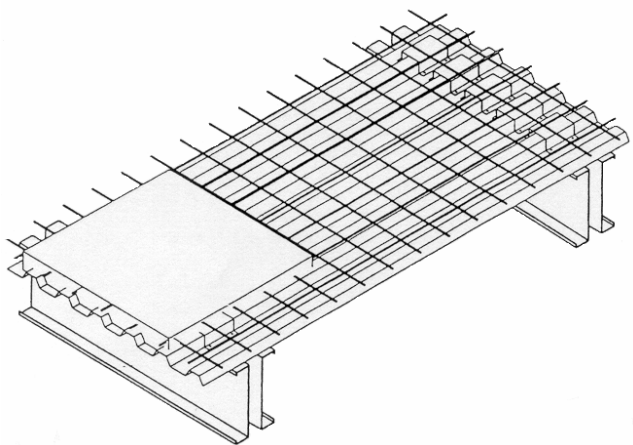
Figure 3.5 Composite floor system

3.2.3 Light steel composite beams

The general form of construction is similar to conventional composite construction, but composite light steel beams do not use hot rolled steel I beams, composite light steel beams use cold formed double C steel sections. Nailed shear connectors are used in light composite beams instead of welded ones. These no welded shear connectors use "top hat" or profiled strip steel elements which are fixed by 2, 3 or 4 pins per deck rib. The "top hat" or profiled shape is designed to support properly the reinforcement, in form of mesh, and to help to provide crack control, especially at the supports. They are designed on these bases because the real form is not very important due to the low force that is transferred locally to the concrete.

The light steel composite beams load-span characteristic may be determined from conventional composite theory. If the span capacity of non-composite and composite beams is compared, only the 60% of the composite beam span can be achieved with non-composite beams. The increase in the stiffness is over 5 times that of the steel section. This is one of the main benefits of composite construction (see Figure 3.6).

The design criteria are:

1. Bending resistance of the steel section under construction loads using elastic properties.
2. Bending resistance of the composite section, taking account of partial shear connection
3. Based on elastic composite properties, imposed load deflections
4. Total deflections, which include the deflections at the construction stage and under imposed loads.
5. Minimum natural frequency (> 4 Hz).
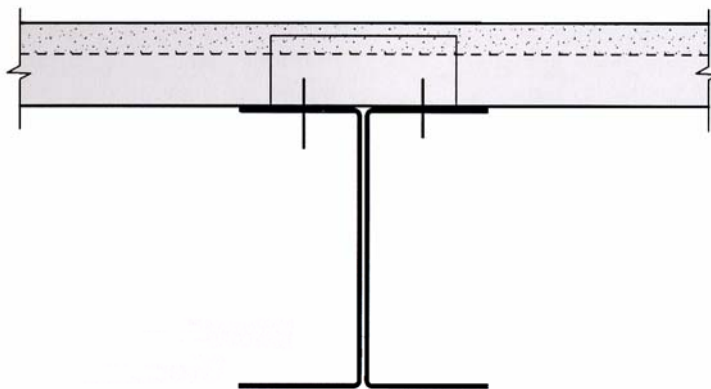
Figure 3.6 Light steel composite beam

3.2.4   Light steel composite wall

Composite walls consist on two skins of decking orientated vertically and in-situ concrete placed between them. Commonly, re-entrant steel decking is used, but also trapezoidal steel decking or inter-locking wall units are used (see Figure 3.7). Ties are placed at approximately 1.0 m apart vertically to resist the concrete pressures during construction. It is difficult to achieve shear transfer at the top of the wall. Because of this the composite wall is more efficient when is slender, so that its composite properties are utilized in resisting buckling at mid-height.

Hence the main advantages of light steel composite walls are the possibility of achieve slender wall construction, a permanent formwork which required no external support and to eliminate bracing acting as a shear wall.

Inter-locking wall units

Composite walls using re-entrant steel decking

Composite walls using trapezoidal steel decking
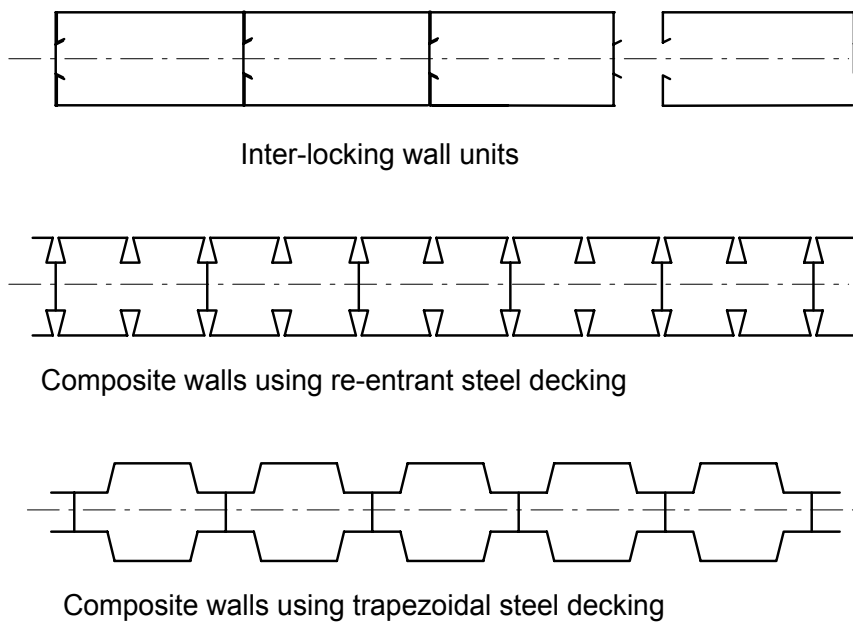
Figure 3.7 Different forms of composite walls using decking and inter-locking units

3.3   TYPES OF SHEAR CONNECTORS

In light steel composite construction welded headed stud connectors cannot be used due to the thin steel decking. As an alternative, nailed strip shear connectors are presented. The most common nailed strip shear connectors are the Hilti HVB shear connectors and profiled shear connectors.

There are some advantages presented by nailed shear connectors compared to welded studs:

- Minimum installation equipment and set-up time.
- They can be used in remote locations and small projects.
- The moisture on site and zinc coatings does not affect the installation quality

3.3.1 Hilti HVB shear connectors

Hilti HVB shear connectors are one of the types of nailed strips shear connector used in light steel composite construction. Hilti HVB shear connectors are cold formed angle shear connectors, fixed by two powder-actuated fasteners placed on one leg of the angle (see Figure 2.8).

The ultimate resistance of Hilti HVB connectors depends on the dimensions of the connector, because there are different sizes of Hilti HVB shear connectors, and on the concrete grade.

Table 3.1 Design resistance of Hilti X-HVB connectors with a concrete grade between C25 and C50

Hilti X-HVB Type	Design Resistance P_{Rd} (kN)	
	NWC	LWC
80	18	16
95	28	25
110	28	25
125	30	25
140	30	25

3.3.2 Profiled shear connectors

Profiled shear connectors have a trapezoidal shape to fit the geometry of metal decks. The profiled shear connectors are designed to be able to fix them with an automatic system to the decking and the beam by powder-actuated fasteners (Hilti), this makes the installation a fast process.

Push-out test has been carried out according to the specifications provided in Eurocode 4 (1992) except for the fact that strip connectors were used instead of headed studs. The push-out test programme and the description of the load-deformation behaviour were written by Fontana, Beck and Bärtschi (2001). Results of the push-out test show that a profiled shear connector will achieve ultimate resistance above 20 kN per fastener. And the test also shows that the connectors will achieve sufficient ductility to allow plastic beam design.

Nailed shear connectors consist on two different elements with the same importance, the strip shear connector itself and the powder-actuated fasteners. The profiled strip is formed by folding a flat zinc-coated steel sheet with a thickness in the range of 1.0 to 2.0 mm. The way on which the steel sheet is folded depends on the geometry that the profiled shear connector needs to have to fit properly. The legs of the connector have to be higher than the metal deck and they act as a diagonal reinforcement of the concrete rib.
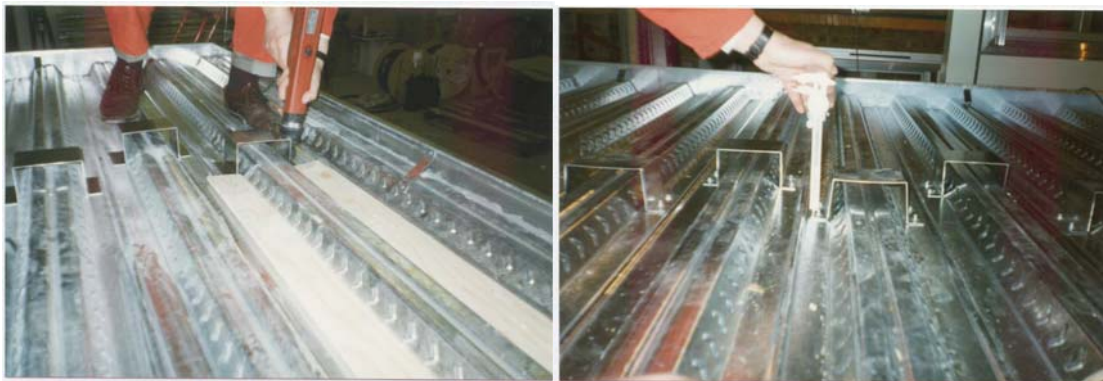
Figure 3.8 Nailed profiled shear connectors

Possible failure mechanism of profiled strip shear connectors

There are three different types of modes of failure depending on where the failure occurs:

1. Failure in the nailed interface: the failure can take place in the interface due to two different situations, shear failure of the nail shank or pullout of nails combined with local bearing deformations in the flange.

2. Failure of the steel of the strip connector: the steel of the strip can fail in three locations:
   a. Net section fracture in the tension leg
   b. Net section fracture in the nailed troughs
   c. Local bearing failure in the nail interface

3. Concrete failure: bearing failure or shear failure of the concrete dowels and shear failure of the concrete rib can occur. Concrete can also fail by splitting of the concrete plate.

To allow plastic design of the composite beam the strip shear connector must develop sufficient plastic deformations at a high load level. But to get a good efficiency of a

profiled shear connector system, it is important to develop an ultimate capacity close to the total nail shear capacity of all fasteners installed. Edge the thickness of the sheet must not be too thick, and on this way a brittle nail shank fracture without local bearing deformations can be avoid. But opposite to this there are some reasons by which a stiff strip connector sheet is required:

- Smaller shear connecting contribution of the open trapezoidal metal deck
- Greater stress in the tension legs as four nails are fixed per trough
- Existence of the ridge in the trough with two nails on both sides of the ridge.

3.4   MATERIAL PROPERTIES

3.4.1   Light steel sections

The thickness of the sheet steel used in cold formed sections, light steel sections, is typically 0.9 to 3.2 mm. The method used to protect the steel of the corrosion is with hot dip galvanizing of preformed strip steel. It protects the steel by loss of the zinc surface.

The specified sheet thickness includes galvanizing. The standard specification for internal environments is G275, which refers to a zinc coating of 275 g/m^2, total on both faces and it means a total zinc thickness of about 0.04 mm (0.02 mm per face). A thicker coating is used where moisture may be present over a long period (G600).Galvanizing gives adequate protection for internal members, also for those adjacent to the boundaries of building envelopes, hence it is usually supplied pre-galvanized. The expected design life of galvanized products in this environment exceeds 60 years.

Zinc-rich paint has to be applied to the exposed steel in some applications. White rust may occur if galvanized sections are stored in moist conditions, but this does not normally affect their long term performance.

By cold reduction of hot rolled coil steel, steel strip is produced. Later it is treated with annealing processes to improve the ductility of the material, due to cold working reduces the ductility of the material. Grades S280 and S350 (yield strength of 280 and 350 N/mm^2, respectively), are the most common recommended grades.
With the cold forming process the yield strength increases. This is due to cold working by the process of "strain hardening". The yield point is not a clearly defined transition point, as in hot rolled steels. The 0.2% strain proof strength is commonly used as an effective yield value.

Light steel sections are shaped to suit particular applications. The basic shapes, especially in building applications, are the C and Z sections, but there is a wide range of variants of these basic shapes, like those with edge lips, internal stiffeners and bends in the webs. Also are common the sections form as modified I section and the top-hat section. The sections are also joined together back to back or toe to toe to form compound sections as double C sections.

Z sections

Lipped Z   Zeta

C sections

Plain   Lipped   Sigma

Special sections

Top hat   Modified *I* sections   Eaves beam

Compound sections
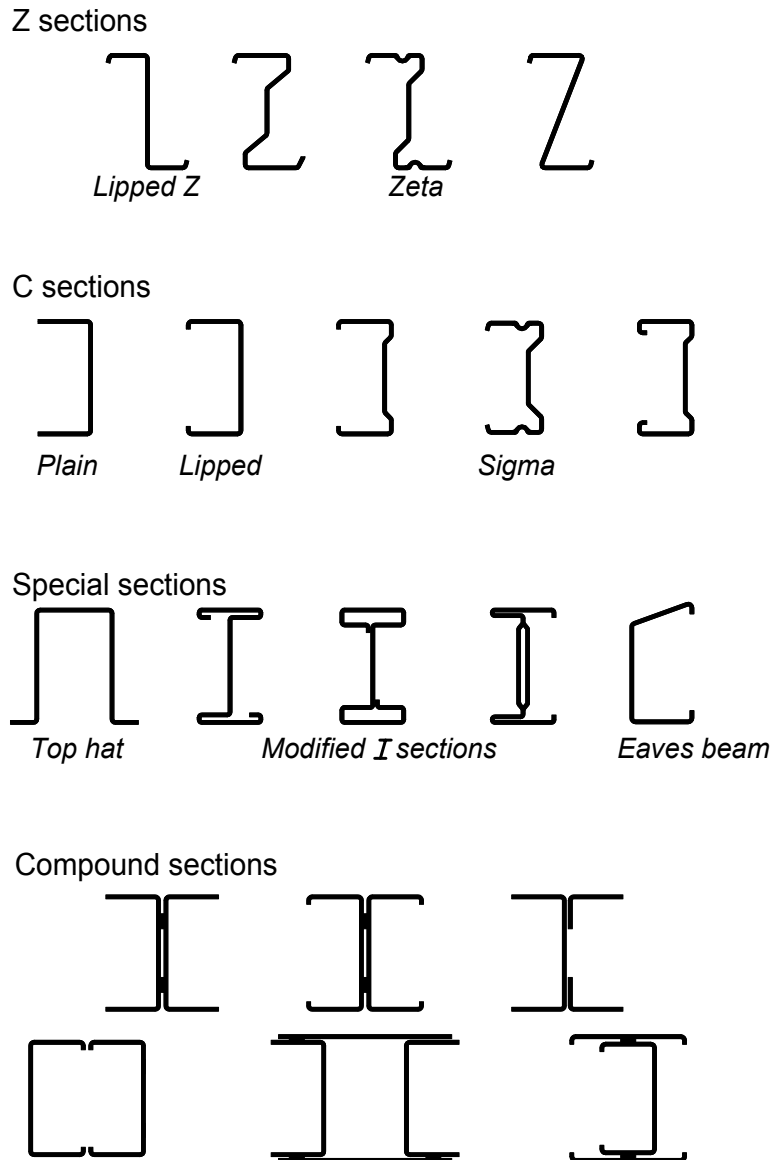
Figure 3.9 Examples of cold formed steel sections

Unstiffened wide and thin plates are not able to resist significant compression, this the reason to used variants of the basic sections shapes with edge lips and internal stiffeners. Stiffened sections are required to a structural efficiency, but in the other hand as more stiff is a section, more difficult is to form it and more difficult to connect it with other elements. It is necessary to find the equilibrium between practicability and structural efficiency.

The main difference between the behaviour of hot rolled and cold formed steel sections is that thin plate elements of cold formed sections tend to buckle locally under compression. Cold formed cross-sections cannot generally reach their full compression resistance based on the amount of material in the cross-section; this is reason why they used to be classified as slender cross sections.

There are also some benefits of cold forming on material properties as the increase in average yield strength. This increase in strength is typically 3 to 10%.

Geometrical properties of cold rolled sections are divided in two big groups, the gross section properties and the reduced section properties which take into account the effect of local buckling. These two different properties are used in different cases. The gross section properties are used to calculate the elastic stiffness of members or the moments in continuous structures. However reduced properties are used to calculate the load capability of the sections. Light steel section properties have been tabulated by Lawson, Chung and Popo-Ola (2002).

3.4.2 Profiled steel decking

Profiled steel decking dimensions are in range of 45 to 80 mm height and 150 to 300 mm trough spacing, (rib spacing). This type of decking typically spans until 3 m or 4.5 m. There are two well known types of decking profiles, the re-entrant profile and the trapezoidal profile. These two types are the most common (see Figure 3.10 and Figure 3.11).

Decking is generally rolled from 0.9 to 1.5 mm thick strip steel. Steel grades for this application are typically S280 or S350 (steel yield strengths of 280 or 350N/mm^2). The steel is galvanized before forming, in the same way as light steel sections.

In composite slabs the steel decking has two main structural functions:

1. During concreting, the decking supports the weight of the wet concrete and reinforcement, together with the temporary loads associated with the construction process. It is normally designed to be used without temporary propping.
2. In service, the decking acts "compositely" with the concrete to support the loads on the floor. Composite action is obtained by shear bond and mechanical interlock between the concrete and the decking.

The decking has also other important functions. The decking may also be used to stabilise the beams against lateral torsional buckling during construction, and to stabilise the building as a whole by acting as a diaphragm to transfer wind loads to the walls and columns. The decking, together with the fabric mesh reinforcement placed in the top of the slab, also helps to control cracking of the concrete caused by shrinkage effects.
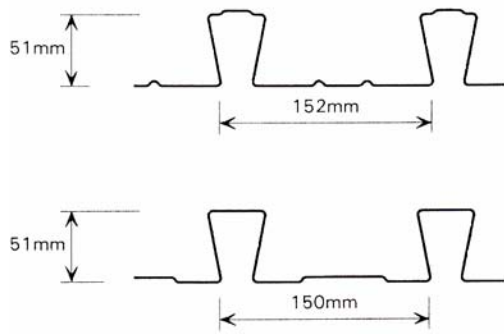
Figure 3.10 Re-entrant profiled steel decking
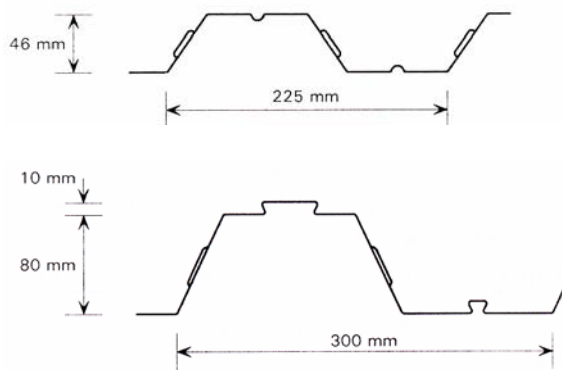
Figure 3.11 Trapezoidal profiled steel decking

4. BASIS OF DESIGN OF COMPOSITE SLABS

4.1 DEFINITION

Composite slabs construction comprises two different elements, steel decking or sheeting, and in-situ concrete. The steel decking is considered as the permanent formwork to the in-situ concrete.

The most efficient use of composite slabs is for spans between 3 and 4 m, and the most common is 3m. The ability of the decking to support the construction loads, without the need for temporary propping, normally dictates these spans. If props are used, longer spans are possible. Also some of the deeper profiles can achieve spans of up to 4.5 m without propping during construction. The maximum span to depth ratio for the deck will normally be 60.

The slab depths largely depend on fire insulation requirements and are usually between 100 and 200 mm. It depends on the time of fire resistant expected. To choose the concrete type it has to take into account that it affects the stiffness of the section and the strength of the shear connectors. Normal weight concrete (NWC) and light weight concrete (LWC) are both used. When the concrete has gained sufficient strength it acts as a composite slab with the tensile strength of the decking. A light mesh of reinforcement is placed in the concrete to reduce the severity of cracking and to increase the fire resistance.
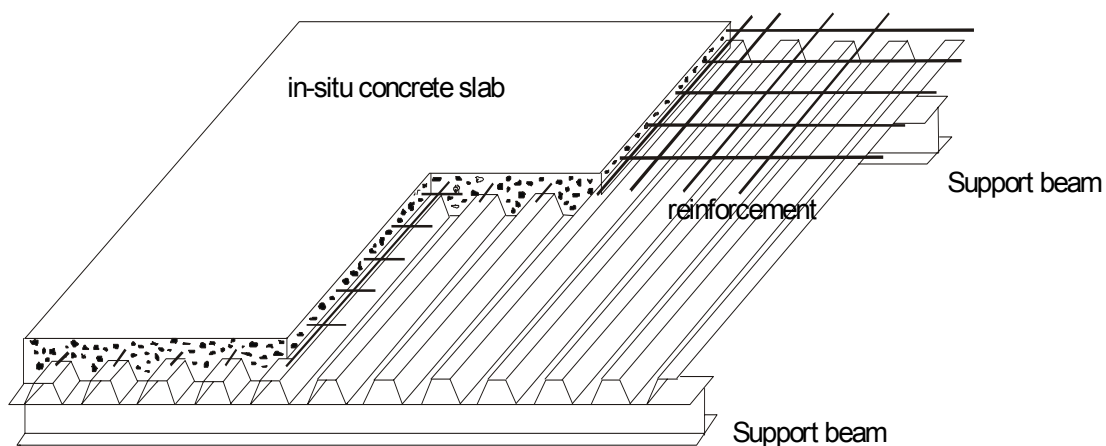
Figure 4.1 Composite slab

4.2 CONSTRUCTION STAGE CONDITION

If the slab is unpropped during construction, the decking alone has to resist the self-weight and the construction loads. In this case subsequent loads are applied to the

composite section. If the slab is propped during construction, all of the loads have to be resisted by the composite section. This can lead to a reduction in the imposed load that the slab can support, due to the increase of the shear in the interface between the concrete and the decking.

