CHAPTER 4

4.0 COMPOSITE BEAMS AND GIRDERS

4.1 INTRODUCTION

One of the major developments in steel design during the last two decades has been a significant trend to compositely designed rolled or welded beams with a deck-slab system. Structural steel design standards governing design of composite members have evolved using primarily an ultimate strength approach, leading up to the limit states concept used in S16.1, the Canadian Standard.

The study of composite structural interaction between steel beams and concrete dates back to as early as 1922 by Mackay et al(4.1) in Canada, later followed by researchers in U.S.A., England, Switzerland, France, Belgium and Germany(4.2). During the mid 30's, a series of tests on composite beams was conducted at the University of Toronto under the auspices of the Canadian Institute of Steel Construction(4.3). These tests gave an indication of the combined strength of concrete slabs bearing on top of steel beams, with and without shear connectors, as well as some with partial encasement of the top flange and a portion of the web. As a result, design tables were printed in the 1937 edition of the CISC Steel Handbook.

In 1941, clauses governing composite beam design for buildings were incorporated in the National Building Code of Canada, as a direct result of the earlier research and testing. It was not until 1944, that the first North American specification for composite bridge girder design was issued(4.4,4.5). In the U.S.A., development of composite building design followed somewhat later. Composite beam design provisions were first introduced in the 1952 edition of the AISC Specification.

Structural designers have long been aware of the advantages of composite beam construction such as the saving of steel, reduction of overall structural depth, and the increase in floor stiffness and load capacity. Prior to the early 1960's, steel beams of rolled and welded 'H' shapes were designed to act compositely with poured-in-place flat-bottom slabs of various thicknesses with connection provided by means of either member embedment or mechanical shear connectors, herein after referred to as "Solid Composite Construction". See Figure 4.1. However, the continuing search for improved material and manpower utilization as well as construction economics has resulted in the evolution of a method referred to herein as "Hollow Composite Construction". See Figure 4.2. Beginning around the mid-sixties, structural designers began to use composite beams incorporating composite deck-slab systems(4.6,4.7,4.8,4.9). The use of steel decks in building construction supplanted the traditional temporary timber planking previously required for building access and for the safety of workers both above and below, offering as well immediate access to other building trades. The economy of this method of floor construction was enhanced during the same period, with the introduction of headed stud shear connectors welded through the fluted or cellular deck units into the top flange of the steel beams. Interim steps on some early projects called for pre-punching of the steel deck, either for direct field welding of studs to the steel beams, through the openings, or for shop installation of the studs to a grid, matching the pre-punched deck holes. Neither practice proved to be very satisfactory and the weld-through practice evolved.

The following sections of this chapter are dedicated to a more detailed review of design methodology for composite beam members as permitted by S16.1, including the latest amendments to the standard.

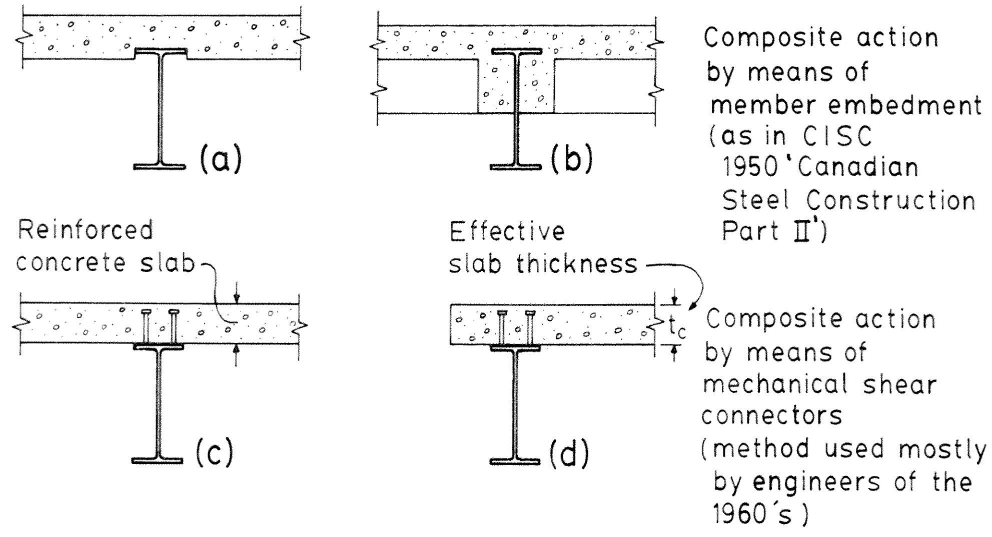


Figure 4.1
Solid Composite Construction

Composite Beam and Girder Floor Systems

In a composite beam or girder, the steel shape is used primarily to resist tension and shear while the concrete slab acts primarily as a compression resisting element. Thus, the composite beam or girder is usually designed as a simply supported flexural member. Composite action can be achieved in the beams alone, in the girders alone, or in both. The steel members are usually W-shapes, although welded W-shapes and HSS shapes may also be used. Trusses are specifically dealt with in Chapter 5 and stub-girders are discussed in considerable detail in Chapter 6.

4.2 EFFECTIVE THICKNESS OF CONCRETE SLAB

The concrete slab may be cast on temporary forms, sheet steel forms or steel deck. If the concrete slab is cast with a flat underside, or on a corrugated or fluted sheet steel form, the strength and stiffness of the composite member can be calculated based on the overall slab thickness in accordance with Clause 17.2 of Standard S16.1 (also see definitions in Sections 1.2 and 1.4) and a floor so constructed is known as ***Solid composite*** construction. However, if concrete is cast on steel deck (See Figure 1.12d) only the concrete cover thickness above the top of the steel deck is effective for composite action with the steel shape. This type of configuration is referred to here as ***Hollow composite*** construction.

Hollow composite floors have become a very commonly used gravity load resisting system for four main reasons.

- The use of steel deck eliminates the need for formwork shoring and provides a wide effective width of concrete slab for composite interaction with the steel shape.
- Composite steel decks also serve as positive concrete reinforcement.
- Welding of headed studs directly through the steel deck provides economical interconnection of beam and deck-slab.
- Steel decks used in a cellular configuration allow the passage of in-floor electrical and communication services.

Although most of the design tables and the worked example in this chapter apply specifically to hollow composite floor construction, the design considerations and methodology outlined also apply for the most part to solid composite floor construction.

4.3 EFFECTIVE WIDTH OF CONCRETE TOP FLANGE

For a T-beam formed by a steel section and a concrete cover slab, only part of the concrete top flange is effective. The effective width, under elastic conditions, is a function of beam span, Poisson's ratio, and the shape of the moment diagram[4.10,4.11]. Based on elastic theories, semi-empirical design rules for determining effective slab width have been adopted by S16.1. These rules may be applied to hollow composite members as well as solid composite members because strain measurements across the slab width have indicated that shear lag is no more severe in a steel deck-slab than in a formed solid slab[4.12]. The applicability of the existing design rules for computing effective slab widths of composite beams designed by ultimate strength method is explained by the Commentary to S16.1 as published in the CISC Handbook of Steel Construction, 3rd Edition. The design effective width of concrete specified by Clause 17.3.2 of S16.1 is described in Section 1.4 and illustrated in Figure 1.13.

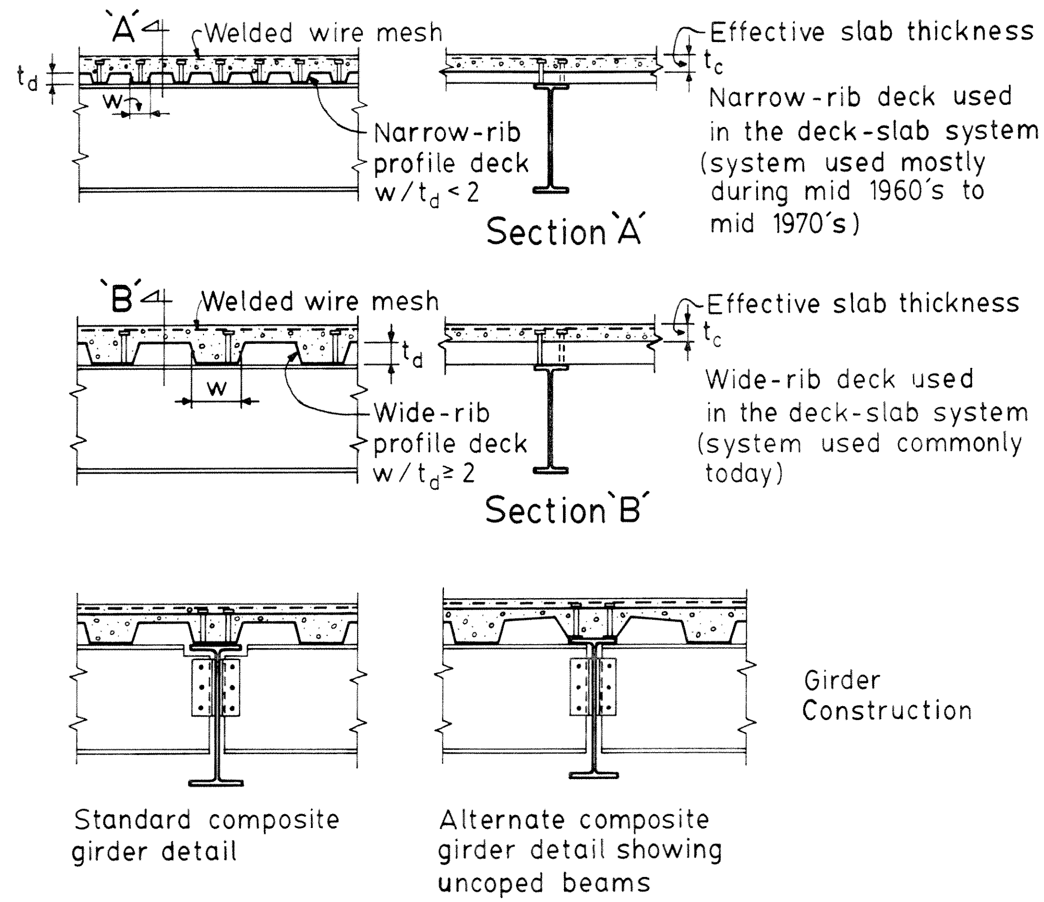


Figure 4.2
Hollow Composite Construction

The design of a composite floor member involves the assessment of its performance at various limit states including evaluation of:

- the ultimate strength of the composite section,
- the ultimate capacity of the bare steel section during construction stages, and
- the performance of the composite member both as an individual member and as part of the overall floor system, when subjected both to specified loads and during construction.

4.4 FLEXURAL STRENGTH OF A COMPOSITE SECTION

The ultimate flexural capacity of a composite member depends on the degree of shear connection provided, the compressive resistance of the effective concrete slab (plus any steel in compression, in those instances where the neutral axis falls within the steel section), and the tensile yield resistance of the steel shape. Shearing of the concrete adjacent to the steel/concrete connection will be discussed in Section 4.9. Three flexural modes of failure therefore exist:

- shear connection failure,
- crushing of concrete, and
- full yielding of steel section.

Composite members with full shear connection can exhibit full composite action. Composite members with partial shear connection lack the strength to provide full composite action and will likely fail at the shear connections. When failure occurs either by crushing of concrete and/or full yielding of steel, large deflections prior to failure are the norm.

Limit states design of a composite beam requires a designer to satisfy strength and stability criteria such that the factored member resistance under the ultimate limit state condition is greater than, or equal to, the effect of factored external loads. In general, factored load means the product of a specified load and its load factor and factored resistance means the product of member resistance and the applicable performance factor[4.13,4.14]. By neglecting concrete tensile strength, the factored ultimate limit state moment resistance, M_{rc}, can be calculated by the following procedure.

Locate Plastic Neutral Axis

The plastic neutral axis of a composite section can be located by comparing the factored compressive resistance of the effective concrete slab, $0.85\ \phi_c\ b_1\ t_c\ f'_c$, and the factored tensile resistance of the steel shape, $\phi\ A_s\ F_y$, where

ϕ_c = performance factor for concrete, 0.60
ϕ = performance factor for steel, 0.90
b_1 = effective width of concrete slab, mm
t_c = effective thickness of concrete slab, mm
f'_c = specified compressive strength of concrete at 28 days, MPa
A_s = cross-sectional area of steel shape, mm^2
F_y = specified minimum yield strength of steel, MPa

Case 1 – Neutral Axis in Concrete (full shear connection)

If $\phi\ A_s\ F_y$ is less than $0.85\ \phi_c\ b_1\ t_c\ f'_c$, the plastic neutral axis (P.N.A.) falls within the concrete as shown in Figure 4.3. The ultimate flexural capacity is reached when the steel shape is fully yielded.