The construction load, taken to act in addition to the self-weight of the slab and beam, for the decking design is:

- An intensity of load of 1.5 kN/m^2 acting over a plan area of 3m × 3m
- Elsewhere, a reduced load of 0.5 kN/m^2

The construction loads take account of the sequential nature of the concreting and the operations on the decking. Therefore, design cases to be considered are:

1. single span loaded to 1.5 kN/m^2 plus self-weight; adjacent spans loaded to 0.5 kN/ m^2 plus self-weight, or
2. single span loaded to 1.5 kN/ m^2 plus self-weight; adjacent spans not loaded.

Case 1 is maximum elastic moment at the supports, and 2 correspond to the maximum elastic moment in mid-span.

The design of continuous decking is based on an elastic distribution of moments for the construction loads. The elastic moment resistance of the section is established taking account of the effective breadth of the thin steel elements in compression. No moment redistribution is allowed and the common critical conditions would be the negative moment at the supports.

4.3   COMPOSITE STAGE CONDITION

The most common mode of failure of the composite slab is due to the breakdown of shear bond. The ultimate moment resistance of composite slabs is determined by the breakdown of bond and mechanical interlock between the decking and the concrete, known as shear bond. Composite slabs are usually designed as simply supported members, and the slip between the decking and the concrete usually occurs before the plastic moment resistance of the composite section is reached.

Eurocode 4 permits the design of composite slabs as continuous slabs by placing reinforcement in the negative moment region. There are two methods of design of composite slabs permitted by Eurocode 4. The traditionally used which is called "m" and "k" method, and an alternative method based on the principles of partial shear connection.

The performance of a particular deck profiled in a composite slab can only be well assessed by test. Design by testing consists of two main parts with different purposes.

The test is carried out in two states:

- A dynamic part to identify those cases where there is an inherently brittle bond between the concrete and the steel. (10,000 load cycles up to 1.5 times the working load).
- Following the dynamic part, a static load is applied and increased until failure occurs.

The test results are then presented in terms of empirical constants (m and k) that can be used to quantify the interaction between the steel and concrete. The constants, m and k, are not normally provided by the decking manufacturers, they use this information themselves to present to the designers a range of load-span tables for their specific products.

For the alternative method in the EC4, tests are required too. A characteristic longitudinal shear resistance is defined based on tests. Then this resistance is used in a modified partial shear connection analysis.

The design for serviceability is based on deflection limits but no deflection limits are specified in Eurocode 4 for the deflection of the deck after concreting. As a further check, it is recommended that the increased weight of concrete due to ponding should be included in the design of the support structure if the predicted deflection, without including the effect of ponding, is greater than one tenth of the overall slab depth.

The use of simple design rules to ensure adequate deflection behaviour of a composite slab is accepted practice. Limits for span-to-depth ratios for slabs using NWC and LWC are given. Confirming that the slab satisfies these limits will ensure that excessive deflections are avoided. The effective span of the decking is defined as the smaller of:

- The distance between the centres of the supports.
- The clear span between the supports plus the effective depth of the concrete slab.

The values in the table 4.1 apply to slabs under uniformly distributed loading, with nominal continuity reinforcement (0.1%) over the intermediate supports, and not for slabs with full continuity reinforcement over the supports.

Deflections should be calculated explicitly for slabs that fail to satisfy span-to-depth ratio limits. The stiffness of the slab can be determined using normal reinforced concrete design rules (assuming fully effective bond between the decking and the concrete).

Table 4.1 General rules for the slab: maximum span-to-depth ratios[(*)]

	NWC	LWC
Single spans	30	25
End spans	35	30
Internal spans	38	33

(*)Values apply to supported spans with nominal continuity reinforcement, subject to uniformly distributed loading.

There is always a risk of cracking in the concrete in all composite slabs. This is due to the restraint to drying shrinkage provided by the steel decking and primary steelwork, even though the decking effectively acts as reinforcement and helps to distribute the shrinkage strains so that large cracks do not form. However, cracks do not normally mean a durability or serviceability hazard. Only when the surface of the slab is used as a wearing surface, or where terrazzo or other "rigid" floor covering are to be used, may specific reinforcement be required in order to control the cracking.

When cracking is an issue, reinforcement percentages in excess of 0.3% will normally be required in order to limit crack widths to the typically specified limit of approximately 0.3 mm. In composite slabs mesh, rather than bars, is generally used to control cracking.

The greatest risk of cracking is normally over supporting beams, owing to the combination of drying shrinkage and flexural action. Induced joints may be used to reduce the risk of random cracking at these locations. With the induced joints it is possible to control where the crack will be form.

It is possible that larger crack widths over the intermediate supports will occur with propped construction, because the full self-weight of the slab is applied in the composite slab on removal of the props. Reinforcement of 0.5% of the slab cross-sectional area should be sufficient in these cases to control cracking.

4.4   FIRE RESISTANCE

The measure to improve the fire resistance is to increase the reinforcement. To satisfy requirements for the fire condition, an increased size of mesh may need to be used, or extra bars may need to be placed in the troughs of the deck. In either case, the additional reinforcement is used to compensate for the loss of strength of the decking at elevated temperatures. A design guidance covering this aspect is normally given by the decking manufacturers in their design tables. However, established guidance on minimum slab thicknesses, minimum decking thickness, and the corresponding mesh sizes, for particular fire resistance periods is given in the following tables for trapezoidal decking and for re-entrant decking.

The data in table 4.2 may be used directly for trapezoidal decking profiles of 45 to 60 mm nominal depth (h_p is the differential height between the thin and the most depth

parts of the decking). For decking profiles greater than 60 mm deep, slab depths given in the table should be increased by (h_p – 60) mm. For decking profiles that are between 45 and 55 mm deep, when spans are not greater than 3 m, the specified minimum slab depths may be reduced by (55 – h_p) mm.
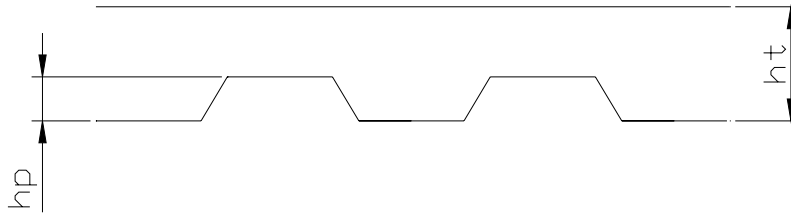
Figure 4.2 Composite slab cross section

Table 4.2 Fire resistance specifications for trapezoidal decking

| Maximum Span (m) | Fire resistance (hours) | Minimum dimensions | | | Mesh Size |
| | | Deck thickness t (mm) | Slab thickness h_t (mm) | | |
			NWC	LWC	
2.7	1	0.8	130	120	A142
3.0	1	0.9	130	120	A142
3.0	1 1/2	0.9	140	130	A142
3.0	2	0.9	155	140	A193
3.6	1	1.0	130	120	A193
3.6	1 1/2	1.2	140	130	A193
3.6	2	1.2	155	140	A252

The data in table 4.3 applies to re-entrant decking profiles of 38 to 50 mm nominal depth. For profiles greater than 50 mm deep, the specified minimum slab depth should be increased by (h_p – 50) mm.

Table 4.3 Fire resistance specification for re-entrant decking

| Maximum Span (m) | Fire resistance (hours) | Minimum dimensions | | | Mesh Size |
| | | Deck thickness t (mm) | Slab thickness h_t (mm) | | |
			NWC	LWC	
2.5	1	0.8	100	100	A142
2.5	1 1/2	0.8	110	105	A142
3.0	1	0.9	120	110	A142
3.0	1 1/2	0.9	130	120	A142
3.0	2	0.9	140	130	A193
3.6	1	1.0	125	120	A193
3.6	1 1/2	1.2	135	125	A193
3.6	2	1.2	145	130	A252

4.5   EXAMPLE GUIDE

In composite slabs there are three possible modes of behaviour based on the level of interaction between the concrete and the steel decking:

— Complete interaction
— Partial interaction
— Zero interaction

And there are also three likely collapse mechanisms depending on the characteristics of the slab:

• Failure type 1: applied moment exceeds moment resistance.
• Failure type 2: ultimate load resistance is governed by the steel concrete interface.
• Failure type 3: applied vertical shear exceeds shear resistance.

EXERCICE

1. **Check metal deck during construction**

   Consider the self-weight of the deck and the wet concrete, and these loads have to be over all the deck. Consider the construction loads and distribute them to have the more unfavourable situation for both, maximum sagging bending moment, and maximum hogging bending moment (positive and negative moments). They have to be less than the moment resistance of the deck, $M_{p.Rd}^{+}$ and $M_{p.Rd}^{-}$.

2. **Check deck deflection in construction**

$$\delta = k \times \frac{5}{384} \times p \times L^4 \times \frac{1}{E \times I_{eff}}$$ (4.1)

   k=1.0 for simply supported decking
   k=0.41 with two equal spans (3 supports)
   k=0.52 with three equal spans
   k=0.49 with four equal spans
   I_{eff} is the second moment of area of the effective section

   Limit: L/180 or 20 mm (4.2)

3. **Check design of composite slab – at ULS**

   Assume that slab acts as a series of simply supported beams. The moment resistance of the slab has to be greater than the applied moment.

Design bending moment: $M_{Sd} = \dfrac{[\gamma_G \times G + \gamma_Q \times Q] \times L^2}{8}$ (4.3)

Position of plastic neutral axis: X

$$X = \frac{A_p \times f_{yp} / \gamma_{ap}}{0.85 \times B \times f_{ck} / \gamma_c}$$ (4.4)

where
A_p is the area of the deck (mm^2/m)
B width took as 1000 mm
f_{yp} is the tensile strength of the deck
$\gamma_{ap} = 1.10$ is the partial safety factor of the deck

$z = d_p - 0.5 \times X$ (4.5)

where
d_p is the total depth of the slab without half of the deck height plus the deck thickness.

Moment resistance of the slab:

$$M_{ps,Rd} = A_p \times \frac{f_{yp}}{\gamma_{ap}} \times z$$ (4.6)

4. Check longitudinal shear (m-k method)

Design shear force: $V_{sd} = \dfrac{[\gamma_G \times G + \gamma_Q \times Q] \times L}{2}$ (4.7)

Longitudinal resistance is:

$$v_{L,Rd} = B \times d_p \times \left(m \times \frac{A_p}{B \times L_s} + k \right) \times \frac{1}{\gamma_{vs}}$$ (4.8)

The m-k method is semi-empirical. Direct relationship is established with the longitudinal shear load capacity of the sheeting. L_s depend on the type of loading. Uniform load applied to the entire span L simply supported beam, $L_s = L/4$
$\gamma_{vs} = 1.25$ is the partial safety factor of longitudinal shear

5. Check vertical shear

Design shear force: $V_{sd} = \dfrac{\left[\gamma_G \times G + \gamma_Q \times Q\right] \times L}{2}$ (4.9)

Vertical shear resistance is:

$V_{v.Rd} = b_o \times d_p \times k_1 \times k_2 \times \tau_{Rd}$ (4.10)

where
b_o is the average concrete rib width (over 1 m)

$k_1 = 1.6 - d_p$ (4.11)

$k_2 = 1.2 + 40 \times \rho$ (4.12)

$\rho = \dfrac{A_p}{b_o \times d_p}$ (4.13)

$\tau_{Rd} = 0.25 \dfrac{f_{ck}}{\gamma_c}$ (4.14)

6. Serviceability limit state

Calculate the deflections with the average second moment of composite slab. Deflections would not be design criteria for slabs that satisfy span-to-depth ratio limits.

5. BASIS OF DESIGN OF COMPOSITE BEAMS

Composite beams consist in a concrete flange and a steel section. The concrete flange is in compression and the steel section is largely in tension. Shear connectors make possible to transfer the forces between the two materials. Hence the roll that shear connectors play is very important, because with them it is possible to have a composite behaviour.
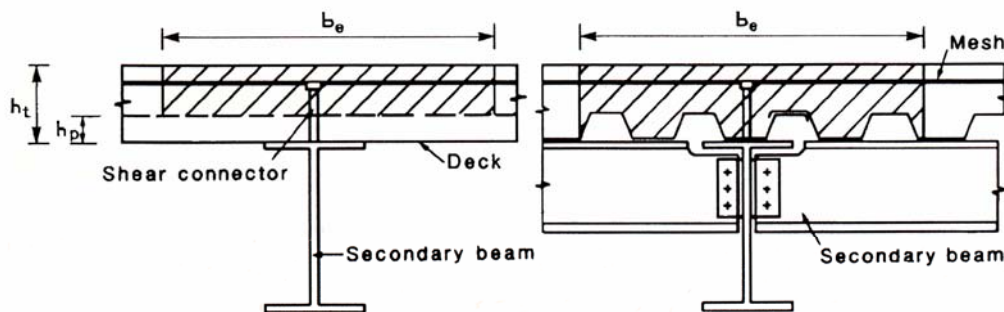
Figure 5.1 Example of composite beams

5.1   CONSTRUCTION CONDITION

First of all the composite beam has to be design to support the construction conditions. In unpropped construction, the steel beam is sized to support the self weight of the concrete slab and other construction loads. The difference presented in propped construction at this stage, is that the span of the beam considered is the distance between two consecutive supports which can be props or edge supports, and not the total span of the beam.

A load of 0.5 kN/m² is assumed to be applied to the entire area of steel decking and it would be logical to take this same load as applied to the beam. It is treated as an imposed load.

The steel beam is then designed in accordance with Eurocode 3. Secondary beams, that are the beams perpendicular to the decking and the ones will be covered by this study, are laterally restrained by the steel decking, and can develop their full moment resistances. In primary beams the lateral restraint is provided only by the beam to beam connections and their buckling resistance is based on the effective length of the beam between these points.

5.2   EFFECTIVE SLAB WIDTH

Not the whole slab is considered for the design; there is an effective breadth of slab. For compatibility between designs at ULS and SLS the effective breadth is taken as L/8 on each side of the secondary beam, being L the span length. This results in L/4, but not exceeding the actual slab width acting with each beam. Effective breadth is represented by b_{eff}.

5.3   PLASTIC ANALYSIS OF COMPOSITE ACTION

Once the steel and concrete are acting compositely, beams are designed plastically. Plastic stress blocks are rectangular, unlike elastic stress blocks which are triangular, and the plastic moment resistance is independent of the sequence of loading (i.e. propped or unpropped construction). Like it is independent to the sequence of loading, the moment resistance of the section is compared to the total factored moment applied to the beam.

Materials strengths to be used in the plastic analysis are:

Concrete: 0.85 f_{ck}/γ_c (γ_c=1.5)   (5.1)

0.57f_{ck} or 0.45f_{cu} (5.2)

$f_{ck} \approx 0.8\ f_{cu}$ (5.3)

Steel: f_y/γ_a (γ_a= 1.05)   (5.4)

0.95f_y (5.5)

Compressive resistance of the concrete slab is therefore:
$$R_c = \frac{0,85 f_{ck}}{\gamma_c} \times b_{eff} \times h_c$$ (5.6)
$$R_c = 0.57 \times f_{ck} \times b_{eff} \times h_c$$ (5.7)
where
h_c is the depth of the concrete slab above the profiled decking

Tensile resistance of the steel section is:

$$R_s = \frac{A_a \times f_y}{\gamma_a}$$ (5.8)
$$R_s = 0.95 \times f_y \times A_a$$ (5.9)
where
A_a is the area of the steel beam

Moment resistance: $M_{pl.Rd}$

a)   **Plastic neutral axis (PNA) in concrete slab: $R_c \geq R_s$**
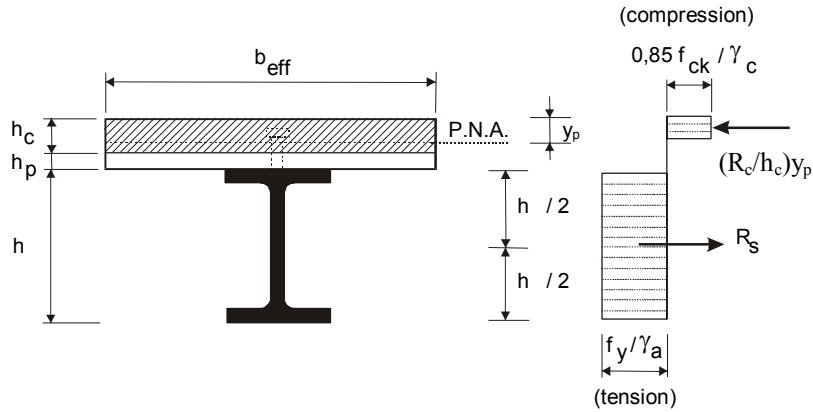
Figure 5.2 Plastic stress blocks when PNA lies in concrete slab

$$M_{pl,Rd} = R_s \left[\frac{h}{2} + h_c + h_p - \frac{R_s}{R_c} \times \frac{h_c}{2} \right] \tag{5.10}$$

$$y_p = \frac{R_s \times h_c}{R_c} \quad \text{from} \quad \frac{R_c}{h_c} \times y_p = R_s \tag{5.11}$$

where
h_c is the height of concrete slab above the deck
h_p is the depth of the profiled decking
h is the depth of the steel section
y_p is the depth of PNA since the upper surface of the slab

b)   **Plastic neutral axis in flange of steel beam: $R_c \leq R_s$ and $R_c > R_w$**
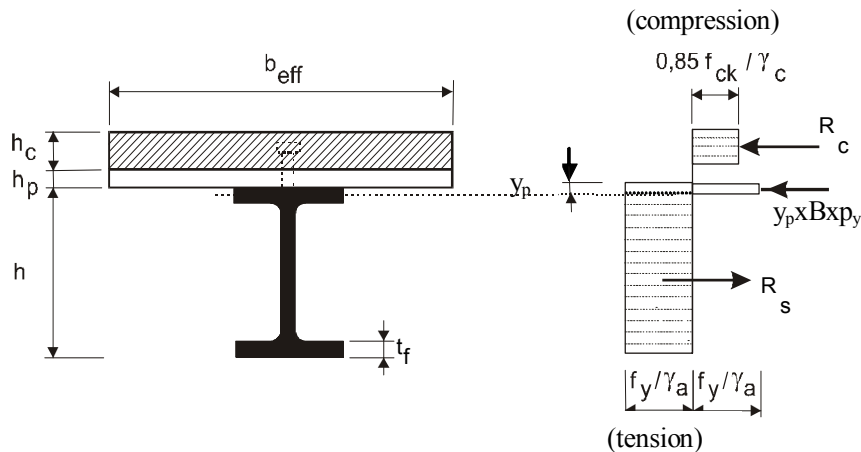
Figure 5.3 Plastic stress blocks when PNA lies in flange of steel beam

R_w is the tensile resistance of the web of beam:

$$R_w = 0.95 \times f_y \times t_w \times \left(h - 2t_f\right) \tag{5.12}$$

where
t_w is the web thickness
t_f is the flange thickness

The depth of web in compression should not exceed $38t_w\varepsilon$ to be treated as "Class 2".

where

$$\varepsilon = \sqrt{\left(\frac{235}{f_y}\right)} \tag{5.13}$$

y_p is the part of the steel flange which is in compression:

$$2 \times B \times p_y \times y_p = R_s - R_c \quad \rightarrow \quad y_p = \frac{R_s - R_c}{2 \times B \times p_y} \tag{5.14}$$

And the compression force in the steel flange is:

$$y_p \times B \times p_y = \frac{R_s - R_c}{2} \tag{5.15}$$

where

$$p_y = \frac{f_y}{\gamma_a} \tag{5.16}$$

Moment respect the upper fibre of the top flange:

$$M_{pl,Rd} = R_c\left(\frac{h_c}{2} + h_p\right) + R_s\,\frac{h}{2} - \left[2 \times \frac{R_s - R_c}{2} \times \frac{R_s - R_c}{2Bp_y} \times \frac{1}{2}\right] \tag{5.17}$$

$$M_{pl,Rd} = R_c\left(\frac{h_c}{2} + h_p\right) + R_s\,\frac{h}{2} - \frac{(R_s - R_c)^2}{4Bp_y} \tag{5.18}$$

c) **Plastic neutral axis within web: $R_c \leq R_s$ and $R_c < R_w$**
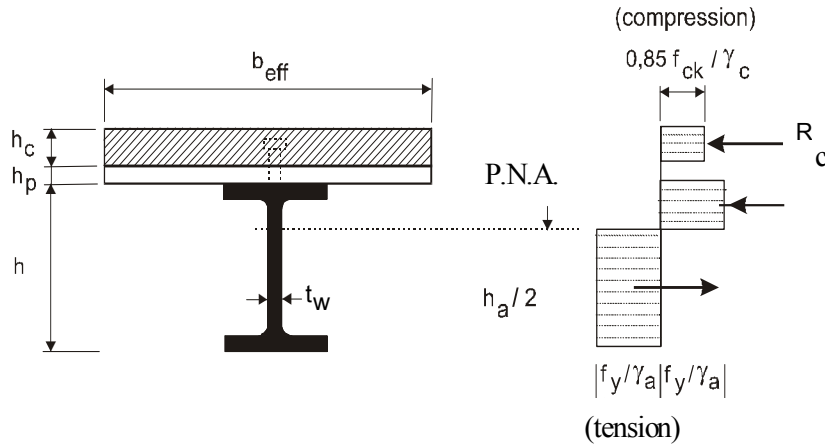
Figure 5.4 Plastic stress blocks when PNA lies within the web

For simplicity, the web-flange fillet is ignored. The tensile force N_{pla1} is balanced by an equal and opposite force acting in an equivalent position on the other side of the centre of gravity of the steel section. There is therefore an area of the web, of width t_w, at a stress of f_y/γ_a to balance the force R_c. The advantage of this expression is the use of the plastic moment resistance of the steel beam $M_{apl.Rd}$ which can be taken directly from standard tables for rolled steel sections.

Moment relation respect to the centre of gravity of the steel beam:

$$M_{pl,Rd} = M_{apl,Rd} + R_c \left[\frac{h_c + 2h_p + h}{2} \right] - \frac{R_c^2}{R_w} \times \frac{h}{4} \tag{5.19}$$

where
$M_{apl,Rd}$ is the plastic moment resistance of the steel section alone.

5.4 SHEAR RESISTANCE

- Pure shear: the shear resistance of the web is taken as shown below:

$$V_{pl,Rd} = \frac{f_y}{\sqrt{3}\gamma_a} \times A_v = 0,58 f_y \frac{A_v}{\gamma_a} \tag{5.20}$$

where
A_v is the shear area of the section

- Combined bending and shear: the interaction equation used to consider at the same time the bending moment and the shear is:

$$M_{sd} \leq M_{fRd} + \left(M_{Rd} - M_{fRd} \right) \left[1 - \left(\frac{2V_{sd}}{V_{plRd} - 1} \right)^2 \right]$$ (5.21)

where
$M_{f,Rd}$ is the moment resistance of the section considering only the flanges
M_{Sd} and V_{Sd} are the applied moment and shear force respectively at the cross section considered

If $V_{Sd} \leq 0,5 \ V_{pl,Rd}$ no reduction to the moment resistance is made

5.5   SHEAR CONNECTION (FULL AND PARTIAL)

There are different types of shear connectors. If welded shear connection is used, the headed stud common dimensions are: 19 mm diameter and 100 mm height. Using this type of connectors, studs are welded through the decking, but they have some limitations.

Limitations:
- The zone where the shear connectors are to be welded cannot be paint. Hence the top flange of the beam should not be painted or the paint removed.
- The galvanized steel should be less than 1.25 mm thick, and it has to be clean and free from moisture.
- Connectors should be capable of resisting uplift forces caused by the tendency of the slab to separate from the beam.

Resistances of stud shear connectors

To calculate the resistances of headed stud shear connectors, there are two design equations to cover the different possibilities of failure:

1. Failure of the concrete:

$$P_{Rd}^a = 0,29\alpha d^2 \sqrt{f_{ck} \frac{E_c}{\gamma_v}}$$ (5.22)

2. Shear failure of the stud, at its weld collar:

$$P_{Rd}^b = 0,8 f_u \frac{\pi d^2}{4\gamma_v}$$ (5.23)

P_{Rd} = smaller $\left(P_{Rd}^a ; P_{Rd}^b \right)$ (5.24)

where
f_u is the ultimate tensile strength of the steel used in the studs (normally 500 N/mm^2)

$\alpha = 0,2\left(\dfrac{h}{d}+1\right) \le 1,0$ takes into account the height and diameter of the stud

$\gamma_v = 1,25$ is the partial safety factor at the ultimate limit state
These formulae apply for stud diameters smaller than 22 mm.

Degree of shear connection

In the plastic design of composite beams, the longitudinal shear force to be transferred between the points of zero and maximum moment should be the smaller of R_c or R_s. If so, full shear connection is provided.

If less shear connectors than the number required for full shear connection are provided it is not possible to develop the full plastic moment resistance of the composite section. In this case the degree of shear connection may be defined as:

$$\frac{N}{N_f} = \frac{R_q}{R_s} \text{ for } R_s < R_c \tag{5.25}$$

$$\frac{N}{N_f} = \frac{R_q}{R_c} \text{ for } R_c < R_s \tag{5.26}$$

where
R_q is the total shear force transferred by the shear connectors between the points of zero and maximum moment
N_f is the number of shear connectors for full shear connection
N is the number of shear connectors provided over the relevant part of the span.

Moment resistance of a composite section with partial shear connection

When R_q, resistance of shear connection, is less than both R_c and R_s there is no full shear connection and the moment resistance is reduced.

$$R_q = N_a \times Q_p \tag{5.27}$$

There are two different methods to determining the reduced moment resistance:

1.   "Linear-interaction" approach

$$M_{Rd} = M_{apl,Rd} + \frac{N}{N_f}\left(M_{pl,Rd} - M_{apl,Rd}\right) \tag{5.28}$$

where
$M_{pl,Rd}$ is the moment resistance of the composite section for full shear connection
$M_{apl,Rd}$ is the moment resistance of the steel section

An adequate design is that which satisfies $M_{Sd} \leq M_{Rd}$. This check may be repeated at point load positions by redefining N as the number of shear connectors from the support to the point considered.

2. Stress block method

This method is exact in that the equilibrium of the section is solved by equating the force in the concrete to the force transferred by the shear connectors R_q. No design formulae are given in Eurocode 4. The stress block method leads to significantly higher moment resistance than the linear interaction method for degrees of shear connection between 0.4 and 0.7.

a) Plastic neutral axis in flange of steel beam: $R_q \geq R_w$

$$M_c = R_s \frac{h}{2} + R_q \left(h_c + h_p\right)\left(1 - \frac{R_q}{2R_c}\right) - \frac{\left(R_s - R_q\right)^2}{4Bp_y} \tag{5.29}$$

That is the same as:

$$M_c = R_s \frac{h}{2} + \frac{N}{N_f} R_c \left(h_c + h_p\right)\left(1 - 0.5\frac{N}{N_f}\right) - \frac{\left(R_s - R_q\right)^2}{4Bp_y} \tag{5.30}$$

b) Plastic neutral axis within the web: $R_q < R_w$

1) Web Class 1 or Class 2

$$\frac{d}{t_w} \leq 76\varepsilon \quad \text{or} \quad \frac{d}{t_w} \leq \frac{76\varepsilon}{1 - R_q/R_v} \tag{5.31}$$

$$M_c = M_{apl,Rd} + R_q \left\{\frac{h}{2} + \left(h_c + h_p\right)\left(1 - \frac{R_q}{2R_c}\right)\right\} - \frac{R_q^2}{R_v}\frac{d}{4} \tag{5.32}$$

That is the same as:

$$M_c = M_{apl,Rd} + \frac{N}{N_f} R_c \left\{\frac{h}{2} + \left(h_c + h_p\right)\left(1 - 0.5\frac{N}{N_f}\right)\right\} - \left(\frac{N}{N_f}\right)^2 \frac{R_c^2}{R_v}\frac{d}{4} \tag{5.33}$$

2) $\dfrac{d}{t_w} > \dfrac{76\varepsilon}{1 - R_q/R_v}$ \hfill (5.34)

$$M_c = M_{apl,Rd} + R_q \left\{\frac{h}{2} + \left(h_c + h_p\right)\left(1 - \frac{R_q}{2R_c}\right)\right\} - \frac{R_q^2 + \left(R_v - R_q\right)\left(R_v - R_q - 2R_o\right)}{R_v}\frac{d}{4}$$

$$\tag{5.35}$$

where

M_c is the reduced moment resistance

d is the clear depth of the steel web

t_w is the thickness of the steel web

$\varepsilon = (275/p_y)^{0,5}$

$R_v = d \times t_w \times p_y$ is the resistance of clear web depth

$R_o = 38 \times \varepsilon \times t_w^2 \times p_y$ is the resistance of slender web

Minimum degree of shear connection

A minimum degree of shear connection limit is introduced in order to ensure adequate deformation capacity of the shear connectors. In principle, the use of stress-block method imposes greater deformations on the shear connectors at failure and therefore, any general limit is conservative for the linear interaction method.

The general limits on the degree of shear connection for a composite slab (with $b_o/h_p \geq$ 2 and $h_p \leq$ 60mm):

$L \leq 25$ m $N/N_f \geq 1-(355/f_y)$ $(1-0,04\ L_e) \geq 0,4$

$L > 25$ m $N/N_f \geq 1.0$

L is the beam span

Influence of deck shape

The deck shape influence on the strength of shear connectors. Strength reduction factor k_t for shear connectors is calculated with the following equation:

$$k_t = \frac{0,7}{\sqrt{N_r}} \times \frac{b_0}{h_p} \times \frac{(h-h_p)}{h_p} \qquad (5.36)$$

with

$k_t \leq 1.0$ for $N_r = 1$

$k_t \leq 0.8$ for $N_r = 2$

where

N_r is the number of studs per trough ($N_r < 3$)

b_0 is the average trough width

h is the stud height

This formula applies to the strength of the shear connectors when the steel decking crosses the beams and where the shear connectors project at least 35 mm above the top of the decking. A further limit is that $h < h_p + 75$ mm in evaluating k_t.

The coefficient 0.7 has been established on basis of test evidence. It is a reduction from the coefficient 0.85 used in previous guidance. It is also recognised that the formula may be unconservative for shear connectors in pairs and therefore the upper limit on k_t is 0.8.

For primary beams, where the decking is placed parallel to the beams, the constant in the above equation is reduced from 0.7 to 0.6. However, no further reduction is made for the number of shear connectors in this case and the limit on k_t is 1.0 for $N_r = 1$ or 2.

5.6   TRANSVERSE REINFORCEMENT

The longitudinal shear strength of the concrete slab should be checked in order to ensure transfer of force from shear connectors into the slab without splitting the concrete. This may require provision of transverse reinforcement perpendicular to the beam. Potential shear planes through the slab lie on either side of the shear connectors.

The shear resistance per unit length of shear plane along the beam is:

$$v_{Rd} = 2.5 A_{cv} \eta \, \tau_{Rd} + A_e \, f_{sk} / \gamma_s \leq 0.2 \, A_{cv} \, \eta \, f_{ck} / \gamma_c \qquad (5.37)$$

where
A_{cv} is the cross-section area of concrete per unit length in any shear plane
τ_{Rd} is the basic shear strength of concrete (see table 5.1)
A_e is the amount of the reinforcement crossing each shear plane
f_{sk} is the yield strength of the reinforcement
η is taken as 1,0 for normal weight concrete and $0.3 + 0.7(\rho/2400)$ for lightweight concrete.

Table 5.1 Basic shear strength of concrete (ENV 1994 (1994))

Strength class of concrete	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck} (N/mm^2)	20	25	30	35	40	45	50
τ_{Rd} (N/mm^2)	0.26	0.30	0.34	0.38	0.42	0.46	0.50

A component arising from the tensile strength of the deck may be added to the longitudinal shear resistance. Its full strength can be used when the deck crosses the beam (secondary beams), and is continuous. When the deck is discontinuous, the anchorage force developed by the shear connectors may be included, provided both ends of the deck are properly attached. The anchorage force per unit length of the beam is given as:

$$v_{pd} = \frac{N_r}{s} \frac{\left(4dt_s f_{yp}\right)}{\gamma_{ap}} \tag{5.38}$$

where
N_r is the number of shear connectors in each group on the beam flange
d is the stud diameter
t_s is the sheet thickness
f_{yp} is the design strength of the sheet steel used to form the profile decking
s is the shear connecting spacing

The coefficient 4 should be replaced by (1+ a /(1.1d)) where "a" is the distance of the edge of the sheet from the centre of the stud. This approach is very conservative and full end anchorage is achieved when the edge distance, a, exceeds 2d. In practice the edge distance should exceed 40 mm.

The total longitudinal shear resistance, for an internal beam, per unit length of the beam is determined by shear failure along two shear planes and is therefore equal to $2(v_{Rd} + v_{pd})$. Where the decking is not properly anchored or where longitudinal sheet overlaps are close to the beam, the contribution of the decking should be neglected.

5.7   LOCAL BUCKLING

In the ultimate limit state analysis of composite beams, it is important to consider the possibility of local buckling. This is done by defining the class of cross-section. As the plate elements in structural sections are relatively thin compared with their width, when loaded in compression (as a result of axial loads applied to the whole section and/or from bending) they may buckle locally. The disposition of any plate element within the cross section to buckle may limit the axial load carrying capacity, or the bending resistance of the section. Avoidance of premature failure arising from the effects of local buckling may be achieved by limiting the width-to-thickness ratio for individual elements within the cross section. This is the basis of the section classification approach. The composite beam section classification is based on an effective width of slab acting together with the steel beam.

Properties of each section class:

Class 1 and 2: capable of developing the full plastic bending moment. Class 1 sections can also rotate after formation of a plastic hinge, but this is not important for simply supported beams.
Class 3: because of local buckling in the part of the steel section under compression, full plastic moment resistance cannot be achieved, although stresses in the extreme fibres of the steel section can reach yield.
Class 4: local buckling occurs before yield is reached in the extreme fibres.

Classification of composite section:

Compression flange:

For simply supported beams the compression flange is the top flange, and it is restrained from buckling by the concrete slab to which it is attached by shear connectors. Flange buckling is assumed to be prevented, and the flange may be defined as Class 1.

Web:

If the plastic neutral axis (PNA) lies in the concrete slab or the upper flange of the section, the composite section can be considered as Class 1 since the web is in tension throughout. When the PNA is in the web, the slenderness of the web should be checked to determine the classification of the web, and hence the classification of the cross-section. But this condition is not common in simply supported beams.

Therefore in composite beams, to know if it is possible that local buckling occurs or not, it is necessary to analysis where the PNA lies. The plastic axial resistance of the steel beam, in tension, is represented by R_s and of the concrete slab, in compression, by R_c:

$$R_s = \frac{A_a \times f_y}{\gamma_a} \tag{5.39}$$

$$R_c = \frac{0,85 f_{ck}}{\gamma_c} \times b_{eff} \times h_c \tag{5.40}$$

where
$\gamma_a = 1.05$
$\gamma_c = 1.5$
A_a is the area of the steel beam
b_{eff} is the effective width of the slab

The concrete in the ribs is ignored so the maximum depth of concrete in compression is limited to the thickness of the slab above the profiles h_c. Considering the longitudinal equilibrium of the composite section it can be seen that the PNA is located in the thickness h_c of the concrete of the slab if $R_c > R_s$.
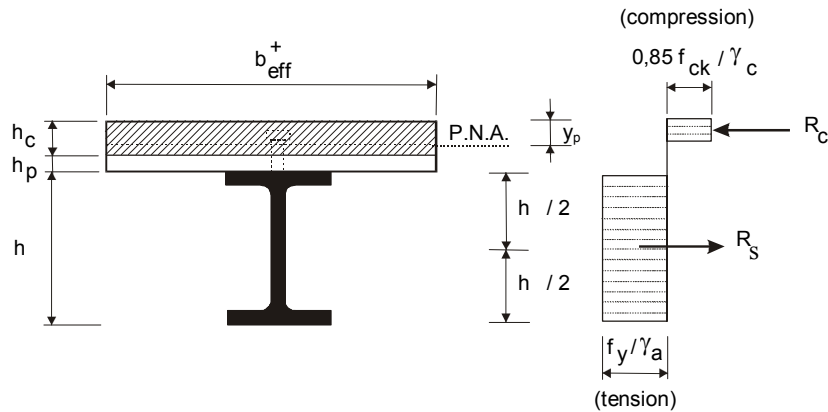
Figure 5.5 Plastic distribution of the normal stresses (example of plastic neutral axis in the slab).

The depth of the plastic neutral axis y_p measured from the upper surface of the concrete slab is given by:

$$y_p = \frac{R_s}{R_c} \times h_c \qquad (5.41)$$

If $y_p < h_c$, the PNA lies within the concrete slab and the composite section is classified as Class 1, so it is capable of developing the full plastic bending moment, and the local buckling is not considered in the design of the composite beam.

5.8   SERVICEABILITY CONDITIONS

Serviceability limit states concerns three different aspects:

  a) Control of deflections
  b) Cracking control
  c) Vibration response

It is common to base assessments at the limit state study on elastic behaviour. To do not be necessary to take considerations of post-elastic effects, limits are usually placed on the stresses existing in beams at the serviceability limit state. However no stress limits are given in Eurocode 4, because it is argued that:

- Slight yielding in the positive moment region has a limited effect on deflections
- The beneficial effects of continuity on deflection are ignored.

Deflection limits are not specified in Eurocode 4 and reference is made to Eurocode 3 for limits on deflections due to permanent and variable loads. Nowadays there are no limits on deflections in Eurocode 4 and either in Eurocode 3.

5.8.1 Control of deflections

Deflections are calculated using the second moment of area of the composite section based on elastic properties. So first of all the second moment of area has to be calculated. Under positive moment the concrete may be assumed to be uncracked. To calculate the second moment of area, the composite section is considered as a transformed steel section. The second moment of area of the composite section, expressed as a transformed steel section, is:

$$I_c = \frac{A_a\left(h_c + 2h_p + h\right)^2}{4\left(1 + nr\right)} + \frac{b_{eff}h_c^3}{12n} + I_{ay} \tag{5.42}$$

where
n is the ratio of the elastic moduli of steel to concrete, taking into account the creep of the concrete when it is relevant
r is the ratio of the cross-sectional area of the steel section relative to the concrete section
I_{ay} is the second moment of area of the steel section

The common value of the ratio I_c/I_{ay} is in the range of 2.5 to 4.0. These values indicate that one of the main benefits we can get with the composite action is in terms of reduction of deflections.

Table 5.2 Deflection limits (ENV 1993-1-1 (1993))

Conditions	δ_{max} (sagging in the final state)	δ_Q (due to variable loading)
Roofs generally	L/200	L/250
Roofs frequently carrying personnel other than for maintenance	L/250	L/300
Floors generally	L/250	L/300
Floors and roofs supporting brittle finish or non-flexible partitions	L/250	L/350
Floors supporting columns	L/400	L/350

The modular ratio, n, represents the ratio of the elastic modulus of steel to the time-dependent modulus of concrete. For normal weight concrete the typically modular ratios that may be used are 6.5 for short term (variable) loading, and 20 for long term (permanent) loading in an internal environment. The values of secant elastic modulus of concrete under short term loads are given in table 5.3. The elastic modulus under long term loads is affected by creep, which causes a reduction in the stiffness of the concrete.

Table 5.3 Secant elastic modulus of concrete

Strength Class of Concrete	20/25	25/30	30/37	35/45	40/50	45/55	50/60
E_{cm} (kN/mm^2)	29	30,5	32	33,5	35	36	37

Influence on deflections of partial shear connection

This effect is very important because deflections increase due to the effect of slip in the shear connectors. However these effects are ignored in composite beams designed for full shear connection. For cases of partial shear connection using shear connectors, the deflection, δ, is increased according to:

$$\frac{\delta}{\delta_c} = 1 + C \times \left(1 - \frac{N}{N_f}\right) \times \left(\frac{\delta_a}{\delta_c} - 1\right)$$ (5.43)

where
N/N_f is the degree of shear connection
δ_c is the deflection of the composite beam with full shear connection
δ_a is the deflection of the steel beam under the same loads
C is a coefficient, taken as 0.3 for unpropped construction and 0.5 for propped construction.

This coefficient, C, is different for unpropped and propped construction due to the higher force in the shear connectors at serviceability state in propped construction than in unpropped construction.

In Eurocode 4 no account of slip is taken in umpropped beams when $N/N_f \geq 0.5$ because it is argued that deflections are already conservative.

Shrinkage induced deflections

Deflections produced by shrinkage are considered only in some specific cases. Eurocode 4 is ambiguous about deflections arising from shrinkage of the concrete slab. It states that shrinkage deflections need only be calculated for simply supported beams when span to depth ratio of the beam exceeds 20, and when the free shrinkage strain of the concrete exceeds 400 x 10^{-6}. In practice, these deflections will only be significant for spans greater than 12 m in exceptionally warm dry atmospheres.

The curvature, K_s, due to a free shrinkage strain, ε_s, is:

$$\varphi_s = \frac{\varepsilon_s \left(h_c + 2h_p + h\right) A_a}{2\left(1 + nr\right) I_c}$$ (5.44)

where

n is the modular ratio appropriate for shrinkage calculations, n ≈ 20.

The deflection due to this curvature for simple supported beams is:

$$\delta_s = 0.125 \ k_s \ L^2 \tag{5.45}$$

This formula to calculate the deflection due to shrinkage ignores continuity effects at the supports; hence it probably over-estimates shrinkage deflections by a considerable margin.

5.8.2 Crack control

Crack control is not always necessary, only where the proper functioning of the structure or its aesthetic aspects, appearance, would be impaired. Where it is necessary to control cracking, the amount of reinforcement should exceed a minimum value. Due to this extra reinforcement cracks are distributed uniformly in the negative moment region. This minimum percentage of reinforcement, ρ, is given by:

$$\rho = \frac{A_S}{A_c} \times 100\% = k_c \times k \times \frac{f_{ct}}{\sigma_s} \times 100\% \tag{5.46}$$

where
k_c is a coefficient due to the bending stress distribution in the section with a value between 0.4 and 0.9.
k is a coefficient accounting for the decrease in tension strength (k ≈ 0.8).
f_{ct} is the effective tensile strength of concrete. A value of 3 N/mm^2 is the minimum adopted.
σ_s is the maximum permitted stress in the reinforcement.

A typical value of ρ is 0.4% to 0.6% which is well in excess of the minimum of 0.2% necessary for shrinkage control and transverse load distribution. These bars do not need to be placed along the entire beam, only in the negative moment region of the beams or slabs.

5.8.3 Vibration response

Potential vibration response may be necessary to be check especially for long span beams. Natural frequency of a beam is:

$$f = \frac{18}{\sqrt{\delta_{sw}}} \ \text{cycles/sec} \tag{5.47}$$

where
δ_{sw} is the instantaneous deflection (in mm) caused by the self weight of the floor and other permanent loads on to the composite beam.

A minimum limit on natural frequency is proposed as 4 cycles/sec for most building applications except where is vibrating machinery, or 3 cycles/sec for car parks. The limit may be raised to 5 cycles/sec for special buildings such as sport halls.

6.   DESIGN EXAMPLES

The following worked examples consist on the design of a composite beam using light steel sections, unpropped and propped during construction. In both examples the dimensions of the beam are the same to make easier the comparison, although under these particular conditions the beam can have a greater span if is propped. The worked example considers different aspects:

- Design of the beam at the construction stage
- Design of the beam at the composite stage
- Serviceability limit stage considerations
    - Deflections
    - Transverse reinforcement
    - Vibrations

6.1   DESIGN EXAMPLE OF AN UNPROPPED 4.5 m SPAN COMPOSITE BEAM TO EUROCODE 4.

Design data

Imposed load:

Imposed load 2.5 kN/m^2

Floor dimensions:

Span L = 4.5 m
Beam spacing b = 1.5 m
Slab depth h_t = 130 mm
Depth above profile h_c = 80 mm
Deck profiled height h_p = 50 mm

Unpropped construction throughout

Shear connectors:

Top-hat shear connectors attached with 4 studs to the beam.