The depth of the rectangular concrete stress block, 'a', can be found by equating the factored ultimate compressive component to the tensile component of the moment couple of the composite section,

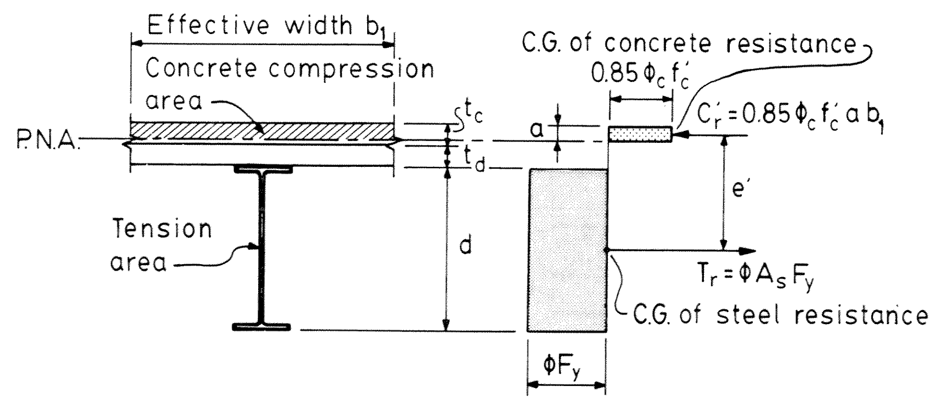


Figure 4.3
Neutral Axis Falls within Effective Slab Thickness ($a \leq t_c$) (Case 1)

$$0.85\ \phi_c\ f'_c\ b_1\ a = \phi\ A_s\ F_y$$

and solving for 'a',

$$a = \frac{\phi\ A_s\ F_y}{0.85\ \phi_c\ f'_c\ b_1} \qquad 4.1$$

Note: Value of 'a' should always be less than or equal to t_c.

M_{rc} can then be computed by finding the internal moment arm, e', as

$$M_{rc} = e'\ \phi\ A_s\ F_y \qquad 4.2$$

in which

$$e' = \frac{d}{2} + t_o - \frac{a}{2} \qquad 4.3$$

where d = depth of steel section in millimetres
t_o = overall depth of deck-slab in millimetres

In the case of solid composite construction, $t_c = t_o$

In order to achieve composite action concrete and steel must act as a single unit. If headed studs are used as the shear transfer device, the principal force that must be resisted by these stud connectors is the sum of the factored horizontal shears between the points of maximum and zero moment, V_h, as illustrated in Figure 4.4.

If full shear connection is required the designer must ensure that

$$Q_r \geq \phi\ A_s\ F_y \qquad 4.4$$

Where Q_r = sum of the factored resistances of all shear connectors between the point of maximum moment and the adjacent point of zero moment (also see Section 2.5, stud shear connectors)

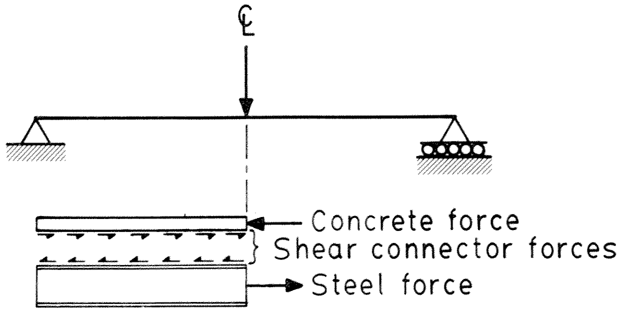


Figure 4.4
Concrete-Steel Interface Shear Forces

Case 2 – Neutral Axis in Steel (full shear connection)

Case 2 refers to the situation where the plastic neutral axis lies in the steel section when full shear connection is provided. This occurs when the resistance of the composite section is governed by concrete compressive capacity, i.e.,

$$0.85\ \phi_c\ f_c'\ b_1\ t_c < \phi\ A_s\ F_y$$

The factored concrete compressive resistance is computed as,

$$C_r' = 0.85\ \phi_c\ f_c'\ b_1\ t_c \qquad 4.5$$

As shown in Figure 4.5, part of the steel section is now in compression.

The factored compressive resistance of the steel area in compression can be written as,

$$C_r = \frac{\phi\ A_s\ F_y - C_r'}{2} \qquad 4.6$$

By taking moments about the centroid of the steel area in tension, the factored moment resistance can be found:

$$M_{rc} = C_r'\ e' + C_r\ e \qquad 4.7$$

where e′ and e are the lever arms as shown in Figure 4.5. The lever arms can be computed once the exact location of the P.N.A. is found.

a) P.N.A. in steel flange i.e. when $C_r \leq \phi\ b\ t\ F_y$

$$e = \frac{(A_s d - b t_1^2)}{2(A_s - b t_1)} - \frac{t_1}{2} \qquad 4.7a$$

$$e' = e + \frac{t_1}{2} + t_o - \frac{t_c}{2} \qquad 4.7b$$

where $t_1 = \dfrac{C_r}{\phi\ b\ F_y}$

b) P.N.A. in steel web, i.e. when $C_r > \phi\ b\ t\ F_y$

$$e = d - d_2 - d_3 \qquad 4.7c$$

$$e' = d + t_o - d_2 - \frac{t_c}{2} \qquad 4.7d$$

where $d_2 = \dfrac{A_s d/2 - A_{cr}(d - d_3)}{A_s - A_{cr}}$

$$d_3 = \frac{bt^2/2 + d_1 w(t + d_1/2)}{A_{cr}}$$

$$d_1 = (A_{cr} - bt)/w$$

$$A_{cr} = \frac{C_r}{\phi\ F_y}$$

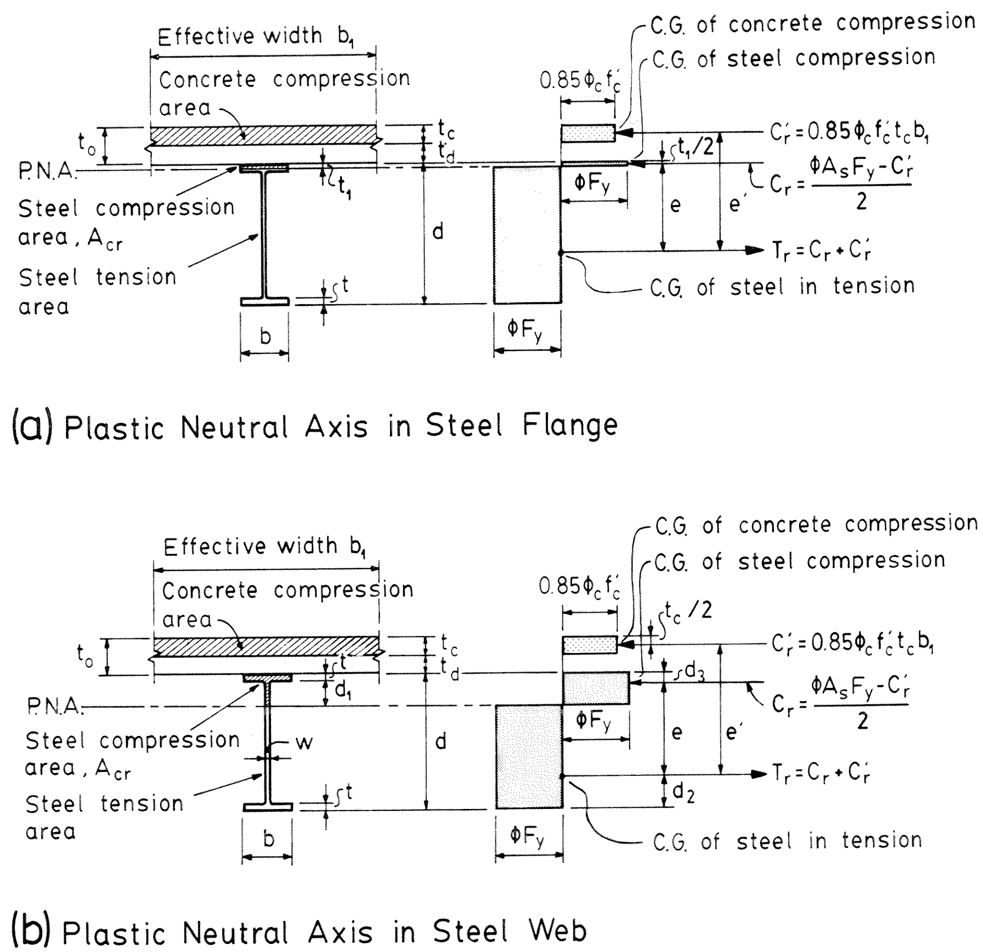


Figure 4.5
Force Equilibrium of Composite Section with Full Shear Connection (Case 2)

For Case 2, V_h is equal to C_r'. Since full shear connection is required, the following requirement must be satisfied.

$$Q_r \geq 0.85\ \phi_c\ f_c'\ b_1\ t_c \qquad 4.8$$

Case 3 – Partial Shear Connection

In many design situations, full shear connection is not necessary. In fact, most composite floor beams are constructed with partial shear connection in the range of 50% to 70% of that theoretically required for full composite action (i.e. Q_r required for Case 1 or Case 2).

There are three main reasons for this:

- 50% or higher shear connection usually provides an ultimate flexural capacity greater than 80% of the full composite flexural capacity(4.15).
- Construction considerations such as deflection under fresh concrete and beam depths available, particularly in the case of unshored construction, may govern the selection of the steel beam section.

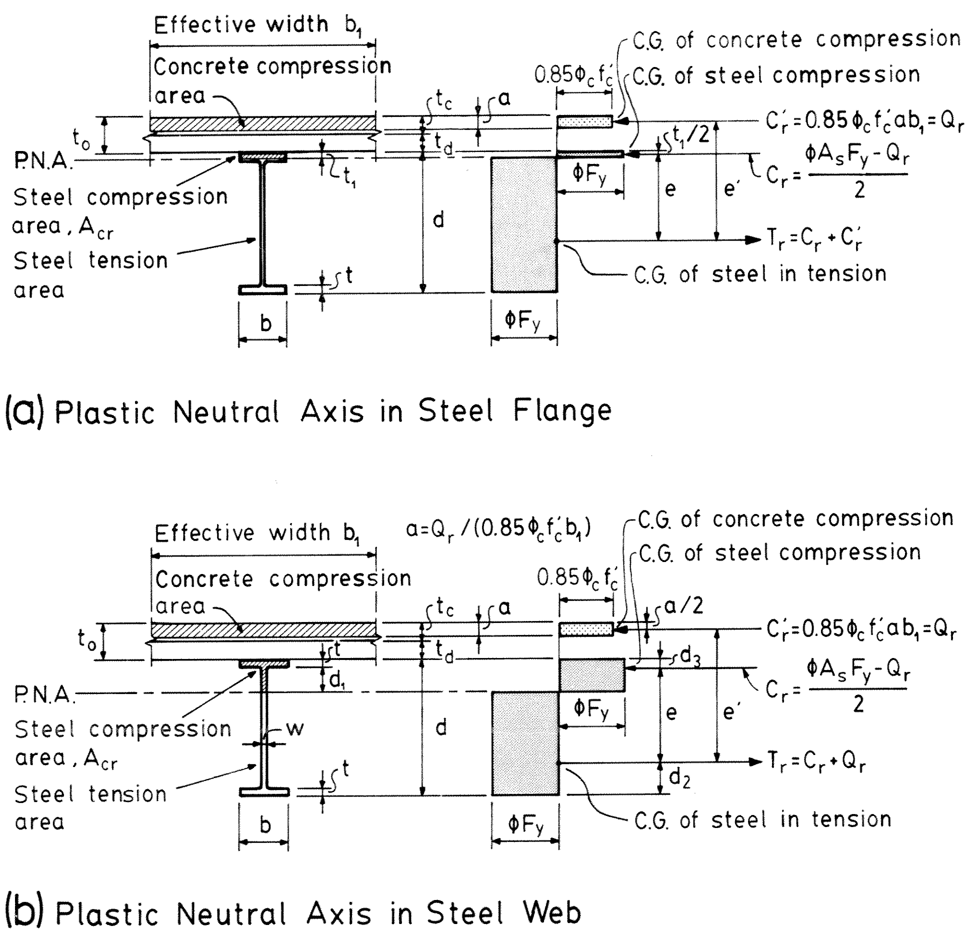


Figure 4.6
Force Equilibrium of Composite Section with Partial Shear Connection (Case 3)

- Physical shape and layout of deck flutes on a composite beam often prevent the economical distribution of sufficient stud connectors for the attainment of full shear connection without installation of studs in pairs and thus reducing their efficiency.

Partial shear connection is calculated as the ratio of Q_r to the lesser of $\phi\ A_s\ F_y$ and $0.85\ \phi_c\ f_c'\ b_1\ t_c$ expressed as a percentage. For flexural resistance calculations, a partial shear connection lower limit of 50% is specified in Clause 17.4.4 of S16.1. Composite beams with less than 50% shear connection may not behave as a composite member through the entire loading range up to the ultimate state as predicted by the method described herein(4.15). If composite action is required for deflection considerations only, the lower limit is reduced to 25%, since composite behaviour up to specified load level only is of concern. Considering the model as shown in Figure 4.6, the plastic neutral axis of a Case 3 member always falls within the steel shape. C_r' is restricted to the total horizontal shear provided by the shear connectors, thus

$$C_r' = Q_r = V_h \qquad 4.9$$

C_r' is assumed to act at the centroid of the assumed concrete stress block whose depth, 'a', can be determined by writing

$$0.85\ \phi_c\ f_c'\ b_1\ a = Q_r$$

and solving for 'a',

$$a = \frac{Q_r}{0.85\ \phi_c\ f_c'\ b_1} \qquad 4.10$$

C_r can then be computed,

$$C_r = \frac{\phi\ A_s\ F_y - Q_r}{2} \qquad 4.11$$

followed by locating the plastic neutral axis (in steel).

M_{rc} for partial shear connection is calculated using

$$M_{rc} = C_r e + Q_r e' \qquad 4.12$$

Equations 4.7a to 4.7d are applicable, ***provided*** t_c is replaced by 'a'.

Detailed discussions on stud shear connector strengths and spacings for use with solid or hollow composite floor members have been covered in Chapter 2.

4.5 SHEAR STRENGTH

It is assumed that the web of the steel shape carries all the vertical shear force of the composite section. The factored shear resistance can be computed as:

$$V_r = \phi\ A_w\ F_s \qquad 4.13$$
where $A_w = d\ w$

All Canadian rolled W-shapes with F_y = 300 MPa (except W410×39) have web slenderness conforming to Cl. 13.4.la of S16.1, hence $F_s = 0.66\ F_y$. For all other shapes and for sections rolled in higher strength steel, refer to Clause 13.4.1 of S16.1 for the computation of factored shear resistance.

4.6 LATERAL SUPPORT OF UNSHORED MEMBERS

Since composite action cannot be realized before concrete hardens, only the bare steel flexural capacity is available for load support during construction. The load carrying capacity of a steel shape therefore is assessed based on the sequence of construction as described in Chapter 3.

In the case of solid composite floor construction, unshored construction is possible if the forms carrying the fresh-concrete are supported by the steel beams and girders. The flexural strength of a simply supported steel member depends on the lateral support provided by the formwork and/or the framing members. Concrete forms may be designed to provide this lateral support to the steel beam. In accordance with Clause 19.3.1 of S16.1, each of the bracing members that are spaced at intervals should be designed to resist 1% of the compressive force in the top flange at the point of support. In the event that steel deck or form is used to provide a continuous lateral support, it should be designed and connected to the top flange in such a way that it can resist a uniformly distributed lateral force for the length of the compression flange equal to 5% of the maximum axial force in the flange. See Clause 19.3.2 of S16.1.

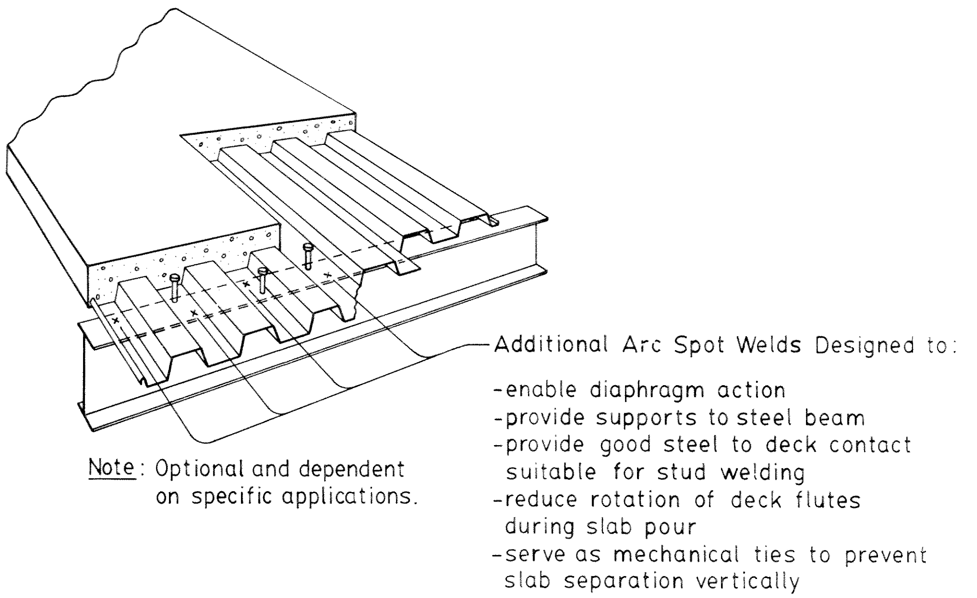


Figure 4.7
Functions of Arc Spot Welds (Puddle Welds) in Hollow Composite Floors

4.7 DESIGN OF UNSHORED HOLLOW COMPOSITE MEMBERS

Temporary shoring and formwork can be eliminated in the construction of hollow composite floor systems if the steel deck and all the bare steel members are adequately designed to carry the fresh-concrete condition loads (inclusive of construction live loads) throughout various construction stages.

In the case of a hollow composite beam, the steel deck is usually connected to the steel beam by means of arc spot welds[4.16] (or puddle welds). These welds, 1) enable the deck and beams to act as a diaphragm as described in Chapter 2, 2) permit the steel deck to act as continuous lateral and torsional support to the steel beams, 3) assist in preventing vertical separation between the deck-slab system and the steel shape, and 4) serve other construction purposes. See Figure 4.7. Stud shear connectors will perform a duplicate function and are frequently used in lieu of some of the deck-to-beam arc spot welds.

Before the steel deck is welded to the steel beam's top flange, the unbraced length, L′ is equal to the beam span. The steel beam is required to carry the mass of the steel deck itself plus a nominal amount of construction load while the steel deck is being placed. After the steel deck is connected to the top flange, the steel beam may be considered continuously braced and hence it can be designed as a laterally supported beam to carry the fresh-concrete condition loads (inclusive of construction loads during concrete placement); see Fig. 4.8, and 4.9b.

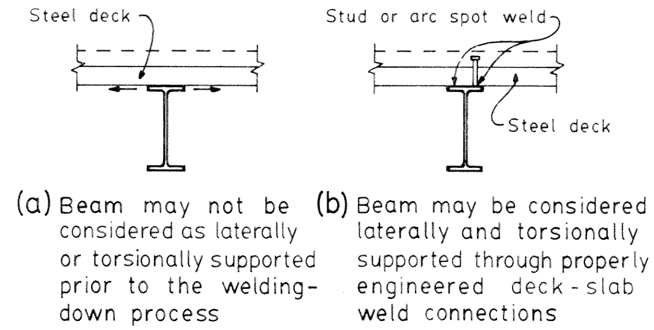


Figure 4.8
Lateral Support Conditions of Hollow Composite Beams under Construction

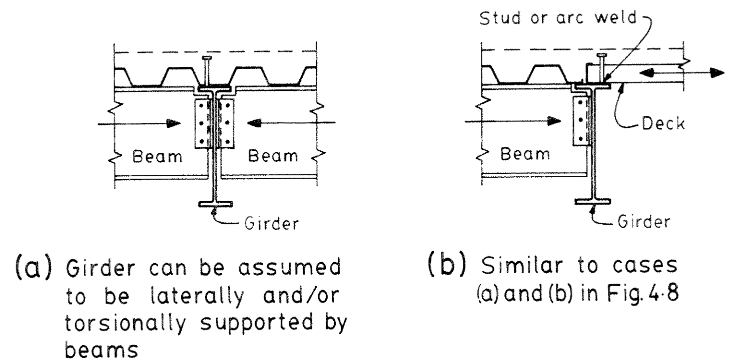


Figure 4.9
Lateral Support Conditions of Hollow Composite Girders under Construction

For a hollow composite girder, the steel deck is usually installed with flutes parallel to the girder span and hence continuous lateral and torsional supports cannot be assumed. In this situation, lateral support of a girder is provided at intervals where beams are framed into it (Figure 4.9a). The unbraced length, L′ to be used for design is the spacing of two adjacent beams near the mid span as shown in the example of Figure 4.10. Undoubtedly, there will be applications where the stiffness of deck and the deck-to-girder connection could be considered to provide additional lateral support to the girder top flange, if required.

Laterally Unsupported Members

When continuous lateral support is not provided to a steel W-shape which is subjected to in-plane bending, lateral-torsional instability may govern the flexural capacity of the member. The factored moment resistance of a laterally unsupported W-shape, M_r', can be determined in accordance with Clause 13.6 of S16.1 as follows (also see Table 4.1).

$$M_r' = \phi\, M_u \quad \text{if} \quad M_u \leq \frac{2}{3} M_p \qquad 4.14$$

$$M_r' = 1.15\, \phi\, M_p \left(1 - \frac{0.28 M_p}{M_u}\right) \text{ but not greater than } \phi\, M_p \qquad 4.15$$

$$\text{if } M_u > \frac{2}{3} M_p$$

$$\text{where } M_u = \frac{\pi}{\omega L'} \sqrt{E\, I_y\, G\, J + \left(\frac{\pi E}{L'}\right)^2 I_y\, C_w} \qquad 4.16$$

$M_p = Z_x F_y$ for Class 1 and Class 2 sections
$\omega = 0.6 + 0.4\, M_1/M_2$ (see Figure 4.10)

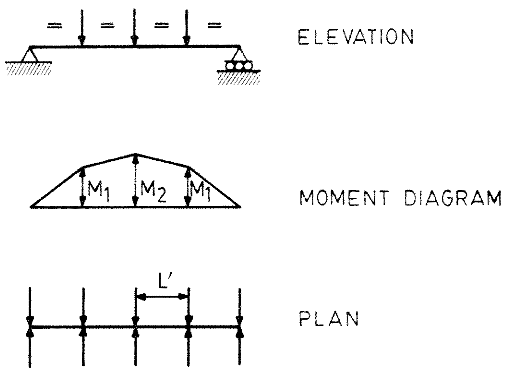


Figure 4.10
Unbraced Length of a Composite Girder
Prior to Composite Action

The equivalent bending coefficient, ω, is used to modify the unbraced length, L', depending on the shape of the moment diagram within the unbraced length. ω assumes a value of 1.0 where the moment diagram within the unsupported length is uniform or nonlinear as shown in Figure 4.11.