Materials:

Steel: Grade S280
            Nominal value of yield strength $f_y = 280 \text{ N/mm}^2$
            Partial safety factor $\gamma_a = 1.05$

            Design strength $f_d = \dfrac{f_y}{\gamma_a} = \dfrac{280}{1.05} = 267 \text{ N/mm}^2$

Concrete: Normal weight concrete strength class C30/37
            Density = 2400 Kg/m^3 (23.55 kN/m^3)

Loading:

Self weight of the concrete slab

Weight =
$$\left[130 \times 10^3 - 50(120+30) \times \frac{10^3}{300}\right] \times \frac{1}{10^3} \times \frac{23.55}{10^9} = 150 \times \frac{23.55}{10^9} = 2.47275 \times 10^{-6} \text{ kN/mm}^2 =$$
$$= 2.47275 \text{ kN/m}^2$$

Construction Stage:

Concrete slab   =   2.47 kN/m^2
Steel deck   =   0.15 kN/m^2
Reinforcement (allow) =   0.04 kN/m^2
Steel beam   (allow)   =   0.05 kN/m^2

Total   =   2.71 kN/m^2

Construction Load   =   0.50 kN/m^2

Composite Stage:

Concrete slab   =   2.47 kN/m^2
Steel deck   =   0.15 kN/m^2
Reinforcement (allow) =   0.04 kN/m^2
Steel beam   (allow)   =   0.05 kN/m^2

Total   =   2.71 kN/m^2

Ceiling and services = 0.50 kN/m^2

Imposed:

Total imposed load = 2.5 kN/m^2

Initial selection of beam size

The response of a DOUBLE GENERIC C SECTION is going to be studied. A suitable section for imposed load of 2.5 kN/m^2 would be a **200 x 65 x 1.6 Grade S280**

Section properties and dimensions:

$h = 200$ mm
$b = 2 \times 65 = 130$ mm
$t_w = 2 \times 1.6 = 3.2$ mm
$t_f = 1.6$ mm
$A_a = 10.82$ cm^2
$I_{xx} = 646.4$ cm^4
$W_{xx} = 64.65$ cm^3

Nominal value of yield strength $f_y = 280$ N/mm^2 ($t_f < 40$ mm)

Construction stage design

Ultimate limit stage loading

Dead load factor $\gamma_G = 1.35$
Imposed load factor $\gamma_Q = 1.5$

Slab + beam = 2.71 x 1.35 = 3.6585 kN/m^2
Construction = 0.50 x 1.5 = 0.75 kN/m^2
Total = 4.4085 kN/m^2

Total design load = F = 4.4085x 4.5 x 1.5 = 29.7574 kN

Design moment = $M_{sd} = \dfrac{29.7574 \times 4.5}{8} = 16.7385$ kNm

It is assumed that the beam in the construction stage is laterally restrained by the decking since the decking spans perpendicular to the beam and is directly attached to it.

Moment resistance of the steel beam = $M_{a.Rd}$

$$M_{a.Rd} = W_{xx} \times f_d = \frac{64.65 \times 267}{10^3} = 17.24 \text{ kNm} > 16.7385 \text{ KNm}$$

Composite stage design

Ultimate limit stage loading

Slab + beam = 2.71 x 1.35 = 3.6585 kN/m^2
Ceiling + services = 0.50 x 1.35 = 0.675 kN/m^2
Imposed load = 2.5 x 1.5 = 3.75 kN/m^2

Total = 8.0835 kN/m^2

Total design load = F = 8.0835 x 4.5 x 1.5 = 54.5636 kN

Design moment = $M_{sd} = \dfrac{54.5636 \times 4.5}{8} = 30.6920$ kNm

Effective width of compression flange: b$_{eff}$

$$b_{eff} = \frac{2 \times l_o}{8} = \frac{2 \times 4.5}{8} = 1.125 \text{ m} < 1.5 \text{ m (beam spacing)}$$

1. **Compressive resistance of slab**: R$_c$

$$R_c = \frac{0,85 f_{ck}}{\gamma_c} \times b_{eff} \times h_c$$

where
f_{ck} is the characteristic strength of concrete = 30 N/mm^2

γ_c is the partial safety factor for concrete = 1.5

$$R_c = \frac{0.85 \times 30}{1.5} \times 1125 \times \frac{80}{10^3} = 1530 \text{ kN}$$

2. **Tensile resistance of steel section**: R$_s$

$$R_s = A_a \times f_d$$
$$R_s = 10.82 \times 267 \times \frac{10^2}{10^3} = 288.4813 \text{ kN}$$

3. Moment resistance with full shear connection:

Since $R_s < R_c$ the plastic neutral axis (P.N.A.) lies in the concrete flange, therefore the moment resistance of the composite beam is:

$$M_{pl.Rd} = R_s \left[\frac{h}{2} + h_c + h_p - \frac{R_s}{R_c} \times \frac{h_c}{2} \right]$$

$$M_{pl.Rd} = 288.4813 \times \left[\frac{200}{2} + 80 + 50 - \frac{288.4813}{1530} \times \frac{80}{2} \right] = 64174.9817 \text{ kNmm}$$

$$M_{pl.Rd} = 64.1750 \text{ kNm} > M_{sd} = 30.6920 \text{ kNm}$$

The plastic neutral axis depth is:

$$y_p = \frac{R_s}{R_c} \times h_c = \frac{288.4813}{1530} \times 80 = 15.0840 \text{ mm}$$

15.0840 mm < 80 mm → the P.N.A lies in the concrete flange so composite section is Class 1.

4. Shear connector resistance:

In light steel sections, top-hat connectors are used. Each top-hat connector is fixed by four studs. The resistance of each stud has been calculated in several tests by Lawson and Popo-Ola (1998), and it has a value of 18 kN. Hence the resistance of a shear connector is equal to:

Stud resistance = 18 kN
Shear connector resistance $P_u = 4 \times 18 = 72$ kN
$P_{Rd} = 0.8 \times P_u = 57.6$ kN

Due to the geometry of the top-hat, if a connector is placed in each decking elevation between the points of zero and maximum moment the longitudinal shear force transfer is:

$$R_q \text{ (4 studs)} = \frac{P_{Rd} \times L}{2} = \frac{57.6 \times 4.5}{2} = 129.6 \text{ kN}$$

5. Degree of shear connection:

There is a minimum degree of shear connection which for beams with a span equal or less than 5 m is 0.4.

$$\frac{N}{N_f} = \frac{R_q}{R_s} = \frac{129.6}{288.4813} = 0.449 > 0.4$$

The degree of shear connection cannot be higher than 1 and less than a minimum equal to 0.4.

$$1 > 0.449 > 0.4$$

6. Moment resistance with partial shear connection:

Using the linear interaction method, the moment resistance of a composite beam is obtained as follows:

$$M_{Rd} = M_{a.Rd} + \frac{N}{N_f} \times \left(M_{pl.Rd} - M_{a.Rd}\right)$$

where
$$M_{a.Rd} = W_{xx} \times f_d = 17.24 \quad \text{kNm}$$
$$\frac{N}{N_f} = \text{Degree of shear connection}$$
$$M_{pl.Rd} = \text{Moment resistance based on full shear connection}$$

$$M_{Rd} = 17.24 + 0.449 \times \left(64.1750 - 17.24\right) = 38.3254 \quad \text{kNm}$$

$$M_{Rd} = 38.3254 > M_{Sd} = 30.6920 \quad \text{kNm}$$

Increase in moment resistance due to composite action:

$$\frac{M_{Rd}}{M_{a.Rd}} = \frac{38.3254}{17.24} = 2.223$$

7. Vertical shear:

Beam + slab = 2.71 x 1.35 x 4.5 x 1.5 / 2 = 12.3474kN
Ceiling + services = 0.5 x 1.35 x 4.5 x 1.5 / 2 = 2.2781 kN
Imposed load = 2.5 x 1.5 x 4.5 x 1.5 / 2 = 12.6563 kN

Total shear force: $V_{Sd} = 27.2818$ kN

Shear resistance: $V_{pl,Rd}$

$$V_{pl.Rd} = A_v \times \frac{f_d}{\sqrt{3}}$$

where
$$A_v = h t_w$$
$$V_{pl.Rd} = 200 \times 3.2 \times \frac{267}{\sqrt{3}} = 98534.45826 \text{ N}$$
$$V_{pl.Rd} = 98.5344 \text{ kN}$$

$$V_{Sd} = 27.2818 \text{ kN} \leq V_{pl.Rd} = 98.5344 \text{ kN}$$
$$0.5 \times V_{pl.Rd} = 49.267 \text{ kN} > V_{Sd} = 27.2818 \text{ KN} \text{ (there is no interaction bending moment-shear force)}$$

8. Shear buckling resistance:

Shear buckling resistance must be checked if the web slenderness (d/t_w) exceeds 69ε:
$$\frac{d}{t_w} > 69\varepsilon$$

where
d is the height of the web
t_w is the thickness of the web
$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{280}}$$
$$d = h - 2 \times t_f = 200 - 2 \times 1.6 = 196.8 \text{ mm}$$
$$t_w = 2 \times 1.6 = 3.2 \text{ mm}$$

$$\frac{196.8}{3.2} = 61.5$$

$$69\varepsilon = 63.2$$

$61.5 < 63.2 \rightarrow$ therefore the possibility of buckling in shear is not necessary to be checked.

Serviceability limit stage

1.   **Elastic stress**

In Eurocode 4, ENV 1994 (1994), no stress checks are required for normal conditions, so no stress limits are given.

2.   **Deflections**

• Non-composite stage deflection: δ

Self weight of slab and beam = 2.71 kN/mm^2
Design load F = 2.71 x 4.5 x 1.5 = 18.2925 kN

E_a = 205 kN/mm^2

$$\delta = \frac{5FL^3}{384 E_a I_{xx}}$$

$$\delta = \frac{5 \times 18.2925 \times (4.5 \times 10^3)^3}{384 \times 205 \times 646.4 \times 10^4} = 16.3802 \text{ mm}$$

• Composite stage deflection: δ$_c$

Imposed load = 2.5 kN/mm^2
Design Load F = 2.5 x 4.5 x 1.5 = 16.875 kN

$$I_c = \frac{A_a (h + 2h_p + h_c)^2}{4(1 + nr)} + \frac{b_{eff} \times h_c^3}{12n} + I_{xx}$$

$$r = \frac{A_a}{b_{eff} \times h_c} = \frac{10.82 \times 10^2}{1125 \times 80} = 0.01202006$$

n = Modular ratio = 10, for normal weight concrete

$$I_c = \frac{10.82 \times 10^2 \times (200 + 2 \times 50 + 80)^2}{4 \times (1 + 10 \times 0.01202006)} + \frac{1125 \times 80^3}{12 \times 10} + 646.4 \times 10^4 = 46126288.5 \text{ mm}^4$$
$$I_c = 4.61262885 \times 10^7 \text{ mm}^4$$

Deflection with full shear connection

$$\delta_c = \frac{5FL^3}{384E_a I_c}$$

$$\delta_c = \frac{5 \times 16.875 \times \left(4.5 \times 10^3\right)^3}{384 \times 205 \times 4.61262885 \times 10^7} = 2.1175 \, \text{mm}$$

As partial shear connection exists, the effect of slip has to be taken into account:

$$\frac{\delta}{\delta_c} = 1 + 0.3\left[1 - \frac{N}{N_f}\right] \times \left[\frac{\delta_a}{\delta_c} - 1\right]$$

$$\delta_a = \frac{\delta_c \times I_c}{I_{xx}} = \frac{2.1175 \times 4.61266505 \times 10^7}{646.4 \times 10^4} = 15.1109 \, \text{mm}$$

$$\delta = 2.1175 \times \left[1 + 0.3\left(1 - 0.449\right) \times \left[\frac{15.1100}{2.1175} - 1\right]\right] = 4.2643 \, \text{mm}$$

$$4.2643 \approx \frac{L}{1055} < \frac{L}{350}$$

The deflection due to imposed load is satisfactory

- Total deflection:

Construction stage = 16.3802 mm
Imposed load = 4.2643 mm
Ceiling + services $\approx \dfrac{0.5 \times 4.2643}{2.5} = 0.8529 \, \text{mm}$

Total = 21.4973 mm

The limit on the maximum total deflection for a composite beam is the following:

$$\frac{L}{200} = \frac{4500}{200} = 22.5 \, \text{mm} > \text{total deflection} = 21.4973 \, \text{mm}$$

The total deflection is also satisfactory.

3.   Transverse reinforcement

The resistance of concrete flange to splitting will be check, using A142 mesh reinforcement in the slab.

- Shear resistance per shear surface, v_{Rd}

$$v_{Rd} = 2.5 A_{cv} \eta \tau_{Rd} + \frac{A_e f_{sk}}{\gamma_s} \leq 0.2 A_{cv} \frac{\eta f_{ck}}{\gamma_c}$$

$A_e = 142 \text{ mm}^2/\text{m}$
$\eta = 1$ for normal weight concrete
$A_{cv} = 105 \times 10^3 \text{ mm}^2/\text{m}$
$f_{ck} = 30 \text{ N/mm}^2$
$f_{sk} = 460 \text{ N/mm}^2$
$\tau_{Rd} = 0.25 \times \frac{f_{ctk0.05}}{\gamma_c} = 0.25 \times \frac{1.8}{1.5} = 0.3$
$\gamma_c = 1.5$
$\gamma_s = 1.15$

$$v_{Rd} = \left[2.5 \times 105 \times 10^3 \times 1 \times 0.3 + \frac{142 \times 460}{1.15} \right] \times 10^{-3} = 135.55 \text{ kN/m}$$

$$0.2 A_{vc} \frac{\eta f_{ck}}{\gamma_c} = \left[0.2 \times 105 \times 10^3 \times \frac{1 \times 30}{1.5} \right] \times 10^{-3} = 420 \text{ kN/m} > 135.55 \text{ kN/m}$$

- Shear force per unit length, v

Placing the top-hat connectors in the way there are four studs per trough:

$$v_{Sd} = \frac{57.6 \times 0.5}{0.3} = 96 \text{ kN/m} < v_{Rd} = 135.55 \text{ kN/m}$$

A142 mesh is satisfactory

4. Vibration (Simplified approach)

Loading:

Beam + slab = 2.71 kN/m^2
Ceiling + services = 0.50 kN/m^2
10% of imposed load = 0.25 kN/m^2
Total = 3.46 kN/m^2

Total weight of floor, $F = 3.46 \times 4.5 \times 1.5 = 23.355 \text{ kN}$

Increase of second moment of area of the composite section based on elastic properties I_c, by 10% to allow for the increased dynamic stiffness of the composite beam, I_{c1}

$$I_{c1} = 1.1 \times 4.61262885 \times 10^7 = 5.07389174 \times 10^7 \text{ mm}^4$$

Instantaneous deflection caused by re-application of the self weight of the floor and the beam to the composite beam, δ_a

$$\delta_a = \frac{5 \times F \times L^3}{384 \times E_a \times I_{c1}}$$

$$\delta_a = \frac{5 \times 23.355 \times \left(4.5 \times 10^3\right)^3}{384 \times 205 \times 5.07389174 \times 10^7} = 2.6642 \text{ mm}$$

$$\text{Natural frequency} \approx \frac{18}{\sqrt{\delta_a}} = \frac{18}{\sqrt{2.6642}} = 11.0279 \text{ Hz} > 4 \text{ Hz}$$

5. Conclusion

The design is limited basically by the moment resistance of the beam in the construction stage rather than serviceability criteria. The maximum span of the secondary beams that is possible to consider in the design is limited by moment resistance at the construction stage, if the span would be increased the beam would fail before the composite behaviour.

6.2 DESIGN EXAMPLE OF A 4.5 m SPAN BEAM, PROPPED DURING CONSTRUCTION

The main differences on the design of a composite beam propped during construction are the following:

- The beam functions exclusively as composite
- Deflections:
  Sum of:
  - Dead loads on composite section
  - Imposed loads on composite section

Using props during construction sometimes it is possible to get a greater span, but not in all cases, it depends on the imposed loads because all of the loads have to be resisted by the composite section.
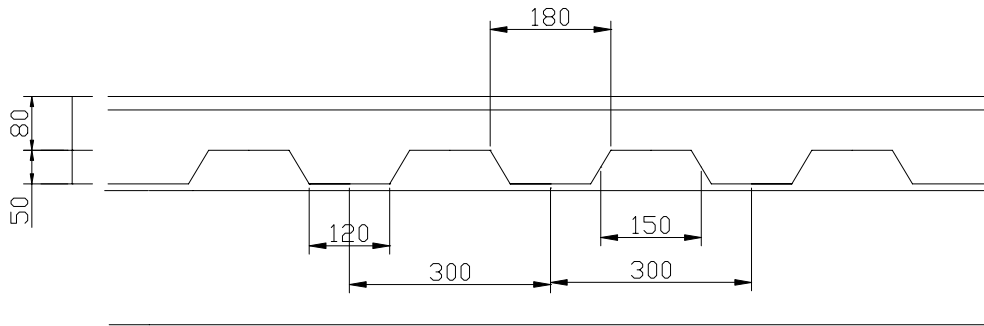
Design data

Imposed load:

Imposed load 2.5 kN/m^2

Floor dimensions:

Span L = 4.5 m
Beam spacing b = 1.5 m
Slab depth h_t = 130 mm
Depth above profile h_c = 80 mm
Deck profiled height h_p = 50 mm

Beam propped during construction. Props are placed in the middle of the span.

Shear connectors:

Top-hat shear connectors attached with 4 studs to the beam.

Materials:

Steel: Grade S280
          Nominal value of yield strength f_y = 280 N/mm^2
          Partial safety factor γ_a = 1.05

$$\text{Design strength } f_d = \frac{f_y}{\gamma_a} = \frac{280}{1.05} = 267 \text{ N/mm}^2$$

Concrete: Normal weight concrete strength class C30/37
          Density = 2400 Kg/m^3 (23.55 kN/m^3)

Loading:

Self weight of the concrete slab

Weight =

$$=\left[130\times10^3-50(120+30)\times\frac{10^3}{300}\right]\times\frac{1}{10^3}\times\frac{23.55}{10^9}=150\times\frac{23.55}{10^9}=2.47275\times10^{-6}\text{ kN/mm}^2$$

Weight = 2.47275 kN/m^2

Construction Stage:

Concrete slab	=	2.47 kN/m^2
Steel deck	=	0.15 kN/m^2
Reinforcement (allow)	=	0.04 kN/m^2
Steel beam (allow)	=	0.05 kN/m^2
Total	=	2.71 kN/m^2
Construction Load	=	0.50 kN/m^2

Composite Stage:

Concrete slab	=	2.47 kN/m^2
Steel deck	=	0.15 kN/m^2
Reinforcement (allow)	=	0.04 kN/m^2
Steel beam (allow)	=	0.05kN/m^2
Total	=	2.71 kN/m^2
Ceiling and services	=	0.50 kN/m^2

Imposed:

Total imposed load	=	2.5 kN/m^2

Initial selection of beam size

The response of a DOUBLE GENERIC C SECTION is going to be studied. A suitable section for imposed load of 2.5 kN/m^2 would be a **200 x 65 x 1.6 Grade S280**

Section properties and dimensions:

$h = 200$ mm
$b = 2\times65 = 130$ mm

$t_w = 2 \times 1.6 = 3.2$ mm

$t_f = 1.6$ mm

$A_a = 10.82$ cm^2

$I_{xx} = 646.4$ cm^4

$W_{xx} = 64.65$ cm^3

Nominal value of yield strength $f_y = 280$ N/mm^2 ($t_f < 40$ mm)

Construction stage design

During the construction stage the span of the beam is half of the final span, due to the prop placed in the middle span.

Ultimate limit stage loading

Dead load factor $\gamma_G = 1.35$

Imposed load factor $\gamma_Q = 1.5$

Slab + beam = 2.71 x 1.35 = 3.6585 kN/m^2

Construction = 0.50 x 1.5 = 0.75 kN/m^2

Total = 4.4085 kN/m^2

Total design load = F = 4.4085x $\dfrac{4.5}{2}$ x 1.5 = 14.8787 kN

Design moment = $M_{sd} = \dfrac{14.8787 \times \dfrac{4.5}{2}}{8} = 4.1846$ kNm

It is assumed that the beam in the construction stage is laterally restrained by the decking since the decking spans perpendicular to the beam and is directly attached to it.

Moment resistance of the steel beam = $M_{a.Rd}$

$M_{a.Rd} = W_{xx} \times f_d = \dfrac{64.65 \times 267}{10^3} = 17.24$ kNm > 4.1846 kNm

Composite stage design

In the composite stage the props have been removed and the beam span is 4.5 m.

Ultimate limit stage loading

Slab + beam = 2.71x 1.35 = 3.6585 kN/m^2
Ceiling + services = 0.50 x 1.35 = 0.675 kN/m^2
Imposed load = 2.5 x 1.5 = 3.75 kN/m^2

Total = 8.0835kN/m^2

Total design load = F = 8.0835 x 4.5 x 1.5 = 54.5636 kN

Design moment = $M_{sd} = \dfrac{54.5636 \times 4.5}{8} = 30.6920$ kNm

Effective width of compression flange: b$_{eff}$

$$b_{eff} = \frac{2 \times l_o}{8} = \frac{2 \times 4.5}{8} = 1.125 \text{ m}$$

1. **Compressive resistance of slab:** R$_c$

$$R_c = \frac{0,85 f_{ck}}{\gamma_c} \times b_{eff} \times h_c$$

where
f_{ck} is the characteristic strength of concrete = 30 N/mm^2

γ_c is the partial safety factor for concrete = 1.5

$$R_c = \frac{0.85 \times 30}{1.5} \times 1125 \times \frac{80}{10^3} = 1530 \text{ kN}$$

2. **Tensile resistance of steel section:** R$_s$

$$R_s = A_a \times f_d$$
$$R_s = 10.82 \times 267 \times \frac{10^2}{10^3} = 288.4813 \text{ kN}$$

3. Moment resistance with full shear connection:

Since $R_s < R_c$ the plastic neutral axis (P.N.A.) lies in the concrete flange therefore the composite section is Class 1. The moment resistance of the composite beam is:

$$M_{pl.Rd} = R_s \left[\frac{h}{2} + h_c + h_p - \frac{R_s}{R_c} \times \frac{h_c}{2} \right]$$

$$M_{pl.Rd} = 288.4813 \times \left[\frac{200}{2} + 80 + 50 - \frac{288.4813}{1530} \times \frac{80}{2} \right] = 64174.9817 \text{ kNmm}$$

$$M_{pl.Rd} = 64.1750 \text{ kNm} > M_{sd} = 30.6920 \text{ kNm}$$

The plastic neutral axis depth is:

$$y_p = \frac{R_s}{R_c} \times h_c = \frac{288.4813}{1530} \times 80 = 15.0840 \text{ mm}$$

15.0840 mm < 80 mm → P.N.A lies in the concrete flange.