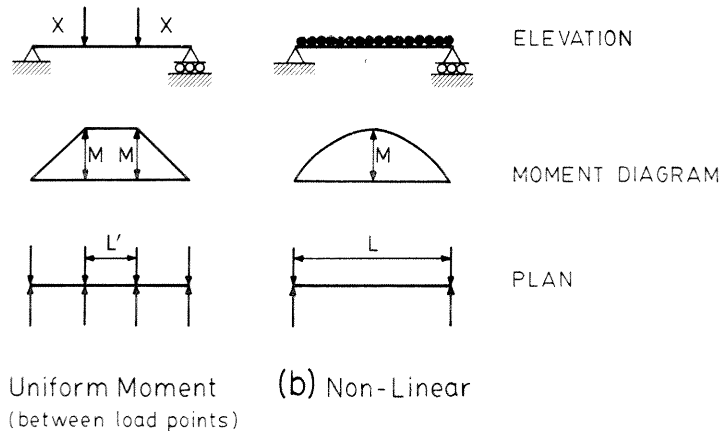


Figure 4.11
Shapes of Moment Diagrams
when Equivalent Bending Coefficient = 1.0

In order to calculate M_u, material and sectional properties including elastic and shear moduli of steel (E,G), moment of inertia about the y-axis, I_y, and the torsional constants of the W-shape, J and C_w, are required. However, M_r' values for various unbraced lengths are listed in Tables 4.1 to 4.8. The L_u values listed in the tables represent the maximum effective unbraced lengths at which lateral-torsional instability is not a problem. Since most L_u values are lower than beam spans of practical design range, one cannot afford to ignore stability investigation of the bare steel member before the steel deck is welded to its top flange in unbraced and unshored composite construction.

4.8 SERVICEABILITY REQUIREMENTS

While an acceptable level of safety against collapse is ensured by procedures discussed elsewhere, the performance of a structural system in various service conditions should also be considered. Serviceability limit states design of a composite beam and girder frame include 1) calculation of vertical deflection under short term and long term specified loads, 2) evaluation of construction deflections, and consideration of possible camber requirements in the case of unshored construction, 3) computation of beam deflection caused by slab shrinkage, and 4) assessment of vibration characteristics of the floor system (see Chapter 7). In addition, some measures should be taken to control concrete cracking as described in Section 1.4d. If unshored construction is selected, yielding of the bottom flange must also be prevented (see S16.1 Clause 17.6).

Although it is not a requirement of S16.1, maximum span-to-depth ratios of 24 and 30, for composite section depth and steel member depth respectively, were recommended in a CSSBI publication[4.9]. It is believed that these criteria will provide a good starting point in the selection of satisfactory composite members. Consideration of the above serviceability criteria, consideration of the standard depths of sections available, and any special design criteria such as high code-specified design loads in areas of anticipated low actual loads (e.g. shopping mall rest areas) will prompt further consideration of these criteria and further vibration analysis as noted in Chapter 7.

Vertical Deflection

Floor structural members deflect or sag under load. This vertical deflection, if excessive, can cause cracking of ceilings and partitions or prevent proper fit of doors. Visible distortions may cause undesirable psychological feelings to some people. If dead load deflection (especially member deflection due to fresh-concrete condition loads) is found to be excessive, it can usually be controlled by shop cambering the steel members or by shoring.

In order to prevent excessive deflection due to specified live loads (and a portion of superimposed dead load, e.g. partitions), the composite beam should be proportioned with adequate flexural stiffness. For simple span members of floors and roofs supporting construction and finishes not susceptible to cracking, a deflection limit of 1/300 of span is suggested as acceptable in Appendix I of S16.1. The appropriate deflection limit depends primarily on the type of ceiling and partitions supported by the floor. A limit of 1/360 of the span under full specified live loading is generally considered acceptable for simple span floor members supporting finishes susceptible to cracking. This maximum limit is also suggested in Appendix I of S16.1.

Since a floor member is usually subjected to occupancy loads after the concrete has attained its specified strength, member deflection should be calculated based on the elastic sectional properties of the composite section. The elastic stiffness of a composite section can be computed by means of the transformed section method. It is customary to calculate the moment of inertia of a composite section, I_t, by regarding the effective concrete cross-section (in compression) as equivalent steel cross-section, using the elastic modular ratio E/E_c. The estimate of live load deflection is complicated by the fact that concrete creeps under sustained loads. These long-term loads include dead loads (that are carried by the composite section) and a portion of the service live load. If the member is shored during construction the composite section must sustain all the fresh-concrete condition loads, superimposed dead loads, and a portion of the live load for a long period in service. In the case of unshored construction, the fresh-concrete condition loads are carried by the steel shape and hence only the sustained superimposed dead and live loads are responsible for concrete creep deflection. The amount of live load that contributes to creep depends on the type of floor occupancy. The sustained live load of a typical floor in an office building is a small fraction of its total occupancy load (about 25%)[4.37], but in a warehouse, a large portion of the service load may stay in place throughout its useful life.

For a compositely designed beam or girder, creep deflections occur when the effectiveness of the concrete slab under long term loading decreases, leading to an increase in steel stresses. For this reason, stresses and deflections caused by dead load on the composite section are usually determined with composite section properties calculated using a factored steel to concrete modular ratio equal to approximately 2.5 times the elastic modular ratio[4.17,4.18].

Steel-concrete interface slip and partial shear connection may reduce the stiffness of a composite member[4.12,4.15]. In the case of a hollow composite beam (ribs of slab running perpendicular to the beam) the ribbed profile of the deck-slab may also increase flexibility of the composite member. In lieu of tests or analysis, S16.1 provides a rule for the computation of effective moment of inertia, I_e, such that the reduction of composite member stiffness due to these effects can be accounted for during the member deflection computation.

$$I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s) \qquad 4.17$$

where I_s = moment of inertia of steel beam
I_t = transformed moment of inertia of composite beam
p = fraction of full shear connection
(use $p = 1$ for full shear connection)

If the effect due to concrete creep is NOT accounted for in a deflection computation or via a test, the estimated deflection under specified loads (using effective composite moment of inertia I_e, calculated on the basis of the unfactored elastic modular ratio E/E_c) must be increased by a minimum of 15%, in accordance with Clause 17.3.1.1(b) of S16.1.

Composite Beam Deflection due to Shrinkage

Tests[4.19] of small plain concrete specimens, exposed to air at 50 percent relative humidity, produced unit length changes due to drying shrinkage strains in the range of 600 to 800 micro strain. Since the plain concrete specimens were free to shrink without any restraint, the measured shrinkage values are referred to as free shrinkage of concrete. It was found that an average of one third of the estimated free shrinkage occurred within the first month and that 90% of the estimated free shrinkage had taken place at the end of an 11 month period. (Shrinkage measurements in these tests were terminated at the end of 38 months.)

The rate and total amount of drying shrinkage in a concrete specimen depend on factors such as:

- curing practices,
- surface/volume ratio,
- average relative humidity of surrounding air,
- average surrounding air temperature,
- water-cement ratio of the concrete mix (slump),
- size and type of aggregate,
- time from initial set of concrete, and
- amount of restraint from steel reinforcement.

See references [4.20,4.21,4.22].

Volumetric reduction of concrete due to drying shrinkage may also contribute to flexural deflection of composite members. Since the concrete slab may shorten but the steel shape does not, therefore, shrinkage due to drying of the concrete slab is partially restrained by the existence of steel reinforcement and by attachment to the steel shape. The slab shrinkage measured in such a test specimen is referred to as restrained shrinkage.

A drying shrinkage deflection pilot test was carried out by Robinson[4.23]. The test specimens consisted of two simply supported W410×54 hollow composite beams, 9 metre span, using 76 mm deep wide-rib profile composite decks (from two deck manufacturers) with ribs running perpendicular to the beam span. A 65 mm thick cover slab of normal density concrete with 152×152 MW9.1×MW9.1 welded wire mesh, forming a flange width equal to 2290 mm, was used on each test beam. The slump of the concrete mix was measured at 75 mm. Instrument readings of beam deflections due to slab shrinkage, together with room temperature and relative humidity of the laboratory atmosphere, were taken for a period of 50 days. Shrinkage readings along the length of concrete top flange, as well as those of a small unreinforced calibration specimen, were taken for the two test beams. Unlike the free shrinkage measurements previously discussed, a measurable increase in beam deflections due to shrinkage of the concrete slab was noted during the first 40 days of recording. After this, no discernible increase of beam deflection was observed for the remainder of the test period (see Figure 4.12). For the beams tested, the measured slab shrinkage amounts to about 2/3 of the free shrinkage measurement of the calibration specimen. Using these results to calculate slab shrinkage of the test specimen gave a computed slab shrinkage strain of about 350 micro strain.

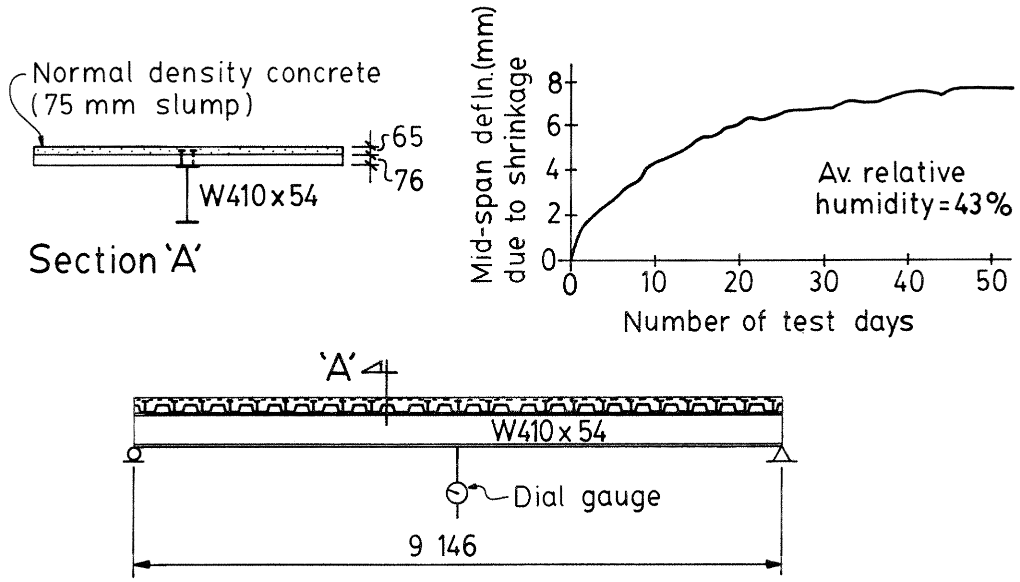


Figure 4.12
Shrinkage Deflection Tests
at McMaster University

In the case of a hollow composite girder, where the deck flutes run parallel to the span of the girder, a reduction in slab shrinkage (due to additional restraint of deck steel) should be assumed during the estimation of deflections due to drying shrinkage of slabs. The amount of restrained shrinkage in slabs of hollow composite spandrel members should generally be less than that found from interior members, due to the lowering of effective slab to steel area ratio. Thus, in computing deflections of spandrel members due to restrained slab shrinkage, values lower than 350 micro strain may be considered appropriate.

Drying shrinkage of slabs at beam supports of interior bay composite beams can cause a tension stiffening effect[4.21]. The use of rebar details as in Figure 1.16, and member-end connection details further reduce the beam deflection. (Also see worked example, 4.14.) It is fair to conclude that the proposed restrained shrinkage strain of 200 micro strain for use with composite member design as given by Viest[4.4] appears to be appropriate.

Control of Yielding Deformation

If no temporary shores are provided, the bare steel shape must support the fresh-concrete condition loads (see definition Table 3.3), and hence its bottom flange experiences higher tensile strains than that of a shored member. In order to prevent permanent deformation due to steel yielding under service load conditions, the total bottom steel tensile strain due to full specified loads must be within the elastic range[4.24]. In accordance with Clause 17.6 of S16.1, stresses in the tension flange due to loads applied before the concrete strength attains 0.75 f'_c plus superimposed stresses due to the remaining specified loads considered to act on the composite section shall not exceed $0.9F_y$, if the member is unshored during construction. Assuming elastic behaviour, this requirement can be written as

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} \leq 0.9\ F_y \qquad 4.18$$

Where

M_b = moment due to specified fresh-concrete condition load acting on bare steel section,
M_t = moment due to specified superimposed loads acting on composite section,
S_x = elastic section modulus of bare steel section,
S_t = elastic section modulus with respect to bottom flange of composite section.

Camber Requirements

Deflection of an unshored steel member during construction depends on the member's stiffness and support conditions. Deflection during concrete placement can result in higher concrete consumption (if concrete is screeded level) and thus can lead to additional loading similar to 'ponding' on a roof member. Secondary members are supported by primary members which also deflect, and deflections of primary and secondary flexural members are interactive and additive. Thus the final steel framed floor, including steel deck, beams and girders, may be subjected to more than the calculated load.

Cambering of beams and/or girders for fresh-concrete condition loads is a common method of avoiding this problem when the calculated member deflection during concreting exceeds 20 mm. Cold bending in a gag press, hydraulic jacking at third points in a simple jig, or heat cambering are common and acceptable methods of shop cambering. In addition to cambering for fresh-concrete condition loads, the camber allowance for longer span members, say in excess of 10 to 12 metres, may include the calculated deflection of the **composite** member due to superimposed dead loads and some shrinkage. However, this approach can create a condition opposite to the 'ponding' condition and care should be taken to ensure uniform concrete cover slab thickness in critical areas. One frequently finds that the "calculated" deflections due to concrete placement do not occur. This phenomenon can be partially explained by the fact that the steel beam may have been specified with a slight over camber. Also, end restraint is provided by almost any connection and some "supporting members" are stiffer than others. One should also consider that hot rolled steel products are not precisely straight. Permissible tolerances for out-of-straightness are published in CAN3-G40.20 "General Requirements for Rolled or Welded Structural Quality Steel"; and using a 9 m W shape as an example, the maximum deviation from "straight" may be 9000/1000 = 9 mm. Clause 26.9.5 of S16.1 requires that beams with bow within straightness tolerance shall be fabricated so that after erection the bow due to rolling or fabrication shall be upward.

4.9 INTERACTION OF COMPOSITE BEAMS OR GIRDERS WITH DECK-SLAB SYSTEMS

Several phenomena related to the interaction between compositely designed steel beams and their load-sharing partner, the deck-slab system, require discussion. Performance and load capacity of stud shear connectors have already been discussed in Chapter 2. Local response of the deck-slab system to the forces produced by the stud shear connectors is an aspect requiring further discussion. Likewise, the response of the deck-slab system to external forces, such as negative bending over the girder caused by secondary framing and deck deflections due to applied loads, concrete shrinkage accumulations at areas of least restraint, and changes in deck-slab cross-sectional area due to the fluted nature of the steel deck profile, all may influence local performance.

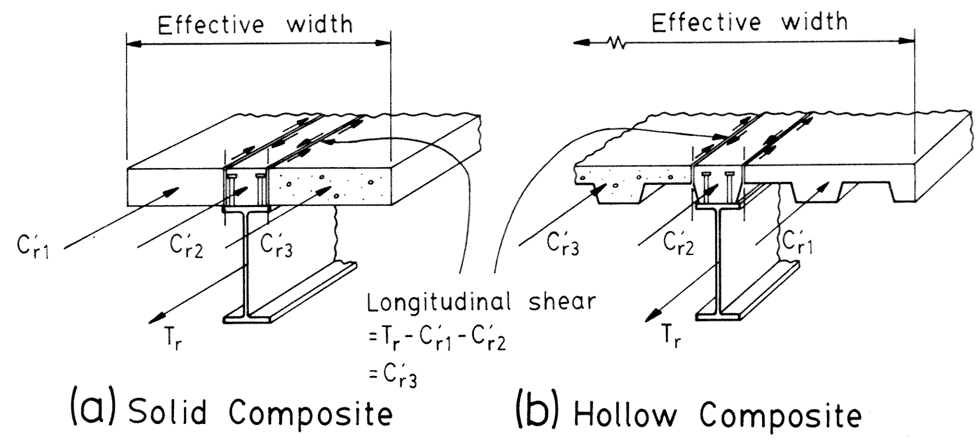


Figure 4.13
Longitudinal Shear due to Composite Action

If we examine the shear force transfer mechanism from steel girders to studs to deck-slabs, we will find a tendency for the concrete slab to shear adjacent to the girder, with the highest slab shear stresses found at the first deck flute because of the abrupt reduction in slab area (see Figs. 4.13a and 4.13b).

Researchers Johnson and Oehlery[4.25] found an occasional tendency for wedging action to split the concrete at stud locations, in lab specimens (Fig. 4.14). However, this phenomenon has not been observed under field conditions, possibly due to their conclusions that in order to be of significant influence, several of the following criteria must exist:

- a low ratio of span to width of slab,
- a high proportion of the total load applied as a single point load,
- a high intensity of longitudinal shear for which transverse reinforcement in excess of 1% of the area of the slab has to be provided,
- shear connection distributed over a narrow width of slab (e.g. a single row of studs).

The longitudinal shear strength of the deck-slab system is related to internal and external influences, and to transverse reinforcement. The following paragraphs will highlight the significant considerations, and differentiate between the "beam" application, with deck perpendicular, and "girder" applications, with deck parallel to the steel member.

Tests on scaled down composite beam specimens using solid slab top chords, by Davies[4.26], revealed that there was no gain in composite beam ultimate strength for amounts of transverse reinforcement in excess of one percent of slab area. His test data further illustrated that a reduction of only four percent of the ultimate beam strength was recorded when transverse reinforcement of 0.47 percent was used instead. From these results, one might conclude that the ultimate flexural strength of a composite beam is rather insensitive to a change in the amount of slab transverse

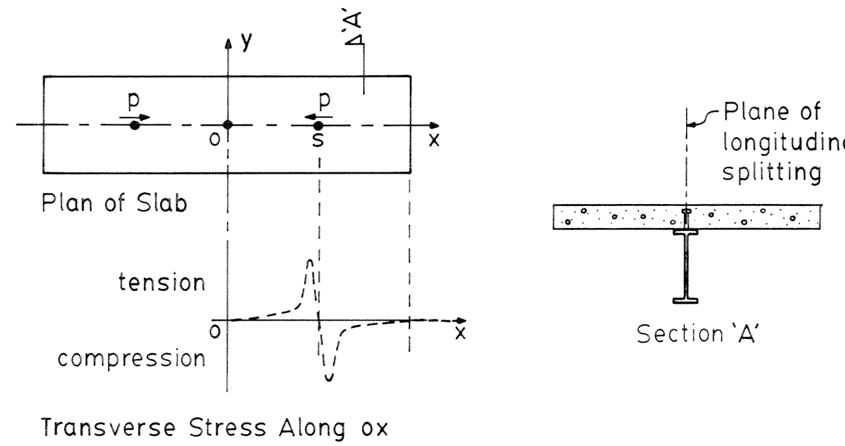


Figure 4.14
Shear Connector Induced
Longitudinal Splitting of Slabs

reinforcing. In the same work[4.26], Davies hypothesized that longitudinal cracks should develop initially in the lower half of the concrete slab, and that only transverse reinforcing close to the bottom of a slab might be credited as effective in resisting such crack action. Later research by Johnson[4.27] indicated that all transverse reinforcement contributes to longitudinal shear strength irrespective of its level in the slab.

The CISC Commentary to S16.1 indicates that longitudinal shear cracking of deck-slabs has not been observed in hollow composite beams, where deck flutes run perpendicular to beam span, possibly because the steel deck provides a measure of reinforcement.

Full-scale hollow composite beam specimens were tested by El-Ghazzi[4.28] and Azmi[4.29]. Although the researchers did not provide a solution to quantify the effects of the amount of transverse reinforcing on the shear resistance of the deck-slab, the test data did show an improvement of 6 percent in the ultimate flexural strength on a 9-metre composite beam resulting from an increase in the transverse reinforcement from 0.096 percent of slab area to 0.263 percent, an increase of 270 percent. In addition, El-Ghazzi suggested that due to the participation of the steel deck, cracks should not necessarily start at the lower part of the slab and then propagate gradually to the top surface, as had been reported[4.26] for solid slab composite beams.

While hollow composite "beams" are generally not critical in longitudinal shear resistance due to the orientation of deck flutes perpendicular to the "beam" spans, detailed design checks should be carried out for hollow composite "girders" where (due to deck flutes being parallel to the steel member) the contribution of the steel deck to the resistance of longitudinal cracking of the deck-slab becomes negligible, especially where steel decks are "cut and spread" (Fig. 1.5c) over a girder to accommodate a particular deck module, to permit fitting of beam flanges under the girder flange (Fig. 4.2), or to permit studs to be welded directly to the girder top flange.

It was noted earlier that in the girder application, external forces can create a more critical situation. Tensile forces in the slab caused by beam-end rotation due to beam deflection under load, deck and shear stud discontinuity causing shrinkage accumulations at the girder, no deck negative bending strength transverse to the girder, and the fact that deck-slab systems over girders will normally be more highly stressed than in the beam-deck-slab configuration, all contribute to this tendency to longitudinal cracking.

With the lack of research data on the girder-deck-slab configuration a need has existed for further information on the influence of slab cracking (from whatever cause), and the influence of transverse reinforcement on slab cracking and ultimate strength of the composite 'girder' section.

Recent tests of such composite girder assemblies by Robinson[4.30] were conducted to compare two levels of slab reinforcement. The assemblies were designed using LSD methods, assuming a 9 m × 9 m bay configuration. An assembly with a nominal 152×152 MW9.1×MW9.1 mesh reinforcement in a 65 mm cover slab on 76 mm wide-rib profile deck on a girder span of 9 m reached full specified loads prior to hair line cracking of the slab. Significant longitudinal slab cracking developed at the attainment of ultimate load. A second assembly, identical in geometry but incorporating additional reinforcing, as shown in Fig. 1.15, attained full ultimate load with better crack distribution and much smaller cracks. No significant variation in ultimate load capacity was found.

It should be noted that test apparatus limitations precluded the application of loads in such a manner that would cause negative transverse bending (other than that caused by the self weight of the structure) across the girder cross-section at the beam girder joints. Further testing incorporating this additional feature would be helpful. However, one might conclude from the two tests above that, even with more realistic transverse bending applied to the section, longitudinal cracking at service loads would be controlled to acceptable levels by the addition of reinforcing similar to the pattern used in the second test.

Study of longitudinal shear capacities of slabs by Buckner et al[4.31] has produced a proposed rational design method as follows:

a) The longitudinal shear strength should be sufficient to develop the capacity of the shear connectors, neglecting any transverse forces which might be created by membrane action of the slab.

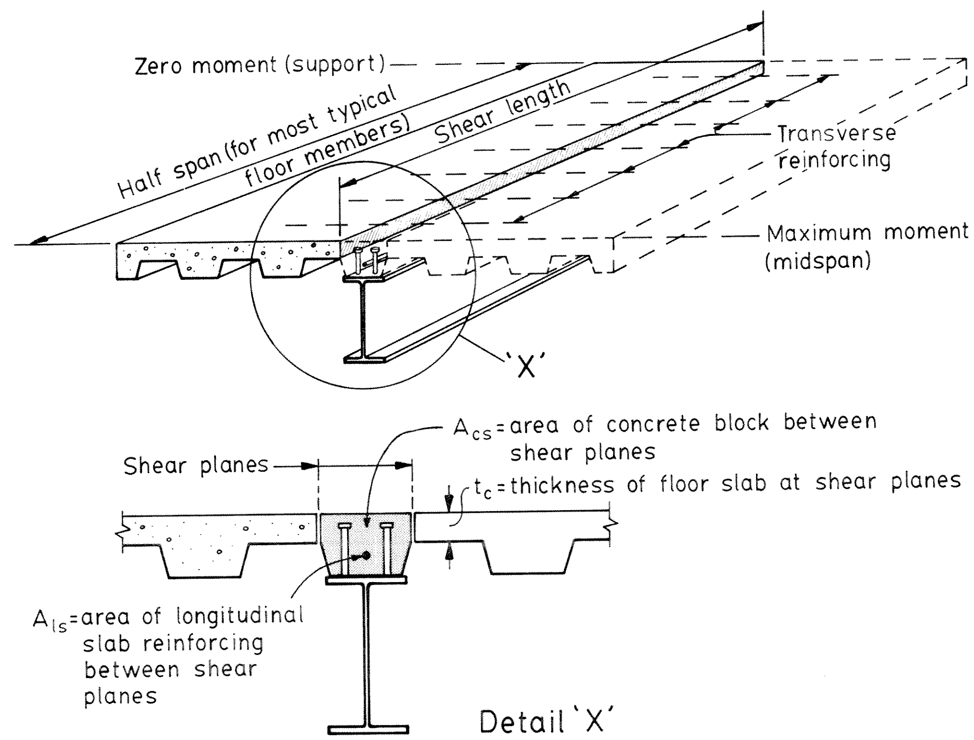


Figure 4.15
Longitudinal Shear Resistance of
Deck-Slabs in Hollow Composite Members

b) The critical planes for longitudinal shear are adjacent to the rows of connectors as indicated in Fig. 4.15. The ultimate longitudinal shear force, V_u, to be transferred across these planes can be approximated by the expression,

$$V_u = Q_u - 0.85 f'_c A_{cs} - A_{ls} f_y$$

in which Q_u = the ultimate horizontal shear capacity of the connectors,
f'_c = the specified compressive strength of concrete,
A_{cs} = the area of concrete block between shear planes,
A_{ls} = the area of longitudinal slab reinforcement between shear planes, and
f_y = the specified yield strength of the reinforcing steel.