4. Shear connector resistance:

In light steel sections, top-hat connectors are used. Each top-hat connector is fixed by four studs. The resistance of each stud has been calculated in several tests by Lawson and Popo-Ola (1998), and it has a value of 18 kN. Hence the resistance of a shear connector is equal to:

Stud resistance = 18 kN
Shear connector resistance $P_u = 4 \times 18 = 72$ kN
$P_{Rd} = 0.8 \times P_u = 0.8 \times 72 = 57.6$ kN

Due to the geometry of the top-hat, if a connector is placed in each decking elevation between the points of zero and maximum moment, the longitudinal shear force transfer is:

$$R_q \text{ (4 studs)} = \frac{P_{Rd} \times L}{2} = \frac{57.6 \times 4.5}{2} = 129.6 \text{ kN}$$

5. Degree of shear connection:

There is a minimum degree of shear connection which for beams with a span equal or less than 5 m is 0.4.

$$\frac{N}{N_f} = \frac{R_q}{R_s} = \frac{129.6}{288.4813} = 0.449 > 0.4$$

The degree of shear connection cannot be higher than 1, and less than 0.4.

$$1 > 0.449 > 0.4$$

6. Moment resistance with partial shear connection:

Using the linear interaction method, the moment resistance of a composite beam is obtained as follows:

$$M_{Rd} = M_{a.Rd} + \frac{N}{N_f} \times \left(M_{pl.Rd} - M_{a.Rd}\right)$$

where

$$M_{a.Rd} = W_{xx} \times f_d = 17.24 \quad \text{kNm}$$

$$\frac{N}{N_f} = \text{Degree of shear connection}$$

$$M_{pl.Rd} = \text{Moment resistance based on full shear connection}$$

$$M_{Rd} = 17.24 + 0.449 \times \left(64.1750 - 17.24\right) = 38.3254 \quad \text{kNm}$$

$$M_{Rd} = 38.3254 > M_{Sd} = 30.6920 \quad \text{kNm}$$

Increase in moment resistance due to composite action:

$$\frac{M_{Rd}}{M_{a.Rd}} = \frac{38.3254}{17.24} = 2.223$$

7. Vertical shear:

Beam + slab = 2.71 x 1.35 x 4.5 x 1.5 / 2 = 12.3474 kN
Ceiling + services = 0.5 x 1.35 x 4.5 x 1.5 / 2 = 2.2781 kN
Imposed load = 2.5 x 1.5 x 4.5 x 1.5 / 2 = 12.6563 kN

Total shear force: $V_{Sd} = 27.2818$ kN

Shear resistance: $V_{pl,Rd}$

$$V_{pl,Rd} = A_v \times \frac{f_d}{\sqrt{3}}$$

where
$A_v = ht_w$

$$V_{pl.Rd} = 200 \times 3.2 \times \frac{267}{\sqrt{3}} = 98534.4459 \text{ N}$$

$$V_{pl.Rd} = 98.5344 \text{ kN}$$

$$V_{Sd} = 27.2818 \text{ kN} \leq V_{pl.Rd} = 98.5344 \text{ kN}$$

$0.5 \times V_{pl.Rd} = 49.267 \text{ kN} > V_{Sd} = 27.2818 \text{ KN}$ (there is no interaction bending moment-shear force)

8. Shear buckling resistance:

Shear buckling resistance must be checked if the web slenderness (d/t_w) exceeds 69ε:

$$\frac{d}{t_w} > 69\varepsilon$$

where
d is the height of the web
t_w is the thickness of the web

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{280}}$$

$$d = h - 2 \times t_f = 200 - 2 \times 1.6 = 196.8 \text{ mm}$$

$$t_w = 2 \times 1.6 = 3.2 \text{ mm}$$

$$\frac{196.8}{3.2} = 61.5$$

$$69\varepsilon = 63.2$$

$61.5 < 63.2 \rightarrow$ therefore the possibility of buckling in shear is not necessary to be checked.

Serviceability limit stage

1.   Elastic stress

In Eurocode 4, ENV 1994 (1994), no stress checks are required for normal conditions, so no stress limits are given.

2.   Deflections

The beam functions exclusively as composite, so the deflections to be considered are composite stage deflections. The deflections caused by the dead weight of the slab and the beam when the props are removed, and the deflections caused by the weight of the floor finished (ceiling&services and imposed loads).

Second moment of area of the composite section:

$$I_c = \frac{A_a \left(h + 2h_p + h_c\right)^2}{4\left(1 + nr\right)} + \frac{b_{eff} \times h_c^3}{12n} + I_{xx}$$

$$r = \frac{A_a}{b_{eff} \times h_c} = \frac{10.82 \times 10^2}{1125 \times 80} = 0.01202006$$

n = Modular ratio = 10, for normal weight concrete

$$I_c = \frac{10.82 \times 10^2 \times \left(200 + 2 \times 50 + 80\right)^2}{4 \times \left(1 + 10 \times 0.01202006\right)} + \frac{1125 \times 80^3}{12 \times 10} + 646.4 \times 10^4 = 46126288.5 \ \text{mm}^4$$

$$I_c = 4.61262885 \times 10^7 \ \text{mm}^4$$

- Slab and beam weight deflections:

Self weight of slab and beam = 2.71 kN/mm^2
Design load F = 2.71 x 4.5 x 1.5 = 18.2925 kN

E_a = 205 kN/mm^2

$$\delta_1 = \frac{5FL^3}{384 E_a I_c}$$

$$\delta_1 = \frac{5 \times 18.2925 \times \left(4.5 \times 10^3\right)^3}{384 \times 205 \times 4.61262885 \times 10^7} = 2.2953 \ \text{mm}$$

- Imposed loads deflections:

Imposed load = 2.5 kN/mm^2
Design load F = 2.5 x 4.5 x 1.5 = 16.875 kN

Deflection with full shear connection

$$\delta_2 = \frac{5FL^3}{384E_a I_c}$$

$$\delta_2 = \frac{5 \times 16.875 \times (4.5 \times 10^3)^3}{384 \times 205 \times 4.61262885 \times 10^7} = 2.1175 \ mm$$

As partial shear connection exists, the effect of slip has to be taken into account on the deflections due to imposed load and also on the deflections of dead loads (Slab+beam weight and ceiling&services).

It is necessary to check the deflection due to the imposed load:

$$\frac{\delta_2^*}{\delta_2} = 1 + 0.5\left[1 - \frac{N}{N_f}\right] \times \left[\frac{\delta_{a2}}{\delta_2} - 1\right]$$

$$\delta_{a2} = \frac{\delta_2 \times I_c}{I_{xx}} = \frac{2.1175 \times 4.61262885 \times 10^7}{646.4 \times 10^4} = 15.1109 \ mm$$

$$\delta_2^* = 2.1175 \times \left[1 + 0.5(1 - 0.449) \times \left[\frac{15.1109}{2.1175} - 1\right]\right] = 5.6955 \ mm$$

$$5.6955 \approx \frac{L}{790} < \frac{L}{350}$$

The deflection due to imposed load is satisfactory

The effect of slip has to be considered also on the deflection due to the slab and beam self weight, when the props are removed.

$$\frac{\delta_1^*}{\delta_1} = 1 + 0.5\left[1 - \frac{N}{N_f}\right] \times \left[\frac{\delta_{a1}}{\delta_1} - 1\right]$$

$$\delta_{a1} = \frac{\delta_1 \times I_c}{I_{xx}} = \frac{2.2953 \times 4.61262885 \times 10^7}{646.4 \times 10^4} = 16.3802 \text{ mm}$$

$$\delta_1^* = 2.2953 \times \left[1 + 0.5(1 - 0.449) \times \left[\frac{16.3802}{2.2953} - 1\right]\right] = 6.1740 \text{ mm}$$

- Total deflection:

Slab+beam weight = 6.1740 mm
Imposed load = 5.6955 mm
Ceiling + services $\approx \dfrac{0.5 \times 5.6955}{2.5} = 1.1391$ mm
Total = 13.0086 mm

The limit on the maximum total deflection for a composite beam is the following:

$$\frac{L}{200} = \frac{4500}{250} = 20.4 \text{ mm} > \text{total deflection} = 18.3892 \text{ mm}$$

The total deflection is also satisfactory.

3. Transverse reinforcement

The resistance of concrete flange to splitting will be check, using A142 mesh reinforcement in the slab.

- Shear resistance per shear surface, v_{Rd}

$$v_{Rd} = 2.5 A_{cv} \eta \tau_{Rd} + \frac{A_e f_{sk}}{\gamma_s} \leq 0.2 A_{cv} \frac{\eta f_{ck}}{\gamma_c}$$

A_e = 142 mm²/m
η = 1 for normal weight concrete
A_{cv} = 105 x 10³ mm²/m
f_{ck} = 30 N/mm²
f_{sk} = 460 N/mm²
$\tau_{Rd} = 0.25 \times \dfrac{f_{ctk0.05}}{\gamma_c} = 0.25 \times \dfrac{1.8}{1.5} = 0.3$
γ_c = 1.5
γ_s = 1.15

$$v_{Rd} = \left[2.5 \times 105 \times 10^3 \times 1 \times 0.3 + \frac{142 \times 460}{1.15} \right] \times 10^{-3} = 135.55 \text{ kN/m}$$

$$0.2 A_{vc} \frac{\eta f_{ck}}{\gamma_c} = \left[0.2 \times 105 \times 10^3 \times \frac{1 \times 30}{1.5} \right] \times 10^{-3} = 420 \text{ kN/m} > 135.55 \text{ kN/m}$$

- Shear force per unit length, v

Placing the top-hat connectors in the way there are four studs per trough:

$$v_{Sd} = \frac{57.6 \times 0.5}{0.3} = 96 \text{ kN/m} < v_{Rd} = 135.55 \text{ kN/m}$$

A142 mesh is satisfactory

4.   Vibration (Simplified approach)

Loading:

Beam + slab = 2.71 kN/m^2
Ceiling + services = 0.50 kN/m^2
10% of imposed load = 0.25 kN/m^2
Total = 3.46 kN/m^2

Total weight of floor, F = 3.46 x 4.5 x 1.5 = 23.355 kN

Increase of second moment of area of the composite section based on elastic properties I_c, by 10% to allow for the increased dynamic stiffness of the composite beam, I_{c1}

$$I_{c1} = 1.1 \times 4.61262885 \times 10^7 = 5.07389174 \times 10^7 \text{ mm}^4$$

Instantaneous deflection caused by re-application of the self weight of the floor and the beam to the composite beam, δ_a

$$\delta_a = \frac{5 \times F \times L^3}{384 \times E_a \times I_{c1}}$$

$$\delta_a = \frac{5 \times 23.355 \times \left(4.5 \times 10^3\right)^3}{384 \times 205 \times 5.07389174 \times 10^7} = 2.6642 \text{ mm}$$

Natural frequency $\approx \dfrac{18}{\sqrt{\delta_a}} = \dfrac{18}{\sqrt{2.6642}} = 11.0279$ Hz > 4 Hz

5.   Conclusion

When the beam is propped during the construction stage, its span is not limited by the moment resistance at the construction stage rather than in the case of an unpropped construction. If props are used, the design is limited basically by the total deflection which is a serviceability criterion.

7.   LOAD-SPAN DESIGN TABLES FOR COMPOSITE BEAMS USING LIGHT STEEL SECTION AND PROFILED SHEAR CONNECTORS

7.1   PROPERTIES OF LIGHT STEEL SECTIONS

7.1.1   Class classification

The particular cold formed sections analysed are Generic C sections which properties have been tabulated by Lawson, Chung and Popo-Ola (2002) (see Annex 1). If we check the class classification for the single light steel sections according to the criteria of Eurocode 3, showed in table 7.1, the result is that all sections are slender (see Annex 2.1).

The following table gives the limiting proportions for compression elements of Class 1 to 3. When any of the compression elements within a section fail to satisfy the limit for Class 3 the whole section is classified as Class 4 (commonly referred to as slender), and local buckling should be taken into account in the design using an effective cross section.

Table 7.1 Maximum slenderness ratios for the elements of a rolled section in compression and bending

Element	Class 1	Class 2	Class 3
Flange	$c / t_f = 10\ \varepsilon$	$c / t_f = 11\ \varepsilon$	$c / t_f = 15\ \varepsilon$
Web subject to bending	$d / t_w = 72\ \varepsilon$	$d / t_w = 83\ \varepsilon$	$d / t_w = 124\ \varepsilon$
Web subject to compression	$d / t_w = 33\ \varepsilon$	$d / t_w = 38\ \varepsilon$	$d / t_w = 42\ \varepsilon$

As the Annex 2.1 tables show, all the single sections are Class 4, and this means that local buckling may occur in compression elements before yield is reached. However in composite construction the sections used are back to back double sections, not single sections, and working compositely with a concrete slab. In this case the class classification of the section is different.

In calculation for the construction stage of a composite beam the classification of the cross section should be based on the plain steel section, but secondary beams are laterally restrained by the steel decking and can develop their full moment resistances.

Once the composite action is achieved, to know if local buckling can occur or not, the position of the plastic neutral axis has to be checked. These calculations have been carried out for all the cases treated in this project, with the different sections and imposed loads (see Annex 2.2). The plastic neutral axis lies within the concrete slab in all the combinations studied; hence the composite sections are classified as Class 1 and local buckling has not to be considered in the design.

7.1.2 Shear buckling

Working with light steel sections it is important to consider the possibility of buckling in shear. Shear buckling resistance must be checked if the web slenderness (d/t_w) exceeds 69ε:

$$\frac{d}{t_w} > 69\varepsilon \tag{7.1}$$

where
d is the height of the web
t_w is the thickness of the web

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

The possibility of buckling in shear has been checked for all the Generic C double sections studied in this project as the table 7.2 shows. Only with two of the Generic C double sections studied and in the case that steel grade S350 is used, shear in buckling may happed. These two sections are 200x65x1.6 and 300x65x2.4 back to back double sections.

The shear buckling resistance of these two sections has to be calculated to know if that means a hazard or not.

Table 7.2 Study of the possibility of buckling in shear of the different sections

D (mm)	t (mm)	tw (mm)	d (mm)	d/tw	69ε (fy=280 N/mm^2)		69ε (fy=350 N/mm^2)	
100	1,2	2,4	97,6	40,667	63,213	ok	56,539	ok
125	1,2	2,4	122,6	51,083	63,213	ok	56,539	ok
125	1,6	3,2	121,8	38,063	63,213	ok	56,539	ok
150	1,6	3,2	146,8	45,875	63,213	ok	56,539	ok
150	1,8	3,6	146,4	40,667	63,213	ok	56,539	ok
165	1,6	3,2	161,8	50,563	63,213	ok	56,539	ok
165	1,8	3,6	161,4	44,833	63,213	ok	56,539	ok
180	1,6	3,2	176,8	55,250	63,213	ok	56,539	ok
180	1,8	3,6	176,4	49,000	63,213	ok	56,539	ok
180	2,0	4	176	44,000	63,213	ok	56,539	ok
200	1,6	3,2	196,8	61,500	63,213	ok	56,539	Buckli.
200	1,8	3,6	196,4	54,556	63,213	ok	56,539	ok
200	2,0	4	196	49,000	63,213	ok	56,539	ok
220	2,0	4	216	54,000	63,213	ok	56,539	ok
220	2,4	4,8	215,2	44,833	63,213	ok	56,539	ok
250	2,4	4,8	245,2	51,083	63,213	ok	56,539	ok
300	2,4	4,8	295,2	61,500	63,213	ok	56,539	Buckli.
300	3,0	6	294	49,000	63,213	ok	56,539	ok

Shear buckling resistance has been calculated with the simple post-critic method for these two sections:

$$V_{ba.Rd} = \frac{d \times t_w \times \tau_{ba}}{\gamma_a} \tag{7.2}$$

where the value of the simple post-critic resistance τ_{ba} depend on the value of the web slenderness $\bar{\lambda}_w$

$$\bar{\lambda}_w = \frac{d/t_w}{37.4 \times \varepsilon \times \sqrt{k_\tau}} \tag{7.3}$$

where
$k_\tau = 5.34$ is the shear buckling coefficient

For both sections, 200x65x1.6 and 300x65x2.4, the value of the web slenderness is:

$$\bar{\lambda}_w = 0.868$$

Therefore

$$0.8 < \bar{\lambda}_w < 1.2$$

And according to the simple post-critic method

$$\tau_{ba} = \left[1 - 0.625\left(\bar{\lambda}_w - 0.8\right)\right]\left(\frac{f_y}{\sqrt{3}}\right) \tag{7.4}$$

With the simple post-critic resistance the shear buckling resistance is calculated for each section, showed in table 7.3, and it can be compared with the shear force V_{Sd} to know if the buckling in shear may be a problem or not with these sections. As it is possible to see in table 7.4, shear force in the cases studied is always lower than the shear buckling resistance of the section, hence shear buckling will never happen.

Table 7.3 Shear buckling resistance

Generic C section	$\bar{\lambda}_w$	τ_{ba} (N/mm^2)	$V_{ba.Rd}$ (kN)
200x65x1.6	0.868	193.43	116.014
300x65x2.4	0.868	193.43	261.032

Table 7.4 Shear force values

200x65x1.6 Unpropped (S350)			200x65x1.6 Propped (S350)			300x65x2.4 Unpropped (S350)			300x65x2.4 Propped (S350)		
Imposed load (kN)	L (m)	V_{Sd} (kN)	Imposed load (kN)	L (m)	V_{Sd} (kN)	Imposed load (kN)	L (m)	V_{Sd} (kN)	Imposed load (kN)	L (m)	V_{Sd} (kN)
1.5	4.7	23.2	1.5	5.3	26.3	1.5	7.3	35.9	1.5	7.9	38.9
2.5	4.6	27.8	2.5	4.9	29.7	2.5	7.0	42.4	2.5	7.3	44.3
3.5	4.4	31.6	3.5	4.6	33.1	3.5	6.8	48.9	3.5	6.9	49.6
4.5	4.3	35.7	4.5	4.4	36.6	4.5	6.6	54.9	4.5	6.6	54.9

7.2   DESIGN CRITERIA

Design tables have been calculated for double Generic C light steel sections (see tables 7.5 to 7.8). These tables contain a wide range of cases although one of the parameters involved has been considered fixed, the beam spacing. On this way the design tables can be used to design composite beams and also to compare how sections span capacity vary with the construction conditions, steel grade and imposed load applied.

Design criteria:

-   a: moment resistance of the beam exceeded in the construction stage
-   b: interaction between shear force and bending moment
-   c: moment resistance of the composite beam with full shear connection exceeded
-   d: moment resistance of the composite beam with partial shear connection exceeded
-   e: limit on the degree of partial shear connection not satisfied. This is not a failure criterion but it is a warning that the shear connection provided is less than the Eurocode limit of 40%.
-   g: total deflection of $L_E/200$ exceeded for unpropped, (equal to $L_E/250$ for propped)
-   h: imposed load deflection of span/350 exceeded
-   i: natural frequency is less than 4 Hz

7.3   DESIGN TABLES. PROPPED AND UNPROPPED BEAMS

The two first tables are for unpropped beams during construction (tables 7.5 and 7.6) and the second ones are for propped beams (tables 7.7 and 7.8). The parameters showed in the tables are the maximum span that each section can achieve followed by the design criteria which limits this maximum span. The number of connectors required for these conditions and the deflection that the beam self weight and the imposed loads mean.

Tables' notation:

L_E: span of the beam (secondary beam)
δ_E: deflection due to imposed loads
δ_S: deflection due to beam self weight
N: number of shear connectors

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 1.5 m
- Steel strength S 280
- Stud resistance 18 kN

Beam is **unpropped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table 7.5 Load-span design table for double Generic C section: unpropped – $f_y = 280$ N/mm^2

IMPOSED LOAD kN/m²	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
100x55x1.2	2,4	a	1,1	9,7	8	2,4	a	1,9	9,7	8	2,4	g	2,5	9,0	7	2,2	b	2,7	7,2	7
125x55x1.2	2,8	a	1,3	10,3	9	2,8	a	2,2	10,7	9	2,8	a	3,0	10,3	9	2,7	g	3,6	9,7	9
125x55x1.6	3,2	g	1,8	13,4	10	3,1	g	2,8	12,3	10	3,0	g	3,6	11,1	10	3,0	g	4,2	10,1	9
150x65x1.6	3,8	g	2,3	15,8	12	3,7	g	3,5	14,6	12	3,6	g	4,5	13,0	11	3,5	g	5,3	11,7	11
150x65x1.8	3,9	g	2,4	16,3	13	3,9	g	4,0	16,2	13	3,7	g	4,7	13,2	12	3,6	g	5,6	12,0	12
165x65x1.6	4,0	g	2,4	16,3	13	4,0	g	4,0	16,0	13	3,9	g	4,8	13,8	12	3,8	g	5,7	12,5	12
165x65x1.8	4,2	g	2,7	17,6	14	4,1	g	4,1	15,8	13	4,0	g	5,1	14,1	13	3,9	g	6,1	12,8	12
180x65x1.6	4,3	g	2,5	16,8	14	4,2	g	4,1	16,2	14	4,1	g	5,2	14,6	13	4,0	d	6,1	13,0	13
180x65x1.8	4,5	g	2,9	18,6	14	4,4	g	4,5	17,1	14	4,3	g	5,5	14,9	14	4,1	d	6,4	13,3	13
180x65x2.0	4,6	g	3,1	19,1	15	4,5	g	4,6	17,0	15	4,4	g	5,8	15,3	14	4,3	d	6,8	13,7	14
200x65x1.6	4,6	a	2,7	17,4	15	4,6	a	4,5	17,4	15	4,5	g	5,8	15,8	14	4,2	d	6,1	12,7	14
200x65x1.8	4,8	g	3,1	19,4	16	4,8	g	5,1	19,0	16	4,6	g	6,1	16,1	15	4,4	d	6,5	13,0	14
200x65x2.0	5,0	g	3,4	20,7	16	4,9	g	5,1	18,7	16	4,8	g	6,4	16,5	15	4,5	d	6,9	13,4	15
220x65x2.0	5,4	g	3,7	22,1	18	5,3	g	5,5	19,6	17	5,1	g	7,0	17,6	17	4,8	d	7,0	13,4	15
220x65x2.4	5,7	g	4,0	23,1	19	5,5	g	6,0	20,6	18	5,4	g	7,6	18,3	17	5,0	d	7,7	14,1	16
250x65x2.4	6,3	g	4,6	25,5	21	6,1	g	6,8	22,6	20	5,9	g	8,5	20,0	19	5,5	d	8,0	14,2	18
300x65x2.4	7,3	g	5,4	29,3	24	7,1	g	8,0	25,8	23	6,7	d	9,4	21,2	22	6,1	d	8,5	14,5	20
300x65x3.0	7,8	g	6,0	31,0	25	7,5	g	8,8	27,1	25	7,2	d	10,6	23,0	24	6,6	d	9,6	15,9	21

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 1.5 m
- Steel strength S 350
- Stud resistance 18 kN

Beam is **unpropped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table 7.6 Load-span design table for double Generic C section: unpropped – f_y = 350 N/mm^2

IMPOSED LOAD kN/m^2	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
100x55x1.2	2,5	g	1,4	10,5	8	2,4	g	2,1	9,5	7	2,3	g	2,7	8,6	7	2,3	g	3,2	7,9	7
125x55x1.2	2,9	g	1,7	12,4	9	2,8	g	2,6	11,2	9	2,8	g	3,3	10,1	9	2,7	g	3,9	9,3	9
125x55x1.6	3,2	g	2,0	13,3	10	3,1	g	3,0	11,8	10	3,0	g	3,8	10,8	10	2,9	g	4,4	9,7	9
150x65x1.6	3,8	g	2,5	15,6	12	3,7	g	3,7	13,9	12	3,6	g	4,7	12,5	11	3,5	g	5,5	11,3	11
150x65x1.8	3,9	g	2,6	16,0	12	3,8	g	3,9	14,3	12	3,7	g	4,9	12,8	12	3,6	g	5,8	11,6	11
165x65x1.6	4,1	g	2,8	17,7	13	3,9	g	4,0	14,9	13	3,8	g	5,1	13,3	12	3,7	g	6,0	12,1	12
165x65x1.8	4,2	g	2,8	17,3	13	4,1	g	4,2	15,2	13	4,0	g	5,4	13,6	13	3,9	g	6,3	12,3	12
180x65x1.6	4,3	g	2,9	17,8	14	4,2	g	4,3	15,8	13	4,1	g	5,6	14,4	13	4,0	g	6,5	12,8	13
180x65x1.8	4,5	g	3,1	18,3	14	4,3	g	4,6	16,2	14	4,2	g	5,8	14,5	14	4,1	g	6,8	13,1	13
180x65x2.0	4,6	g	3,2	18,9	15	4,5	g	4,8	16,7	14	4,4	g	6,1	14,9	14	4,2	g	7,1	13,4	14
200x65x1.6	4,7	g	3,2	19,2	15	4,6	g	4,8	17,1	15	4,4	g	6,1	15,2	14	4,3	g	7,1	13,8	14
200x65x1.8	4,9	g	3,4	19,8	16	4,7	g	5,1	17,6	15	4,6	g	6,4	15,6	15	4,5	g	7,5	14,1	14
200x65x2.0	5,0	g	3,6	20,4	16	4,9	g	5,3	17,9	16	4,7	g	6,7	16,1	15	4,6	g	7,8	14,4	15
220x65x2.0	5,4	g	3,9	21,8	17	5,2	g	5,8	19,2	17	5,1	g	7,3	17,1	16	4,9	g	8,5	15,4	16
220x65x2.4	5,7	g	4,2	22,9	18	5,5	g	6,3	20,1	18	5,4	g	7,9	17,9	17	5,2	g	9,1	16,0	17
250x65x2.4	6,3	g	4,8	25,2	20	6,1	g	7,0	22,1	20	5,9	g	8,8	19,6	19	5,7	g	10,2	17,5	19
300x65x2.4	7,3	g	5,6	28,9	24	7,0	g	8,3	25,2	23	6,8	g	10,4	22,4	22	6,6	g	11,9	19,9	22
300x65x3.0	7,8	g	6,2	30,6	25	7,5	g	9,1	26,7	25	7,3	g	11,3	23,6	24	7,1	g	13,0	20,9	23

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 1.5 m
- Steel strength S 280
- Stud resistance 18 kN

Beam is **propped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table 7.7 Load-span design table for double Generic C section: propped – f_y = 280 N/mm^2

IMPOSED LOAD kN/m^2	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
100x55x1.2	3,3	g	4,2	7,7	11	3,0	g	5,2	5,6	9	2,6	b	4,8	3,7	8	2,2	b	3,9	2,3	7
125x55x1.2	4,0	g	5,2	9,3	13	3,6	g	6,2	6,8	11	3,2	b	6,6	5,1	10	2,8	b	5,4	3,3	9
125x55x1.6	3,9	g	4,9	8,9	12	3,5	g	6,2	6,7	11	3,3	g	6,9	5,3	11	3,1	g	7,3	4,4	10
150x65x1.6	4,5	g	5,8	10,4	15	4,2	g	7,3	7,9	13	3,9	g	8,2	6,3	12	3,6	d	8,3	5,0	12
150x65x1.8	4,6	g	5,9	10,6	15	4,2	g	7,4	8,0	14	4,0	g	8,3	6,4	13	3,7	g	8,7	5,3	12
165x65x1.6	4,9	g	6,2	11,2	16	4,5	g	7,8	8,5	14	4,2	g	8,7	6,7	13	3,8	d	8,2	4,9	12
165x65x1.8	4,9	g	6,3	11,4	16	4,5	g	8,0	8,6	15	4,3	g	8,9	6,9	14	3,9	d	8,8	5,3	13
180x65x1.6	5,2	g	6,6	12,0	17	4,8	g	8,4	9,1	15	4,4	d	9,1	7,0	14	4,0	d	8,2	4,9	13
180x65x1.8	5,3	g	6,7	12,1	17	4,9	g	8,5	9,2	16	4,5	g	9,5	7,4	15	4,1	d	8,8	5,3	13
180x65x2.0	5,3	g	6,8	12,3	17	4,9	g	8,6	9,4	16	4,6	g	9,6	7,5	15	4,3	d	9,3	5,6	14
200x65x1.6	5,6	g	7,2	13,0	18	5,2	g	9,0	9,8	17	4,7	d	9,1	7,0	15	4,2	d	8,2	4,9	14
200x65x1.8	5,7	g	7,3	13,1	18	5,3	g	9,2	10,0	17	4,8	d	9,7	7,5	16	4,4	d	8,8	5,3	14
200x65x2.0	5,8	g	7,4	13,3	19	5,3	g	9,4	10,2	17	5,0	d	10,3	8,0	16	4,5	d	9,3	5,6	15
220x65x2.0	6,2	g	7,9	14,3	20	5,7	g	10,1	10,9	19	5,3	d	10,4	8,0	17	4,8	d	9,4	5,7	15
220x65x2.4	6,4	g	8,2	14,7	21	5,9	g	10,4	11,3	19	5,6	d	11,5	8,9	18	5,0	d	10,4	6,2	16
250x65x2.4	7,1	g	9,0	16,3	23	6,6	g	11,5	12,5	21	6,0	d	11,8	9,1	20	5,5	d	10,6	6,4	18
300x65x2.4	8,1	g	10,3	18,7	27	7,5	d	13,2	14,3	25	6,7	d	12,3	9,5	22	6,1	d	11,2	6,7	20
300x65x3.0	8,5	g	10,8	19,6	28	7,9	g	13,8	15,0	26	7,2	d	14,0	10,8	24	6,6	d	12,7	7,7	21

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 1.5 m
- Steel strength S 350
- Stud resistance 18 kN

Beam is **propped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table 7.8 Load-span design table for double Generic C section: propped – f_y = 350 N/mm^2

IMPOSED LOAD kN/m^2	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
100x55x1.2	3,0	g	3,8	6,8	9	2,7	g	4,8	5,2	9	2,5	g	5,3	4,1	8	2,4	g	5,6	3,4	7
125x55x1.2	3,5	g	4,5	8,1	11	3,2	g	5,7	6,1	10	3,0	g	6,3	4,8	10	2,8	g	6,7	4,0	9
125x55x1.6	3,6	g	4,7	8,4	12	3,4	g	5,9	6,4	11	3,2	g	6,6	5,1	10	3,0	g	7,0	4,2	9
150x65x1.6	4,3	g	5,5	9,9	14	4,0	g	7,0	7,6	13	3,7	g	7,8	6,1	12	3,5	g	8,3	5,0	11
150x65x1.8	4,4	g	5,6	10,1	14	4,1	g	7,2	7,8	13	3,8	g	8,0	6,2	12	3,6	g	8,5	5,1	12
165x65x1.6	4,6	g	5,9	10,6	15	4,3	g	7,5	8,1	14	4,0	g	8,4	6,5	13	3,8	g	8,9	5,3	12
165x65x1.8	4,7	g	6,0	10,8	15	4,4	g	7,7	8,3	14	4,1	g	8,6	6,6	13	3,9	g	9,1	5,5	12
180x65x1.6	4,9	g	6,3	11,3	16	4,6	g	8,0	8,7	15	4,3	g	8,9	6,9	14	4,1	g	9,5	5,7	13
180x65x1.8	5,2	g	7,0	12,7	17	4,7	g	8,2	8,9	15	4,4	g	9,2	7,1	14	4,2	g	9,7	5,8	13
180x65x2.0	5,1	g	6,6	11,8	17	4,8	g	8,3	9,0	15	4,5	g	9,4	7,2	14	4,3	g	10,0	6,0	14
200x65x1.6	5,3	g	6,8	12,3	17	4,9	g	8,6	9,4	16	4,6	g	9,7	7,5	15	4,4	g	10,3	6,2	14
200x65x1.8	5,4	g	6,9	12,5	18	5,0	g	8,8	9,6	16	4,7	g	9,9	7,7	15	4,5	g	10,6	6,4	15
200x65x2.0	5,6	g	7,1	12,8	18	5,2	g	9,1	9,8	17	4,9	g	10,1	7,8	16	4,6	g	10,8	6,5	15
220x65x2.0	6,0	g	7,6	13,8	19	5,5	g	9,7	10,5	18	5,2	g	10,9	8,4	17	5,0	g	11,6	7,0	16
220x65x2.4	6,2	g	7,9	14,3	20	5,8	g	10,2	11,0	19	5,4	g	11,4	8,8	18	5,2	g	12,1	7,3	17
250x65x2.4	6,8	g	8,7	15,7	22	6,4	g	11,2	12,1	21	6,0	g	12,5	9,7	20	5,7	g	13,3	8,0	19
300x65x2.4	7,9	g	10,0	18,1	26	7,3	g	12,9	14,0	24	6,9	g	14,4	11,2	23	6,6	g	15,4	9,3	21
300x65x3.0	8,3	g	10,6	19,2	27	7,7	g	13,6	14,7	25	7,3	g	15,3	11,8	24	6,9	g	16,2	9,8	23

7.4 USE OF DESIGN TABLES

Composite beams design tables are very useful for the engineer in particular during the design stage of a project. They help to get the most economic selection according with the project requirements.

Once the imposed load is determined the engineer has to decide the span - spacing combination more competent. Changing one of these two variables it is possible to get different sections suitable for the load conditions. If the span remains fixed and the spacing is reduced, the tables show the possibility of use a smaller section which obviously is more economic. Hence for a determinate spacing, if a reduction of the steel section is required it is necessary to reduce the span of the beam.

This information can be found within the design tables, however is easier to appreciate these relations if the dates are plotted in graphics (see Figure 7.1).
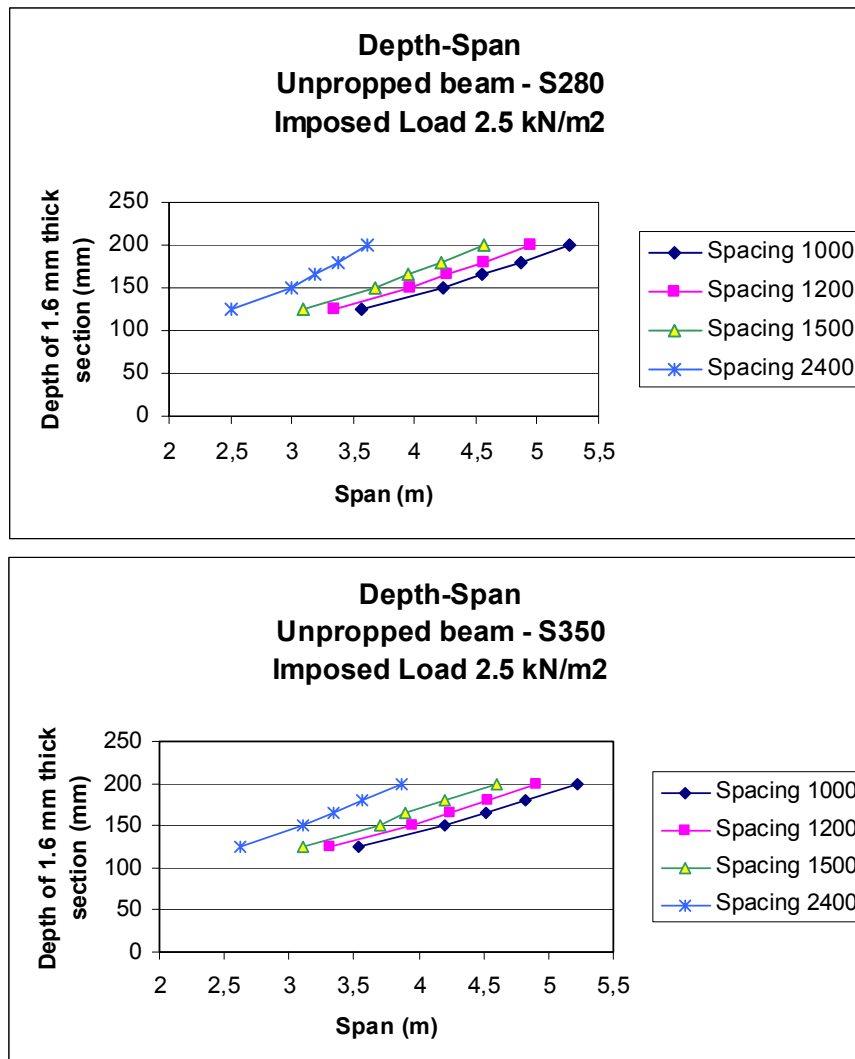
Figure 7.1 Relation between span-depth of 1.6 mm thick section, under an imposed load equal to 2.5 kN/ m^2, depending on the spacing

Other possibility is to know in which imposed load conditions a specific section would respond positively. If the spacing is fixed, the imposed load that the section can resist, varying the span of the beam, can be determinate. This process is very easy with the aid of graphics (see Figure 7.2).

Imposed Load-Span
Spacing 1500 mm
Propped - S280
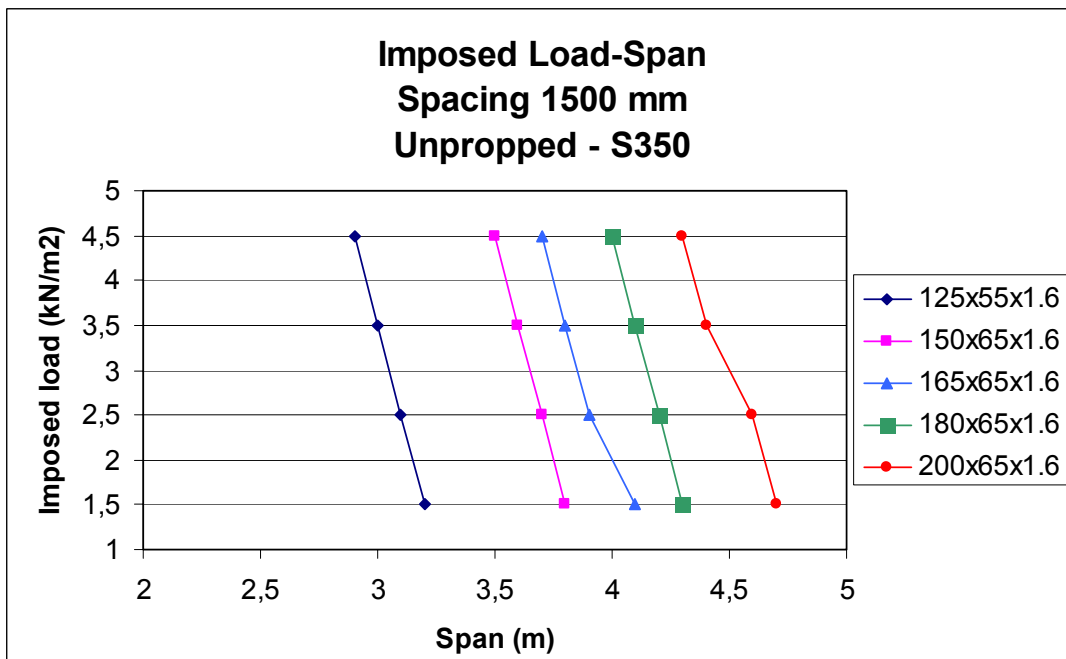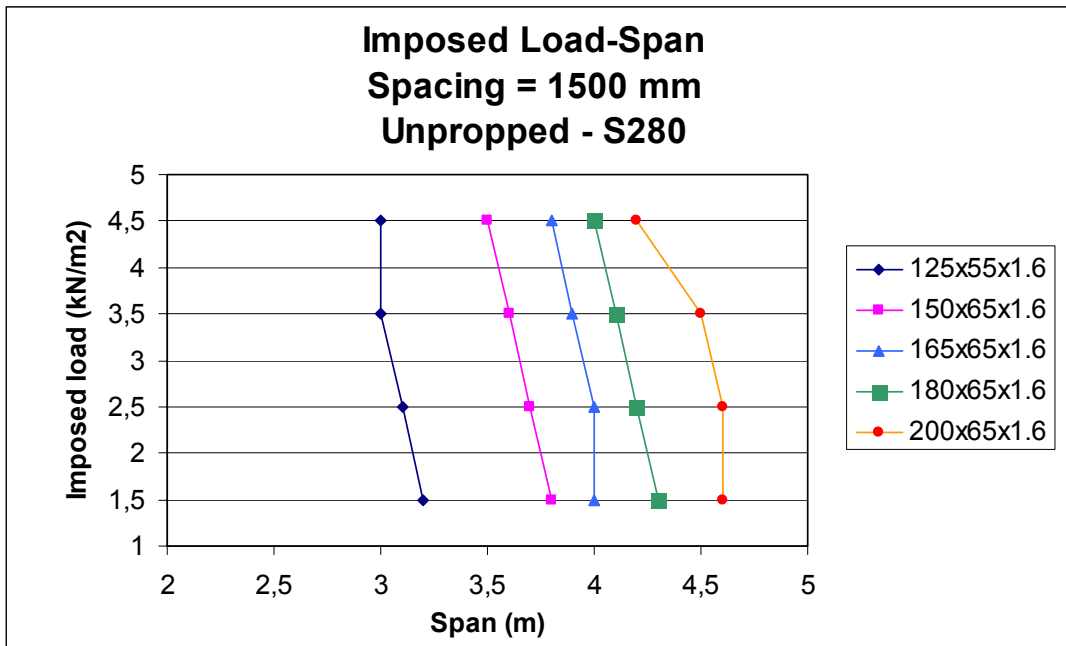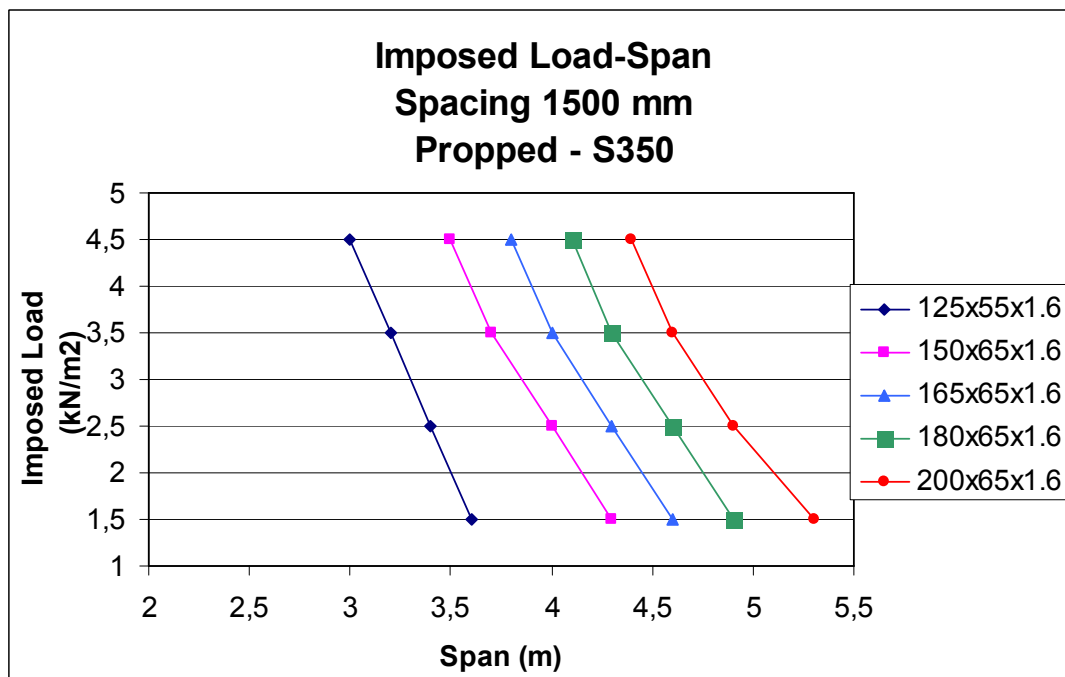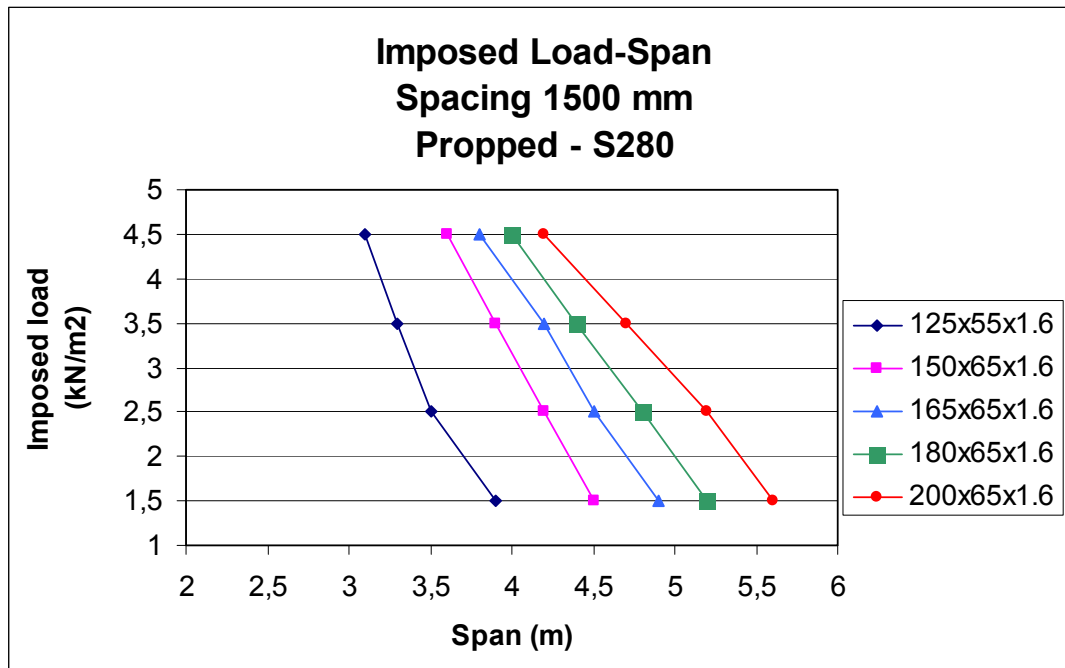
Figure 7.2 Relation between span-imposed load with spacing equal to 1500 mm for different sections.