The distribution of ultimate shear load in the connectors can be assumed to be uniform. Thus, for the case of a slab extending both sides of a member, the nominal ultimate longitudinal shear stress can be expressed as:

$$v_u = V_u/(2l_{sh}\ t_c) \qquad 4.19$$

in which l_{sh} = the shear length under consideration, t_c = slab thickness in shear.

c) The shear stress computed by Eq. 4.19 should not exceed the limits proposed by Mattock, et al[4.32,4.33]. For normal density concrete, these limits are:

$$v_u \le (0.8\ \rho\ f_y + 2.76)\ \text{MPa, and} \le 0.3\ f'_c \qquad 4.20$$

The limits for semi-low density concrete are:

$$v_u \le (0.8\ \rho\ f_y + 1.72)\ \text{MPa}, \le 0.2\ f'_c, \text{ and} \le 6.9\ \text{MPa} \qquad 4.21$$

and for all low density concrete:

$$v_u \le (0.8\ \rho\ f_y + 1.38)\ \text{MPa}, \le 0.2\ f'_c, \text{ and} \le 5.52\ \text{MPa} \qquad 4.22$$

in which ρ = the ratio of transverse reinforcement; and f_y = the specified yield stress of the reinforcement.

d) At least half the reinforcement required by expressions 4.20 to 4.22 should be placed near the bottom of the slab, in the case of a solid slab.

e) The longitudinal shear stress should be assumed to vary linearly from its critical value, given by Eq. 4.19, to zero at the extreme edges of the effective width of slab. The area of transverse reinforcement can be reduced accordingly. In the absence of an established overall performance factor for longitudinal shear, ϕ_v = 0.60 is assumed in all example calculations in this publication.

$$\phi_v\ (V_u + 0.85\ f'_c\ A_{cs} + A_{ls}\ f_y) \ge Q_r \qquad 4.23$$

4.10 SHORED COMPOSITE BEAMS AND GIRDERS

Shored composite beam construction refers to the situation where the steel shapes are supported by shores during placing of the concrete cover slab. These shores remain in place until the concrete has attained about 75% of its 28 day strength. The advantages and disadvantages of shored construction of composite beams and girders incorporating rolled and welded H shapes were

discussed and illustrated in an example by Ritchie and Chien[4.34]. These can be summarized as follows:

Advantages
- eliminates the need to select a larger or deeper section which will normally be required to support construction slab load and to control deflection
- eliminates need for cambering
- eliminates 'ponding' of concrete due to member deflection during slab pour thus controlling concrete quantities, avoiding potential deck overloading and making level screeding easier

Disadvantages
- defeats the traditionally desirable features of steel construction, (i.e. simplicity and immediate access by other building trades)
- requires the additional cost of shoring and shore removal, plus the fact that shores must stay in place several floors below
- more susceptible to creep deflection as the steel/concrete composite section must carry additional long-term dead load
- instantaneous deflection of beams at shore removal accentuates the negative bending at supports, amplifying the tendency to slab cracking
- requires greater accuracy of design calculations and construction quality control, including negative reinforcement at beam-girder joints
- an unshored design will normally have a greater overload capacity than a shored design

Traditionally, most Canadian construction is designed as unshored construction, with the exception being projects using "stub-girder" construction.

4.11 WEB OPENINGS IN COMPOSITE BEAMS

One important aspect of building design is the accommodation of mechanical ducts and electrical conduits within the "plenum" between the floor and the ceiling in an efficient manner. Structural floor systems using stub-girders, trusses and open web steel joists provide natural web openings for the passage of ducts and pipes. For beams and girders with solid webs, air ducts may pass below the beams and girders, or through web openings.

If economical ducts will not pass beneath the beams, beam depth may be reduced by providing a cover plate to the bottom flange but this approach usually results in an uneconomical design. Web openings are the more normal approach. Composite members with web holes are first selected in accordance with design requirements for composite members without web holes. Then, the effects due to the presence of the hole(s) must be evaluated and accounted for. If a web hole is favourably located to avoid regions of high shear, its effect on the strength and the behaviour of the member may be small. Depending on the size and the location of the hole, solutions in order of economic preference might be: an unreinforced hole; a lightly reinforced hole; a heavier or deeper section with an unreinforced hole; a heavier or deeper section with a lightly reinforced hole.

A guide for the design of composite beams with unreinforced web hole(s) is outlined by Redwood and Wong[4.35]. The capacity of bare steel members under construction loads may be evaluated in accordance with the procedures outlined in the CISC Handbook of Steel Construction or by Redwood and Shrivastava[4.36]. Large, heavily reinforced web holes can be very costly to fabricate and should be avoided if possible or at least restricted to a very few primary framing members. In any event, the total cost required to provide passage of service ducts through webs of floor members should be weighed against the additional cost of a storey height increase. Other systems such as stub-girders, trusses and open web steel joists may also be considered.

4.12 SPANDREL MEMBER DESIGN CONSIDERATIONS

Detailing of the spandrel framing/wall interface requires early and thorough attention to avoid

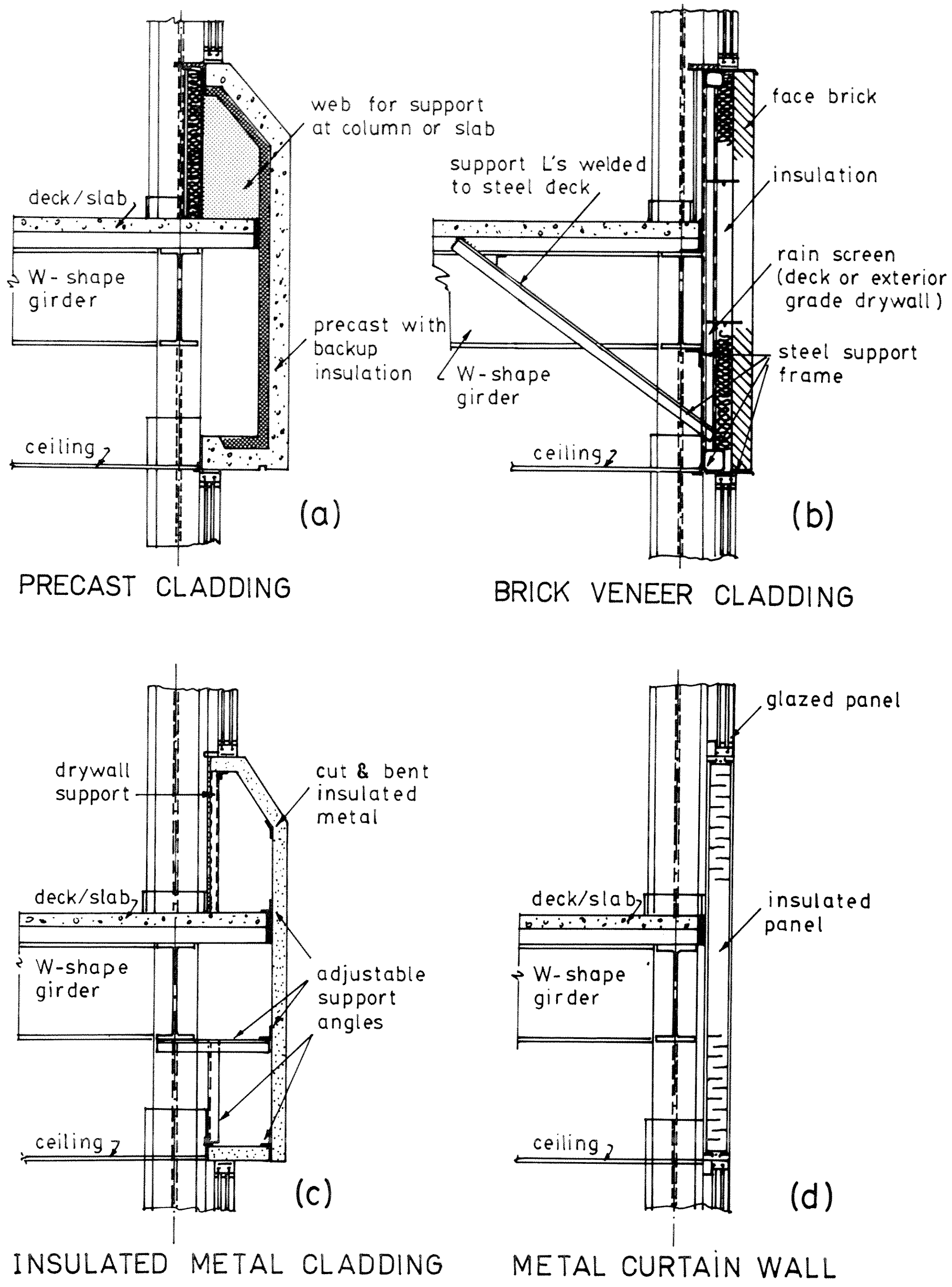


Figure 4.16
Exterior Wall Systems

cost penalties to both the steel frame and the cladding. To begin the design of a spandrel member, the designer must obtain detailed design information on:

- the mass and structural properties of the cladding, including backup material,
- its construction sequence,
- the location of the wall loading in relation to the spandrel member, and
- the position and details of acceptable wall-attachment frames at the spandrel.

Composite spandrel beams and girders incorporating rolled and welded H shapes can be designed relatively 'free' of torsional effects by using details of the type shown in Figures 4.16 (a) to (d). Composite spandrel members, so designed, are usually efficient both structurally and economically. Composite action, for stiffness only, may be considered.

Some practical considerations include:

- in the use of precast concrete cladding with horizontal accentuation, consideration should be given to supporting the precast either directly off column brackets, or off spandrel frame brackets very close to their column connections,
- the load-deflection behaviour of spandrel member at every stage of construction,
- the relative stiffness of adjacent wall-supporting members,
- the structural treatment of torsional load caused by the spandrel details,
- the adjustability of the attachment frames supported by the spandrel members, see Fig. 4.17.
- the treatment of slab overhangs at spandrel beams and girders, (see Figs. 4.18 and 4.19),
- the effects of wind or earthquake loading.

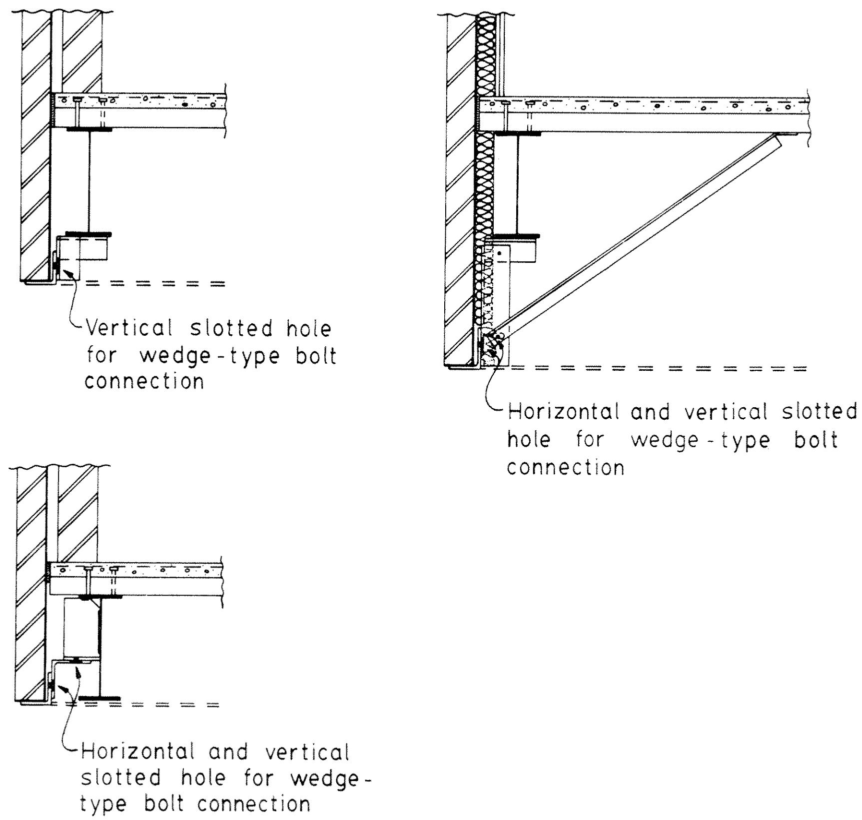
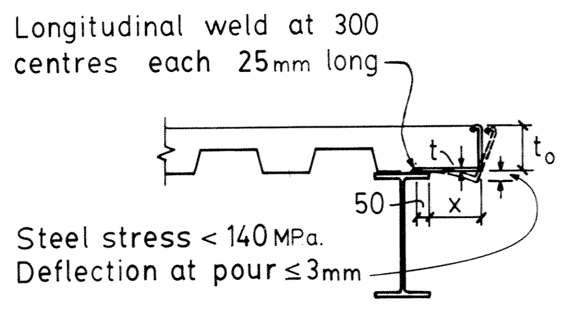


Figure 4.17
Adjustable Wall Supporting Framework Details



t_o (mm)	x (mm)	t (mm)
100	50	0.91
	95	1.22
	140	1.52
	180	1.91
	265	2.67
	340	3.43
140	35	1.22
	80	1.52
	120	1.91
	200	2.67
	285	3.43

Figure 4.18
Required Thickness of Cold-Formed Screed Flash at Spandrel Members

4.13 COMPOSITE BEAM TABLES

The Composite Beam and Girder Selection Tables found in this chapter provide composite and bare steel sectional properties and design data for composite members using W-shapes produced in Canada with nominal depths of 200 mm and deeper, including two WWF-shapes of 700 mm depth. The design data are computed based on CAN3-G40.21-M81 300W steel. Eight combinations of solid slab or cover slab thicknesses and deck depths for two specified concrete strengths are included in Tables 4.1 to 4.8:

Table **4.1** 130 mm solid slab with a specified concrete compressive strength, f'_c, of 20 MPa
Table **4.2** 38 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.3** 51 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.4** 76 mm steel deck and 65 mm cover slab with f'_c of 20 MPa
Table **4.5** 76 mm steel deck and 90 mm cover slab with f'_c of 20 MPa
Table **4.6** 76 mm steel deck and 75 mm cover slab with f'_c of 25 MPa
Table **4.7** 51 mm steel deck and 85 mm cover slab with f'_c of 25 MPa
Table **4.8** 76 mm steel deck and 85 mm cover slab with f'_c of 25 MPa

For each combination, composite sections are listed in descending order of nominal depth and mass of steel shape. The following section properties and design data are tabulated:

- b = flange width of steel shape, in millimetres.
- t = flange thickness of steel shape, in millimetres.
- d = overall depth of steel shape, in millimetres.
- b_1 = effective width of slab used in computing values of M_{rc}, $Q_{r100\%}$, I_t and S_t, in millimetres ($b_1 \leq b + 16t_o$).
- M_{rc} = factored moment resistance of composite section for percentages of shear connection of 50, 75 and 100, in kilonewton metres.
- $Q_{r100\%}$ = required sum of all the factored shear resistances of connectors between point of maximum moment and its adjacent point of zero moment, for 100% shear connection, in kilonewtons, $Q_{r100\%}$ = lesser of $\phi\ A_s\ F_y$ or $0.85\ \phi_c\ b_1\ t_c\ f'_c$.
- I_t = moment of inertia of the composite section, mathematically transformed into steel properties, computed by neglecting concrete in tension, using mass densities shown with each table, in units of 10^6 mm^4.
- S_t = section modulus of the composite section with respect to the extreme fibre of the steel bottom flange based on the value of I_t, in units of 10^3 mm^3.

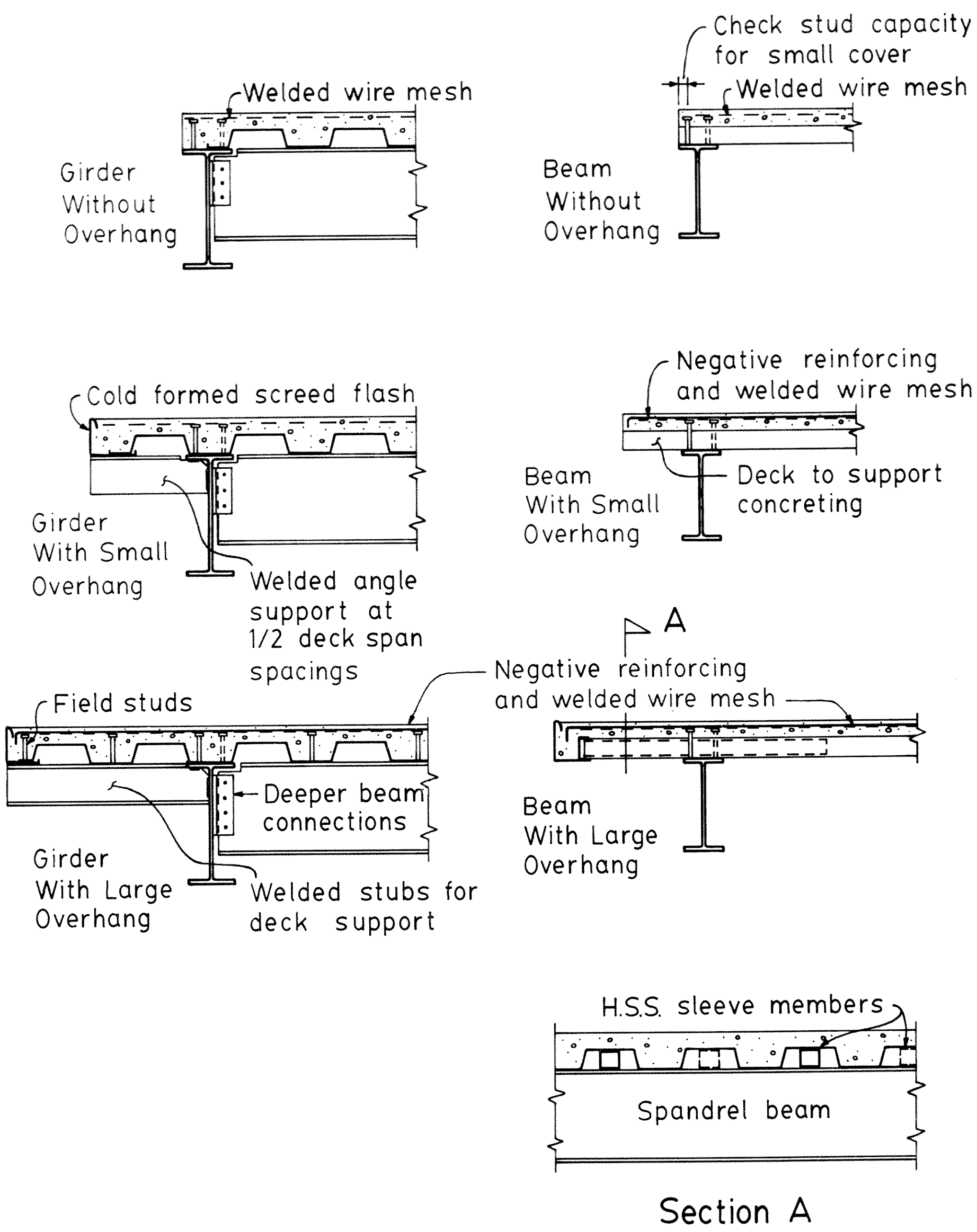


Figure 4.19
Typical Slab Overhang Arrangement for Spandrel Members

M_r = factored moment resistance of laterally supported bare steel section, in kilonewton metres.

V_r = factored shear resistance of steel section, computed in accordance with Clause 13.4.1 of S16.1 for the appropriate h/w ratio, in kilonewtons.

L_u = maximum unsupported length of compression flange of the bare steel member for which no reduction in M_r is required, in millimetres.

I_x = moment of inertia about the x-x axis of the bare steel section, in units of 10^6 mm^4.

S_x = elastic section modulus of bare steel section, in units of 10^3 mm^3.

M_r' = factored moment resistance of the bare steel member for an unsupported length of L′, in kilonewton metres.

4.14 FLOOR DESIGN EXAMPLE

The following example illustrates the design of some typical members in a hollow composite beam and girder floor framing system. One of the composite members will be selected using step-by-step hand calculations, to demonstrate the basic mechanics in the design of a composite member. The result will then be compared to a much quicker solution obtained by using the selection tables. All other members (and components) will be designed by fully utilizing the design aids provided in this publication.

Since all materials that are covered in Chapters 1 to 4 are required for the design of a composite floor, each pertinent area will be duly highlighted in the example.

The quarter floor plan of a typical floor in a multi-storey office building is shown in Fig. 4.E1. Select: interior beams B1, B2, spandrel beam SB, interior girder G and spandrel Girder SG, given specified loads and design requirements as follows:

Fire resistance rating of floor system: 2 hours

Storey height given:

floor to floor height	= 3 660 mm
floor to ceiling height	= 2 590 mm
plenum depth	= 1 070 mm
maximum longitudinal duct depth	= 350 mm

Type of construction: unshored construction with shop-cambered beams and girders

Specified loads:

a) live load = 2.4 kPa (25% sustained), $RF_2 = 0.3 + \sqrt{9.8/A}$

b) fresh-concrete condition dead load = deck + concrete on deflected deck + steel shape

c) superimposed dead loads:

mechanical and ceiling	= 0.5 kPa
fire protection and floor finishes	= 0.2 kPa
partitions	= 1.2 kPa (80% sustained)
spandrel wall	= 3.8 kN/m

d) construction loads: given in Table 3.2

Deflection limitations:

a) interior members:

deflection due to all superimposed specified loads after installation of ceilings ≤ span/300

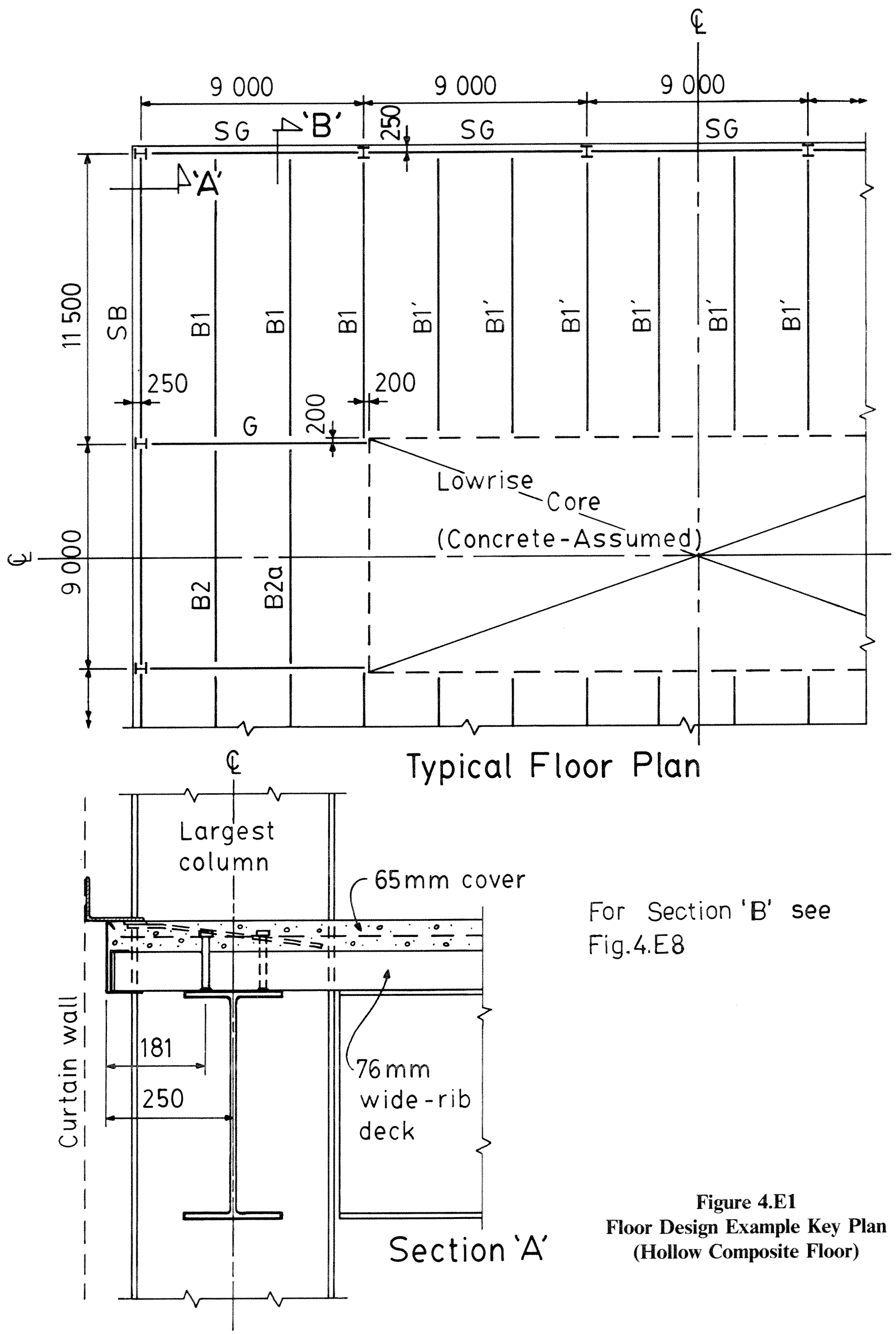


Figure 4.E1
Floor Design Example Key Plan
(Hollow Composite Floor)

b) spandrel members:
deflection due to all superimposed specified loads after concrete casting $\leq$ span/360,
instantaneous deflection due to cladding mass $\leq$ 12 mm (assumed as a chosen criterion for this project)