Design tables show all the combinations that are possible warranting a good behaviour, and using them it is easier to determine which combination is the right one.

8.   CONCLUSIONS

A wide range of possibilities of cold formed sections and light steel framing has been developed in recent years. It is appreciated that the design of these sections may appear to be more complicated than that of hot rolled sections. It is therefore important to understand the design principles and also the practical considerations of the structural use of these sections.

Composite construction is widespread in building practice and the development of light steel composite structures has a great demand because the clear benefits that they incorporate. Using light cold formed sections of 1.2 to 3.0 mm thickness as composite beams, means an economic benefit due to the reduction of material involved.

Differences in design of composite beams with light cold formed sections compared to hot rolled steel sections exist. Light steel sections are slender and the possibility of local buckling has to be checked. However working composite with in-situ concrete has being demonstrated that local buckling of light steel sections is not a common design criterion. Other point which has great importance is the type of shear connectors used. As light steel sections have so thin steel elements, has been necessary to design innovative shear connectors and to carry out several tests with the purpose of improve their performance and determine their design resistance.

Knowing light steel sections special properties, secondary beams of a composite slab have been designed to study the performance of these thin sections working compositely with in situ concrete. Design tables, which are very useful in the design stage, have been computed.

The design tables shown cover a limited range of light steel double Generic C sections possibilities, specially because the beams spacing has been fixed to compare how the span vary with the dimension of the section. However some general conclusions can be observed through design tables:

•   When the steel grade of the sections is S280 and the imposed loads are quite low, with propped construction greater spans can be designed. The reason is that in unpropped construction beams fail because of the moment resistance in construction stage is exceeded.

If imposed loads increase there is almost not difference between the span that can be achieved with propped construction compared to the unpropped one, in both cases total deflections and moment resistance with partial shear connection limit the beam span. When the beam is propped during construction, all the loads have to be resisted by the composite section and this impedes to get greater spans.

•   If the steel of the sections has higher yield strength, grade S350, in propped and unpropped construction and with different imposed loads, the beam span is limited by total deflections. The Eurocode restrictions for total deflections of composite beams

depend on whether props have been used during the constructions :

~   Unpropped construction, total deflection $< L_E/200$
~   Propped construction, total deflection $< L_E/250$

Although in propped construction lower total deflections are achieved, these regulation limits allow greater spans in unpropped construction than in propped one under high imposed loads. This effect is not presented with lower loads due to the bigger spans that with props can be achieved.

Once the imposed load is fixed and knowing the total dimension of the composite slab, the engineer has to select the beams spacing and span. With these decisions taken, design tables give the dimensions, depth and thickness, of the section that will resist these conditions. Light steel sections of 1.2 to 3.0 mm thickness look weak because their slenderness, but they can support imposed loads about 4.5 kN/m^2 with spans of more than 8 m.

Comparing the design tables of light steel sections and heavy cold formed sections (H-CFS), it is possible to observe that greater spans can be achieved using these last ones. But often the spans that light steel sections can offer satisfy the construction demand. H-CFS use steel of 3 to 6 mm thickness so the cost that can be saved by the reduction in weight of material is lower than using light steel sections. Hence using light steel sections when the construction requirements allow it, increase economic and environmental benefits.

9.   BIBLIOGRAPHIC REFERENCES

1.   Galjaard, H. C., Walraven, J. C. (2001), *Static tests on various types of shear connectors for composite structures- Connections between Steel and Concrete*, edited by R. Eligehausen, Vol. Two. University of Stuttgart, Germany.

2.   ENV 1993-1-1 (1993) Eurocode 3. *Design of steel structures. Part 1-1: General Rules and Rules for Buildings*. European Committee for Standardisation (CEN).

3.   ENV 1994-1-1 (1994) Eurocode 4. *Design of composite steel and concrete structures. Part 1-1: General Rules and Rules for Buildings*. European Committee for Standardisation (CEN) .

4.   ENV 1994-2 (1997) Eurocode 4. *Design of composite steel and concrete structures. Part 2: Composite bridges*. European Committee for Standardisation (CEN).

5.   Fontana, M., Beck, H., Bärtschi, R. (2001), *Experimental investigation on the behaviour of strip shear connectors with powder actuated fasteners-Connections between Steel and Concrete*, edited by R. Eligehausen, Vol. Two. University of Stuttgart, Germany.

6.   Lawson, R.M., Chung, K.F., Popo-Ola, S.O. (2002), *Building Design Using Cold Formed Steel Sections-Section Properties and Load Tables,* The Steel Construction Institute.

7.   Lawson, R.M., Popo-Ola, S.O. (2002), *Opportunities for Heavy Cold Formed Steel Sections,* The Steel Construction Institute.

8.   Lawson, R.M., Popo-Ola, S.O. (1998), *Push-out tests with strip connector fastened with HILTI X-EDNK22 pins into cold formed steel sections,* The Steel Construction Institute.

10. SUPPLEMENTARY BIBLIOGRAPHY

- "Cold-Formed Steel Design", Second Edition. Edited by Wei-Wen, Yu, 1991. New York. Wiley Publications.
- "Composite Beam Design to Eurocode 4". Edited by R. M. Lawson and K. F. Chung, 1994. Steel Construction Institute.
- "Composite Slabs and Beams using Steel Decking: best practice for design and construction". Edited by G. H. Couchman, D. L. Mullett and J. W. Rackham, 2000. The Metal Cladding & Roofing Manufacturers Association.
- "Design of Composite Beams Using Precast Concrete Slabs". Edited by S. J. Hicks and R. M. Lawson, 2003. Steel Construction Institute.

ANNEX 1

GENERIC C SECTIONS PROPERTIES

GENERIC C SECTIONS

GROSS SECTION PROPERTIES

Single section
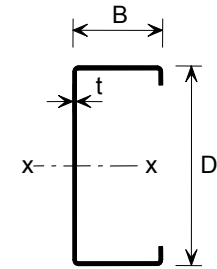
Table A1.1 Section properties of Generic C single sections

D (mm)	B (mm)	T (mm)	Area (cm^2)	Weight (kg/m)	I_{xx} (cm^4)	I_{yy} (cm^4)	r_{xx} (cm)	r_{yy} (cm)	W_{xx} (cm^3)	W_{yy} (cm^3)	J (cm^4)	C_w (cm^6)	e_s (mm)
100	55	1,2	2,67	2,09	44,3	11,5	4,07	2,07	8,87	3,22	0,0120	255	-26,6
125	55	1,2	2,96	2,32	73,9	12,4	5,00	2,05	11,83	3,31	0,0133	401	-25,1
125	55	1,6	3,93	3,08	97,0	16,1	4,97	2,02	15,52	4,28	0,0319	513	-24,6
150	65	1,6	4,63	3,63	165,4	25,6	5,98	2,35	22,06	5,66	0,0376	1127	-28,1
150	65	1,8	5,19	4,08	184,7	28,4	5,96	2,34	24,63	6,27	0,0536	1244	-27,8
165	65	1,6	4,86	3,82	206,2	26,4	6,51	2,33	25,00	5,72	0,0394	1387	-27,3
165	65	1,8	5,46	4,28	230,4	29,3	6,50	2,32	27,93	6,34	0,0563	1532	-27,1
180	65	1,6	5,10	4,00	252,5	27,1	7,04	2,31	28,06	5,77	0,0413	1681	-26,6
180	65	1,8	5,72	4,49	282,2	30,1	7,02	2,29	31,36	6,40	0,0591	1856	-26,4
180	65	2,0	6,34	4,98	311,2	33,0	7,01	2,28	34,59	7,00	0,0812	2023	-26,1
200	65	1,6	5,41	4,25	323,2	28,0	7,73	2,28	32,32	5,83	0,0439	2125	-25,7
200	65	1,8	6,07	4,77	361,3	31,1	7,71	2,26	36,14	6,47	0,0627	2348	-25,5
200	65	2,0	6,73	5,28	398,7	34,0	7,70	2,25	39,87	7,08	0,0862	2560	-25,2
220	65	2,0	7,12	5,59	499,6	35,0	8,37	2,22	45,43	7,14	0,0912	3174	-24,4
220	65	2,4	8,50	6,67	590,9	40,7	8,34	2,19	53,73	8,30	0,1578	3664	-23,8
250	65	2,4	9,21	7,23	802,8	42,1	9,34	2,14	64,23	8,40	0,1709	4903	-22,7
300	65	2,4	10,39	8,15	1249,4	44,1	10,97	2,06	83,31	8,53	0,1928	7444	-21,0
300	65	3,0	12,88	10,11	1531,2	52,4	10,90	2,02	102,09	10,14	0,3762	8780	-20,2
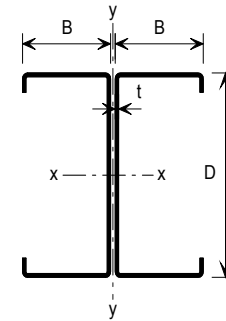
Double section

Table A1.2 Section properties of double Generic C sections

D (mm)	B (mm)	T (mm)	Area (cm²)	Weight (kg/m)	I_{xx} (cm⁴)	I_{yy} (cm⁴)	r_{xx} (cm)	r_{yy} (cm)	W_{xx} (cm³)	W_{yy} (cm³)	J (cm⁴)	C_w (cm⁶)	e_s (mm)
100	55	1,2	5,34	4,19	88,6	42,8	4,07	2,83	17,73	7,79	0,0239	1135	-
125	55	1,2	5,92	4,64	147,8	42,9	5,00	2,69	23,66	7,79	0,0265	1733	-
125	55	1,6	7,85	6,17	194,0	55,9	4,97	2,67	31,05	10,17	0,0637	2198	-
150	65	1,6	9,26	7,27	330,9	87,0	5,98	3,07	44,13	13,39	0,0751	4869	-
150	65	1,8	10,39	8,15	369,4	96,9	5,96	3,05	49,27	14,90	0,1072	5353	-
165	65	1,6	9,73	7,63	412,5	87,0	6,51	2,99	50,01	13,39	0,0789	5859	-
165	65	1,8	10,91	8,57	460,7	96,9	6,50	2,98	55,86	14,90	0,1127	6443	-
180	65	1,6	10,19	8,00	505,0	87,0	7,04	2,92	56,13	13,39	0,0827	6940	-
180	65	1,8	11,44	8,98	564,3	96,9	7,02	2,91	62,72	14,91	0,1181	7634	-
180	65	2,0	12,68	9,95	622,4	106,4	7,01	2,90	69,17	16,37	0,1623	8285	-
200	65	1,6	10,82	8,49	646,4	87,1	7,73	2,84	64,65	13,39	0,0878	8524	-
200	65	1,8	12,15	9,54	722,6	96,9	7,71	2,82	72,27	14,91	0,1254	9379	-
200	65	2,0	13,46	10,57	797,3	106,4	7,70	2,81	79,75	16,38	0,1724	10183	-
220	65	2,0	14,25	11,18	999,2	106,5	8,37	2,73	90,85	16,38	0,1824	12277	-
220	65	2,4	17,00	13,34	1181,9	124,7	8,34	2,71	107,46	19,18	0,3155	14042	-
250	65	2,4	18,41	14,45	1605,5	124,7	9,34	2,60	128,46	19,19	0,3418	18067	-
300	65	2,4	20,77	16,31	2498,9	124,8	10,97	2,45	166,61	19,19	0,3857	25902	-
300	65	3,0	25,76	20,22	3062,3	150,1	10,90	2,41	204,18	23,09	0,7523	30070	-

NOTE: Gross sections properties are independent on the design yield strength, so these properties tables can be used for steel grade S280 and S350 N/mm².

ANNEX 2

SECTIONS CLASS CLASIFICATION

A2.1   SINGLE GENERIC C SECTION CLASS CLASSIFICATION:

The elements of the single Generic C sections, flange and web, have been class classification according to the Eurocode limits.

Table A2.1 Single Generic C section class classification (f_y= 280 N/mm^2, $\varepsilon = 0.92$)

D (mm)	B (mm)	t (mm)	c (mm)	c/tf	Flange	d/t$_w$	Web under Bending	Web under Compression	CLASS
100	55	1,2	27,5	22,92	class4	83,33	-	class4	class4
125	55	1,2	27,5	22,92	class4	104,17	-	class4	class4
125	55	1,6	27,5	17,19	class4	78,13	-	class4	class4
150	65	1,6	32,5	20,31	class4	93,75	-	class4	class4
150	65	1,8	32,5	18,06	class4	83,33	-	class4	class4
165	65	1,6	32,5	20,31	class4	103,13	-	class4	class4
165	65	1,8	32,5	18,06	class4	91,67	-	class4	class4
180	65	1,6	32,5	20,31	class4	112,50	-	class4	class4
180	65	1,8	32,5	18,06	class4	100,00	-	class4	class4
180	65	2,0	32,5	16,25	class4	90,00	-	class4	class4
200	65	1,6	32,5	20,31	class4	125,00	class4	class4	class4
200	65	1,8	32,5	18,06	class4	111,11	-	class4	class4
200	65	2,0	32,5	16,25	class4	100,00	-	class4	class4
220	65	2,0	32,5	16,25	class4	110,00	-	class4	class4
220	65	2,4	32,5	13,54	-	91,67	-	class4	class4
250	65	2,4	32,5	13,54	-	104,17	-	class4	class4
300	65	2,4	32,5	13,54	-	125,00	class4	class4	class4
300	65	3,0	32,5	10,83	-	100,00	-	class4	class4

Table A2.2 Single Generic C section class classification (f_y= 350 N/mm^2, $\varepsilon = 0.82$)

D (mm)	B (mm)	t (mm)	c (mm)	c/tf	Flange	d/t$_w$	Web under Bending	Web under Compression	CLASS
100	55	1,2	27,5	22,92	class4	83,33	-	class4	class4
125	55	1,2	27,5	22,92	class4	104,17	class4	class4	class4
125	55	1,6	27,5	17,19	class4	78,13	-	class4	class4
150	65	1,6	32,5	20,31	class4	93,75	-	class4	class4
150	65	1,8	32,5	18,06	class4	83,33	-	class4	class4
165	65	1,6	32,5	20,31	class4	103,13	class4	class4	class4
165	65	1,8	32,5	18,06	class4	91,67	-	class4	class4
180	65	1,6	32,5	20,31	class4	112,50	class4	class4	class4
180	65	1,8	32,5	18,06	class4	100,00	-	class4	class4
180	65	2,0	32,5	16,25	class4	90,00	-	class4	class4
200	65	1,6	32,5	20,31	class4	125,00	class4	class4	class4
200	65	1,8	32,5	18,06	class4	111,11	class4	class4	class4
200	65	2,0	32,5	16,25	class4	100,00	-	class4	class4
220	65	2,0	32,5	16,25	class4	110,00	class4	class4	class4
220	65	2,4	32,5	13,54	class4	91,67	-	class4	class4
250	65	2,4	32,5	13,54	class4	104,17	class4	class4	class4
300	65	2,4	32,5	13,54	class4	125,00	class4	class4	class4
300	65	3,0	32,5	10,83	-	100,00	-	class4	class4

A2.2   COMPOSITE BEAMS CROSS SECTIONS CLASS CLASSIFICATION

If the plastic neutral axis depth since the upper surface of the slab, y_p, is smaller than the concrete depth above the steel decking, 80 mm, the composite cross section is classified as Class 1. With the compressive resistance of the concrete slab (R_c) and the tensile resistance of the steel section (R_s) is possible to calculate the depth of the plastic neutral axis y_p.

Table A2.3 Composite cross section class checking: unpropped - $f_y = 280$ N/mm^2

IMPOSED LOAD kN/m^2	1.5				2.5				3.5				4.5			
DESIGNATION	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)
100x55x1.2	2,4	816	142,3	14,0	2,4	816	142,3	14,0	2,4	816	142,3	14,0	2,2	748	142,3	15,2
125x55x1.2	2,8	952	157,8	13,3	2,8	952	157,8	13,3	2,8	952	157,8	13,3	2,7	918	157,8	13,8
125x55x1.6	3,2	1088	209,4	15,4	3,1	1054	209,4	15,9	3,0	1020	209,4	16,4	3,0	1020	209,4	16,4
150x65x1.6	3,8	1292	246,9	15,3	3,7	1258	246,9	15,7	3,6	1224	246,9	16,1	3,5	1190	246,9	16,6
150x65x1.8	3,9	1326	277,0	16,7	3,9	1326	277,0	16,7	3,7	1258	277,0	17,6	3,6	1224	277,0	18,1
165x65x1.6	4,0	1360	259,4	15,3	4,0	1360	259,4	15,3	3,9	1326	259,4	15,6	3,8	1292	259,4	16,1
165x65x1.8	4,2	1428	291,1	16,3	4,1	1394	291,1	16,7	4,0	1360	291,1	17,1	3,9	1326	291,1	17,6
180x65x1.6	4,3	1462	271,8	14,9	4,2	1428	271,8	15,2	4,1	1394	271,8	15,6	4,0	1360	271,8	16,0
180x65x1.8	4,5	1530	305,1	16,0	4,4	1496	305,1	16,3	4,3	1462	305,1	16,7	4,1	1394	305,1	17,5
180x65x2.0	4,6	1564	338,1	17,3	4,5	1530	338,1	17,7	4,4	1496	338,1	18,1	4,3	1462	338,1	18,5
200x65x1.6	4,6	1564	288,5	14,8	4,6	1564	288,5	14,8	4,5	1530	288,5	15,1	4,2	1428	288,5	16,2
200x65x1.8	4,8	1632	323,9	15,9	4,8	1632	323,9	15,9	4,6	1564	323,9	16,6	4,4	1496	323,9	17,3
200x65x2.0	5,0	1700	359,0	16,9	4,9	1666	359,0	17,2	4,8	1632	359,0	17,6	4,5	1530	359,0	18,8
220x65x2.0	5,4	1836	379,9	16,6	5,3	1802	379,9	16,9	5,1	1734	379,9	17,5	4,8	1632	379,9	18,6
220x65x2.4	5,7	1938	453,2	18,7	5,5	1870	453,2	19,4	5,4	1836	453,2	19,7	5,0	1700	453,2	21,3
250x65x2.4	6,3	2142	491,0	18,3	6,1	2074	491,0	18,9	5,9	2006	491,0	19,6	5,5	1870	491,0	21,0
300x65x2.4	7,3	2482	553,9	17,9	7,1	2414	553,9	18,4	6,7	2278	553,9	19,5	6,1	2074	553,9	21,4
300x65x3.0	7,8	2652	686,9	20,7	7,5	2550	686,9	21,6	7,2	2448	686,9	22,4	6,6	2244	686,9	24,5

Table A2.4 Composite cross section class checking: unpropped - $f_y = 350$ N/mm^2

IMPOSED LOAD kN/m^2	1.5				2.5				3.5				4.5			
DESIGN.	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)
100x55x1.2	2,5	850	177,9	16,7	2,4	816	177,9	17,4	2,3	782	177,9	18,2	2,3	782	177,9	18,2
125x55x1.2	2,9	986	197,2	16,0	2,8	952	197,2	16,6	2,8	952	197,2	16,6	2,7	918	197,2	17,2
125x55x1.6	3,2	1088	261,8	19,3	3,1	1054	261,8	19,9	3,0	1020	261,8	20,5	2,9	986	261,8	21,2
150x65x1.6	3,8	1292	308,6	19,1	3,7	1258	308,6	19,6	3,6	1224	308,6	20,2	3,5	1190	308,6	20,7
150x65x1.8	3,9	1326	346,2	20,9	3,8	1292	346,2	21,4	3,7	1258	346,2	22,0	3,6	1224	346,2	22,6
165x65x1.6	4,1	1394	324,2	18,6	3,9	1326	324,2	19,6	3,8	1292	324,2	20,1	3,7	1258	324,2	20,6
165x65x1.8	4,2	1428	363,8	20,4	4,1	1394	363,8	20,9	4,0	1360	363,8	21,4	3,9	1326	363,8	21,9
180x65x1.6	4,3	1462	339,8	18,6	4,2	1428	339,8	19,0	4,1	1394	339,8	19,5	4,0	1360	339,8	20,0
180x65x1.8	4,5	1530	381,4	19,9	4,3	1462	381,4	20,9	4,2	1428	381,4	21,4	4,1	1394	381,4	21,9
180x65x2.0	4,6	1564	422,6	21,6	4,5	1530	422,6	22,1	4,4	1496	422,6	22,6	4,2	1428	422,6	23,7
200x65x1.6	4,7	1598	360,6	18,1	4,6	1564	360,6	18,4	4,4	1496	360,6	19,3	4,3	1462	360,6	19,7
200x65x1.8	4,9	1666	404,9	19,4	4,7	1598	404,9	20,3	4,6	1564	404,9	20,7	4,5	1530	404,9	21,2
200x65x2.0	5,0	1700	448,7	21,1	4,9	1666	448,7	21,5	4,7	1598	448,7	22,5	4,6	1564	448,7	23,0
220x65x2.0	5,4	1836	474,9	20,7	5,2	1768	474,9	21,5	5,1	1734	474,9	21,9	4,9	1666	474,9	22,8
220x65x2.4	5,7	1938	566,6	23,4	5,5	1870	566,6	24,2	5,4	1836	566,6	24,7	5,2	1768	566,6	25,6
250x65x2.4	6,3	2142	613,8	22,9	6,1	2074	613,8	23,7	5,9	2006	613,8	24,5	5,7	1938	613,8	25,3
300x65x2.4	7,3	2482	692,4	22,3	7,0	2380	692,4	23,3	6,8	2312	692,4	24,0	6,6	2244	692,4	24,7
300x65x3.0	7,8	2652	858,6	25,9	7,5	2550	858,6	26,9	7,3	2482	858,6	27,7	7,1	2414	858,6	28,5

Table A2.5 Composite cross section class checking: propped - $f_y = 280$ N/mm^2

IMPOSED LOAD kN/m^2	1.5				2.5				3.5				4.5			
DESIGN.	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)
100x55x1.2	3,3	1122	142,3	10,1	3,0	1020	142,3	11,2	2,6	884	142,3	12,9	2,2	748	142,3	15,2
125x55x1.2	4,0	1360	157,8	9,3	3,6	1224	157,8	10,3	3,2	1088	157,8	11,6	2,8	952	157,8	13,3
125x55x1.6	3,9	1326	209,4	12,6	3,5	1190	209,4	14,1	3,3	1122	209,4	14,9	3,1	1054	209,4	15,9
150x65x1.6	4,5	1530	246,9	12,9	4,2	1428	246,9	13,8	3,9	1326	246,9	14,9	3,6	1224	246,9	16,1
150x65x1.8	4,6	1564	277,0	14,2	4,2	1428	277,0	15,5	4,0	1360	277,0	16,3	3,7	1258	277,0	17,6
165x65x1.6	4,9	1666	259,4	12,5	4,5	1530	259,4	13,6	4,2	1428	259,4	14,5	3,8	1292	259,4	16,1
165x65x1.8	4,9	1666	291,1	14,0	4,5	1530	291,1	15,2	4,3	1462	291,1	15,9	3,9	1326	291,1	17,6
180x65x1.6	5,2	1768	271,8	12,3	4,8	1632	271,8	13,3	4,4	1496	271,8	14,5	4,0	1360	271,8	16,0
180x65x1.8	5,3	1802	305,1	13,5	4,9	1666	305,1	14,7	4,5	1530	305,1	16,0	4,1	1394	305,1	17,5
180x65x2.0	5,3	1802	338,1	15,0	4,9	1666	338,1	16,2	4,6	1564	338,1	17,3	4,3	1462	338,1	18,5
200x65x1.6	5,6	1904	288,5	12,1	5,2	1768	288,5	13,1	4,7	1598	288,5	14,4	4,2	1428	288,5	16,2
200x65x1.8	5,7	1938	323,9	13,4	5,3	1802	323,9	14,4	4,8	1632	323,9	15,9	4,4	1496	323,9	17,3
200x65x2.0	5,8	1972	359,0	14,6	5,3	1802	359,0	15,9	5,0	1700	359,0	16,9	4,5	1530	359,0	18,8
220x65x2.0	6,2	2108	379,9	14,4	5,7	1938	379,9	15,7	5,3	1802	379,9	16,9	4,8	1632	379,9	18,6
220x65x2.4	6,4	2176	453,2	16,7	5,9	2006	453,2	18,1	5,6	1904	453,2	19,0	5,0	1700	453,2	21,3
250x65x2.4	7,1	2414	491,0	16,3	6,6	2244	491,0	17,5	6,0	2040	491,0	19,3	5,5	1870	491,0	21,0
300x65x2.4	8,1	2754	553,9	16,1	7,5	2550	553,9	17,4	6,7	2278	553,9	19,5	6,1	2074	553,9	21,4
300x65x3.0	8,5	2890	686,9	19,0	7,9	2686	686,9	20,5	7,2	2448	686,9	22,4	6,6	2244	686,9	24,5

Table A2.6 Composite cross section class checking: propped - $f_y = 350$ N/mm^2

IMPOSED LOAD kN/m^2	1.5				2.5				3.5				4.5			
DESIGN.	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)	L_E (m)	R_c (kN)	R_s (kN)	y_p (mm)
100x55x1.2	3,0	1020	177,9	14,0	2,7	918	177,9	15,5	2,5	850	177,9	16,7	2,4	816	177,9	17,4
125x55x1.2	3,5	1190	197,2	13,3	3,2	1088	197,2	14,5	3,0	1020	197,2	15,5	2,8	952	197,2	16,6
125x55x1.6	3,6	1224	261,8	17,1	3,4	1156	261,8	18,1	3,2	1088	261,8	19,3	3,0	1020	261,8	20,5
150x65x1.6	4,3	1462	308,6	16,9	4,0	1360	308,6	18,2	3,7	1258	308,6	19,6	3,5	1190	308,6	20,7
150x65x1.8	4,4	1496	346,2	18,5	4,1	1394	346,2	19,9	3,8	1292	346,2	21,4	3,6	1224	346,2	22,6
165x65x1.6	4,6	1564	324,2	16,6	4,3	1462	324,2	17,7	4,0	1360	324,2	19,1	3,8	1292	324,2	20,1
165x65x1.8	4,7	1598	363,8	18,2	4,4	1496	363,8	19,5	4,1	1394	363,8	20,9	3,9	1326	363,8	21,9
180x65x1.6	4,9	1666	339,8	16,3	4,6	1564	339,8	17,4	4,3	1462	339,8	18,6	4,1	1394	339,8	19,5
180x65x1.8	5,2	1768	381,4	17,3	4,7	1598	381,4	19,1	4,4	1496	381,4	20,4	4,2	1428	381,4	21,4
180x65x2.0	5,1	1734	422,6	19,5	4,8	1632	422,6	20,7	4,5	1530	422,6	22,1	4,3	1462	422,6	23,1
200x65x1.6	5,3	1802	360,6	16,0	4,9	1666	360,6	17,3	4,6	1564	360,6	18,4	4,4	1496	360,6	19,3
200x65x1.8	5,4	1836	404,9	17,6	5,0	1700	404,9	19,1	4,7	1598	404,9	20,3	4,5	1530	404,9	21,2
200x65x2.0	5,6	1904	448,7	18,9	5,2	1768	448,7	20,3	4,9	1666	448,7	21,5	4,6	1564	448,7	23,0
220x65x2.0	6,0	2040	474,9	18,6	5,5	1870	474,9	20,3	5,2	1768	474,9	21,5	5,0	1700	474,9	22,3
220x65x2.4	6,2	2108	566,6	21,5	5,8	1972	566,6	23,0	5,4	1836	566,6	24,7	5,2	1768	566,6	25,6
250x65x2.4	6,8	2312	613,8	21,2	6,4	2176	613,8	22,6	6,0	2040	613,8	24,1	5,7	1938	613,8	25,3
300x65x2.4	7,9	2686	692,4	20,6	7,3	2482	692,4	22,3	6,9	2346	692,4	23,6	6,6	2244	692,4	24,7
300x65x3.0	8,3	2822	858,6	24,3	7,7	2618	858,6	26,2	7,3	2482	858,6	27,7	6,9	2346	858,6	29,3

ANNEX 3

HEAVY COLD FORMED SECTIONS

A3.1   HEAVY COLD FORMED SECTIONS

Construction and Engineering Products (CEP) is a specialist roll-forming division within the Corus Special Strip Business Unit, located in Newport, South Wales. They are studying the opportunity for use Heavy Cold Formed Sections (H-CFS) in the building construction industry.

H-CFS use steel of 3 to 6 mm thickness. The market assessment shows that there is potential for the use of these heavier cold formed steel sections for applications which lie between the existing light steel cold formed sections and hot rolled sections.

The comparison made between proposed H-CFS and existing hot rolled sections demonstrates that, for similar second moment of inertia, the corresponding weight of H-CFS is less than those of hot rolled sections. Hence cost can be saved by the reduction in weight of material and the H-CFS can be used for longer span applications than the existing cold rolled sections.

In previous projects H-CFS properties have been provided by Popo-Ola and Lawson (2002). Using these properties, the performance of H-CFS as composite beams has been studied considering H-CFS as secondary beams of a composite slab, fixed by Hilti X-HVB shear connectors and a beams spacing equal to 2.4 m.
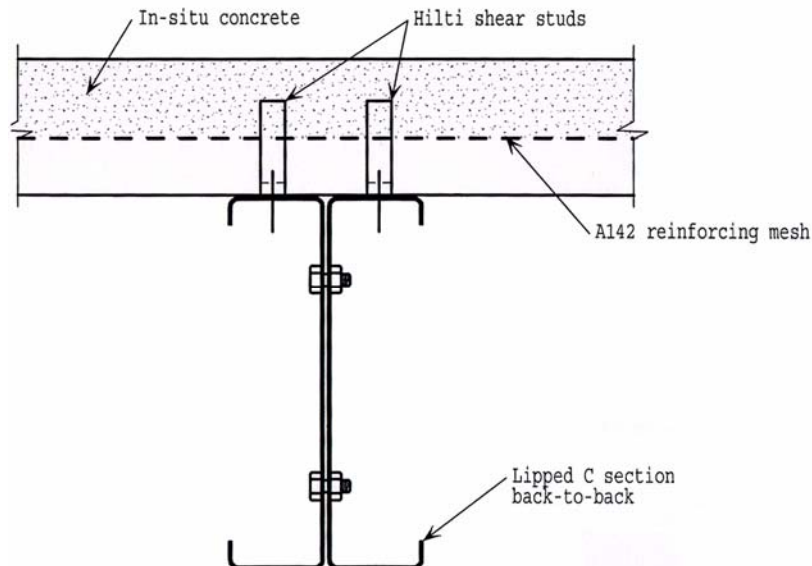
Figure A3.1 Composite beam: H-CFS lipped double C section and Hilti X-HVB shear connectors

Tables A3.1 and A3.2 show the values of the double lipped C and double extended C, heavy cold formed sections properties used for this study.

Tables A3.3 and A3.4 show the results of the span-load calculations carried out. If these results are compared with the results of light steel analysis (see tables 7.5 to 7.8), it is appreciable that bigger spans with greater beams spacing can be achieved using H-CFS. However light steel sections means more economic benefits than H-CFS.

Table A3.1 Section properties of heavy cold formed double lipped C sections

D (mm)	B (mm)	t (mm)	Area (cm^2)	Weight (kg/m)	I_{xx} (cm^4)	I_{yy} (cm^4)	r_{xx} (cm)	r_{yy} (cm)	W_{xx} (cm^3)	W_{yy} (cm^3)	J (cm^4)	C_w (cm^6)	e_s (mm)
200	75	3,0	21,02	16,50	1263,7	222,1	7,75	3,25	126,4	29,6	0,61	20209	-
200	75	4,0	27,47	21,56	1615,8	278,3	7,67	3,18	161,6	37,1	1,44	23827	-
200	75	4,5	30,57	24,00	1778,0	302,9	7,63	3,15	177,8	40,4	2,03	25109	-
250	75	3,0	23,98	18,83	2143,7	222,2	9,45	3,04	171,5	29,6	0,70	31455	-
250	75	4,0	31,43	24,67	2753,8	278,5	9,36	2,98	220,3	37,1	1,64	37218	-
250	75	4,5	35,03	27,50	3037,6	303,2	9,31	2,94	243,0	40,4	2,32	39299	-
300	75	3,0	26,94	21,15	3323,5	222,3	11,11	2,87	221,6	29,6	0,79	45183	-
300	75	4,0	35,39	27,78	4284,6	278,7	11,00	2,81	285,7	37,2	1,85	53590	-
300	75	4,5	39,49	31,00	4735,1	303,5	10,95	2,77	315,7	40,5	2,62	56664	-
350	75	3,0	29,90	23,47	4840,0	222,4	12,72	2,73	276,6	29,6	0,87	61394	-
350	75	4,0	39,35	30,89	6257,8	278,9	12,61	2,66	357,6	37,2	2,06	72946	-
350	75	4,5	43,95	34,50	6926,1	303,8	12,55	2,63	395,8	40,5	2,91	77206	-
400	75	3,0	32,86	25,80	6730,4	222,5	14,31	2,60	336,6	29,7	0,96	80089	-
400	75	4,0	43,31	34,00	8722,9	279,1	14,19	2,54	436,2	37,2	2,26	95288	-
400	75	4,5	48,41	38,00	9666,5	304,1	14,13	2,51	483,4	40,5	3,21	100930	-
450	75	3,0	35,82	28,12	9031,6	222,5	15,88	2,49	401,4	29,7	1,05	101270	-
450	75	4,0	47,27	37,11	11729,4	279,3	15,75	2,43	521,4	37,2	2,47	120617	-
450	75	4,5	52,87	41,50	13012,0	304,4	15,69	2,40	578,4	40,6	3,51	127836	-

Table A3.2 Section properties of heavy cold formed double extended C sections

D (mm)	Btop (mm)	Bbot (mm)	t (mm)	Area (cm^2)	Weight (kg/m)	I_{xx} (cm^4)	I_{yy} (cm^4)	r_{xx} (cm)	r_{yy} (cm)	W_{xx} (cm^3)	W_{yy} (cm^3)	J (cm^4)	C_w (cm^6)
200	75	100	4,0	28,66	22,50	1745,1	375,9	7,80	3,62	163,5	43,0	1,4980	29133
200	75	100	5,0	35,57	27,92	2138,5	471,4	7,75	3,64	200,3	53,9	2,9169	34727
200	75	100	6,0	42,35	33,24	2513,6	567,3	7,70	3,66	235,3	64,8	5,0143	39661
250	75	100	4,0	32,62	25,61	2951,3	376,1	9,51	3,40	222,8	43,0	1,7050	46297
250	75	100	5,0	40,53	31,82	3627,2	471,8	9,46	3,41	273,8	53,9	3,3236	55358
250	75	100	6,0	48,31	37,92	4275,9	568,0	9,41	3,43	322,7	64,9	5,7200	63436
300	75	100	4,0	36,58	28,71	4564,9	376,3	11,17	3,21	288,9	43,0	1,9120	67503
300	75	100	5,0	45,59	35,71	5622,1	472,2	11,12	3,22	355,8	54,0	3,7304	80885
300	75	100	6,0	54,27	42,60	6641,7	568,7	11,06	3,24	420,2	65,0	6,4257	92900
400	75	100	4,0	44,50	34,93	9213,0	376,8	14,39	2,91	441,3	43,1	2,3260	122117
400	75	100	5,0	55,41	43,50	11379,2	473,0	14,33	2,92	544,9	54,1	4,5439	146726
400	75	100	6,0	66,19	51,96	13482,1	570,1	14,27	2,93	645,5	65,1	7,8371	169022

HEAVY COLD FORMED DOUBLE LIPPED C SECTIONS

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 2.4 m
- Steel strength S350
- Pin resistance 18 kN

Beam is **unpropped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table A3.3 Design table for composite beam with double lipped C, H-CFS.

IMPOSED LOAD kN/m²	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
200x75x3.0	5,0	g	3,9	19,6	16	4,8	g	5,7	17,2	15	4,6	g	7,1	15,2	15	4,3	d	6,7	11,0	14
200x75x4.0	5,4	g	4,3	21,1	17	5,2	g	6,3	18,4	17	5,0	g	7,8	16,2	16	4,7	d	8,0	12,7	15
200x75x4.5	5,5	g	4,5	21,7	18	5,3	g	6,6	18,9	17	5,2	g	8,1	16,6	17	4,9	d	8,6	13,5	16
250x75x3.0	5,9	g	4,8	23,2	19	5,7	g	7,0	20,1	18	5,4	d	7,9	16,0	17	4,9	d	7,2	11,3	16
250x75x4.0	6,4	g	5,3	24,9	21	6,2	g	7,7	21,6	20	6,0	d	9,5	18,8	19	5,5	d	8,8	13,3	18
250x75x4.5	6,6	g	5,5	25,6	21	6,4	g	8,0	22,3	21	6,2	g	9,9	19,5	20	5,7	d	9,4	14,3	18
300x75x3.0	6,8	g	5,7	26,6	22	6,6	g	8,2	23,0	21	6,1	d	8,5	16,8	20	5,6	d	7,9	11,9	18
300x75x4.0	7,4	g	6,3	28,6	24	7,1	g	9,1	24,8	23	6,7	d	10,2	19,7	22	6,2	d	9,5	14,1	20
300x75x4.5	7,6	g	6,5	29,4	25	7,4	g	9,5	25,5	24	7,0	d	11,0	21,1	23	6,4	d	10,3	15,0	21
350x75x3.0	7,7	g	6,5	29,8	25	7,4	g	9,4	25,9	24	6,7	d	9,1	17,5	22	6,2	d	8,5	12,5	20
350x75x4.0	8,4	g	7,2	32,2	27	8,1	g	10,7	28,4	27	7,5	d	11,1	20,8	24	6,9	d	10,3	14,8	22
350x75x4.5	8,6	g	7,5	33,2	28	8,3	g	10,9	28,6	27	7,8	d	11,9	22,2	26	7,2	d	11,2	15,9	23
400x75x3.0	8,6	g	7,3	33,1	28	8,2	d	10,2	27,5	27	7,4	d	9,7	18,3	24	6,8	d	9,0	13,0	22
400x75x4.0	9,3	g	8,2	35,7	31	9,0	g	11,8	30,8	29	8,2	d	11,9	21,8	27	7,6	d	11,0	15,6	25
400x75x4.5	9,6	g	8,5	36,9	32	9,3	g	12,3	31,8	30	8,6	d	12,9	23,5	28	7,9	d	12,0	16,8	26
450x75x3.0	9,5	g	8,2	36,5	31	8,9	d	10,9	28,7	29	8,1	d	10,4	19,2	26	7,4	d	9,6	13,6	24
450x75x4.0	10,3	g	9,1	39,3	34	9,9	g	13,1	33,8	33	9,0	d	12,6	22,9	29	8,3	d	11,8	16,4	27
450x75x4.5	10,6	g	9,5	40,5	35	10,2	g	13,7	34,9	34	9,4	d	13,7	24,7	31	8,6	d	12,8	17,7	28

HEAVY COLD FORMED DOUBLE EXTENDED C SECTIONS

BEAM DATA
- Internal beam
- Uniform load
- Beam spacing 2.4 m
- Steel strength S350
- Pin resistance 18 kN

Beam is **unpropped** during construction

SLAB DATA
- Slab depth 130 mm
- Concrete strength C30/37

Table A3.4 Design table for composite beam with double extended C, H-CFS.

IMPOSED LOAD kN/m²	1.5					2.5					3.5					4.5				
DESIGNATION	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N	L_E (m)		δ_E (mm)	δ_S (mm)	N
200x75x100x4.0	5,5	g	4,5	21,7	18	5,3	g	6,5	18,8	17	5,1	g	8,1	16,5	17	4,8	d	7,7	12,1	15
200x75x100x5.0	5,9	g	4,9	22,9	19	5,7	g	7,1	20,0	18	5,5	g	8,8	17,5	18	5,2	d	9,0	13,8	17
200x75x100x6.0	6,2	g	5,2	24,1	20	6,0	g	7,5	20,9	19	5,8	g	9,3	18,3	19	5,5	d	10,1	15,4	18
250x75x100x4.0	6,5	g	5,5	25,5	21	6,3	g	8,0	22,1	21	6,0	d	9,1	17,9	19	5,5	d	8,5	12,7	18
250x75x100x5.0	7,0	g	6,0	27,0	23	6,7	g	8,6	23,4	22	6,5	g	10,6	20,4	21	6,0	d	9,9	14,7	19
250x75x100x6.0	7,4	g	6,4	28,3	24	7,1	g	9,2	24,6	23	6,9	g	11,3	21,4	22	6,4	d	11,3	16,4	21
300x75x100x4.0	7,5	g	6,5	29,1	25	7,3	g	9,4	25,1	24	6,8	d	10,0	18,9	22	6,2	d	9,3	13,5	20
300x75x100x5.0	8,1	g	7,0	31,0	26	7,8	g	10,1	26,6	25	7,4	d	11,7	21,8	24	6,8	d	10,9	15,6	22
300x75x100x6.0	8,5	g	7,5	32,6	28	8,2	g	10,8	28,0	27	7,9	g	13,3	24,4	26	7,3	d	12,4	17,6	24
400x75x100x4.0	9,5	g	8,4	36,4	31	9,1	g	12,1	31,3	30	8,3	d	11,6	21,2	27	7,6	d	10,8	15,1	25
400x75x100x5.0	10,2	g	9,1	38,7	33	9,8	g	13,1	33,2	32	9,1	d	13,7	24,5	30	8,3	d	12,8	17,6	27
400x75x100x6.0	10,7	g	9,8	40,8	35	10,3	g	14,0	34,9	34	9,8	d	15,7	27,7	32	9,0	d	14,7	19,9	29