Materials:

structural steel	: G40.21-M 300W
steel deck	: CSSBI 101 M84, Grade A
shear studs	: 19 mm diameter
concrete	: f'_c = 20 MPa; w_c = 2 300 kg/m^3

Solution

Steel deck: 76 mm wide-rib composite steel deck is selected to span 3 metres.

Fire resistance rating requirement: A floor system consisting of a 65 mm normal density concrete cover slab on 76 mm steel deck with sprayed fire protection on steel beams, girders and deck underside in accordance with ULC Design No. F817 provides the required fire rating.

Steel deck nominal thickness: The steel deck serves as a form and as a construction platform before concrete hardens and subsequently becomes the positive moment reinforcing steel for the cured slab. Although design approaches that are adopted may vary from one deck manufacturer to another, the design procedures for deck-reinforced slabs usually include checking the following criteria:

a) steel deck as form and construction platform
- flexure of steel deck under fresh concrete plus construction loads (see Table 3.2),
- web stability (against crippling at supports) under fresh-concrete and construction loads (see Table 3.2),
- deck deflection due to the total mass of deck and concrete including the effect of concrete ponding ($\Delta \leq$ span/180 $\leq$ 20 mm).

b) deck-reinforced slab
- shear bond between steel deck and concrete,
- flexural tension in steel deck,
- flexural compression in concrete,
- deflection due to superimposed occupancy loads.

In this example, a wide-rib composite steel deck of 0.91 mm nominal thickness that satisfies all the criteria above is selected. Design parameters below are listed in the manufacturer's catalogue.

Dead load due to deck mass, q_d	= 0.10 kPa
Moment of inertia of deck , I_d	= 1.10 $\times$ 10^6 mm^4/m
Dead load due to deck-slab , q	= 2.40 kPa

Interior Beam, B1 — using detailed hand calculations

– live load:

Tributary area, A = 3.0(11.5) = 34.5 m^2

Reduction factor, $RF_2 = 0.3 + \sqrt{9.8/34.5} = 0.83$

Total live load per beam, W_L = 0.83 (2.4)(34.5) = 68.7 kN

– Fresh-concrete condition dead load including concrete ponding:

$w = (1 + 0.20\, w_c\, s^4/I_d)\, s\, q$ Table 3.1, triple span

$= [1 + 0.20(2\,300)(3)^4/1.10\times10^6]\, s\, q$

$= 1.034(3)(2.4)$

$= 7.44$ kN/m

Total fresh-concrete condition load per beam,

$W_c = (7.44 + 0.6)(11.5) = 92.5$ kN (0.6 kN/m steel beam assumed)

– Superimposed dead loads:

Partition load per beam, $W_p = 1.2(34.5) = 41.4$ kN

Other dead loads per beam, $W_{OD} = (0.5+0.2)(34.5) = 24.2$ kN

– Factored loads, moment and shear:

$W_f = \alpha_L W_L + \alpha_D(W_c + W_p + W_{OD})$

$= 1.5(68.7) + 1.25(92.5+41.4+24.2) = 301$ kN

$M_f = W_f L/8 = 301(11.5)/8 = 433$ kN·m

$V_f = W_f/2 = 301/2 = 151$ kN

Trial section: In order to maintain a plenum depth of 1070 mm and to accommodate a 350 mm deep duct without providing web openings, the steel beam depth is restricted to 410 mm as shown in Figure 4.E2.

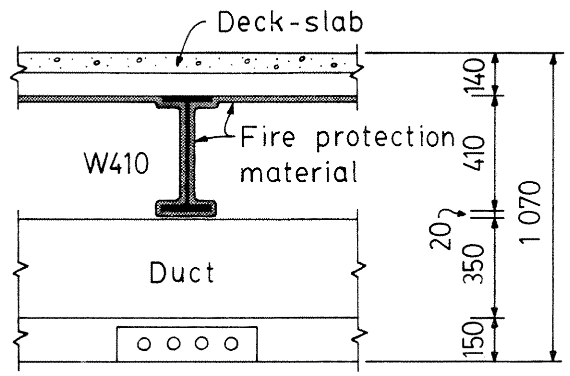


Figure 4.E2
Plenum Depth Computation

Try W410×60 $L/d = 11\,500/410 = 28 < 30$ OK

From CISC Handbook of Steel Construction, Page 6-44,

b = 178 mm	t = 12.8 mm	d = 407 mm	w = 7.7mm
A_s = 7 580 mm^2	I_x = 216$\times$10^{6}mm^4	S_x = 1.06$\times$10^{6}mm^3	Z_x = 1.19$\times$10^{6}mm^3
I_y = 12.0$\times$10^{6}mm^4	J = 328$\times$10^{3}mm^4	C_w = 468$\times$10^{9}mm^6	

self dead load = 0.584 kN/m

– compute factored moment resistance of composite section, M_{rc}

$L/4 = 11\,500/4 = 2\,875$ mm

$16t_o + b = 16(141) + 178 = 2\,434$ mm (Governs)

beam spacing, s $= 3\,000$ mm

Therefore effective slab width, $b_1 = 2\,434$ mm

$0.85\ \phi_c\ b_l\ t_c\ f'_c = 0.85(0.6)(2\ 434)(65)(20)\ 10^{-3} = 1\ 614$ kN
$\phi\ A_s\ F_y = 0.9(7\ 580)(300)\ 10^{-3} = 2\ 047\ \text{kN} > 1\ 614$ kN
Hence, total factored horizontal shear for 100% composite action,
$Q_{r100\%}$ = the lesser of $0.85\ \phi_c\ b_l\ t_c\ f'_c$ and $\phi\ A_s\ F_y$
= 1 614 kN

compute factored shear resistance per shear stud, q_r

$E_c = w_c^{1.5}(0.043)\ \sqrt{f'_c} = (2\ 300)^{1.5}(0.043)\ \sqrt{20} = 21\ 210$ MPa
F_u of shear stud = 415 MPa
$415\ \phi_{sc}A_{sc} = 415(0.8)(19^2\pi/4)\ 10^{-3} = 94.1$ kN (stud diameter $\simeq$ 19 mm, or = ¾ inch)
$0.5\ \phi_{sc}A_{sc}\ \sqrt{f'_c\ E_c} = 0.5(0.8)(19^2\pi/4)\ \sqrt{20(21\ 210)}\ 10^{-3} = 74.3$ kN

q_r = lesser of $0.5\ \phi_{sc}A_{sc}\ \sqrt{f'_c\ E_c}$ and $415\ \phi_{sc}A_{sc}$ (Eq. 2.3)
= 74.3 kN
$2.5t = 2.5(12.8) = 32\ \text{mm} > 19$ mm, and

$$\frac{W_{rib}}{t_d} \geq 2.0 \text{ (wide-rib)}$$

Therefore, there is no reduction in q_r, if not more than one stud per rib. i.e. q_r = 74.3 kN

Minimum number of shear studs per beam requird to provide 50% shear connection

$$= 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\ 614}{74.3}\right) = 21.7,$$ i.e provide at least 22 studs per beam
(use **24 studs** per beam, or 55% connection)

$Q_r = 24(74.3)/2 = 892$ kN

$$a = \frac{Q_r}{0.85\ \phi_c\ b_l\ f'_c} \quad \text{(Eq. 4.10)}$$
$$= \frac{892}{0.85(0.6)(2\ 434)(20)} \times 10^3 = 35.9 \text{ mm}$$

Note: with partial shear connection, the plastic neutral axis, P.N.A. must lie in the steel section. See Fig. 4.E3

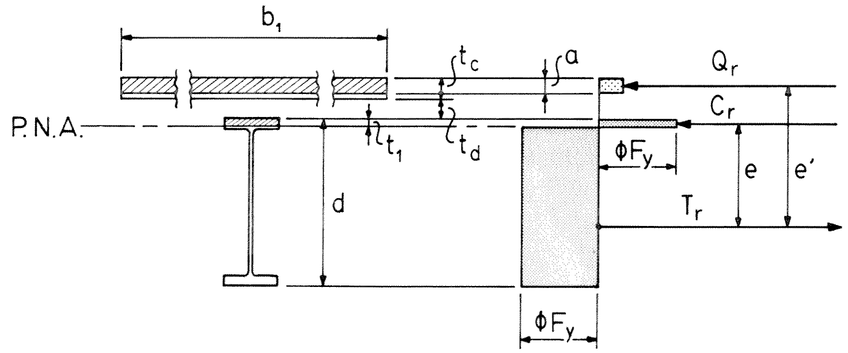


Figure 4.E3
Internal Forces of Beam 'B1'
(with Partial Shear Connection)

$$C_r = \frac{\phi\ A_s\ F_y - Q_r}{2} \quad \text{(Eq. 4.11)}$$

$$= \frac{2\ 047 - 892}{2} = 578 \text{ kN}$$

Locate P.N.A. Factored axial resistance of steel top flange
$= \phi\ b\ t\ F_y$
$= 0.9(178)(12.8)(300)\ 10^{-3} = 615\ \text{kN} > 578$ kN
i.e. P.N.A. in steel top flange

Distance from top of steel section to P.N.A.,

$$t_l = \frac{C_r}{\phi b\ F_y} = \frac{578}{0.9(178)(300)} \times 10^3 = 12.0 \text{ mm}$$

Lever arms,

$$e = \frac{(A_s d - b t_l^2)}{2(A_s - b t_l)} - \frac{t_l}{2} \quad \text{(Eq. 4.7a)}$$

$$= \frac{7\ 580(407) - 178(12)^2}{2[7\ 580 - 178(12)]} - \frac{12}{2}$$

= 275 mm

$$e' = e + \frac{t_l}{2} + t_o - \frac{a}{2} \quad \text{(Eq. 4.7b with } t_c \text{ replaced by a)}$$

$$= 275 + \frac{12.0}{2} + 141 - \frac{34.8}{2} = 405 \text{ mm}$$

Factored moment resistance,

$M_{rc} = C_r e + Q_r e'$ (Eq. 4.12)
$= [578(275) + 892(405)]\ 10^{-3} = 520$ kN•m
$M_f = 433\ \text{kN•m} < 520$ kN•m OK

– factored shear resistance

$V_r = \phi\ A_w\ F_s$ (Eq.4.13)
$= \phi\ d\ w\ (0.66F_y)$
$= 0.9(407)(7.7)(0.66)(300) \times 10^{-3} = 558$ kN
$V_f = 151\ \text{kN} < 558$ kN OK

– construction stage 1 – deck placement (no lateral support provided before deck welded to steel top flange)
$A = 34.5\ m^2 > 16\ m^2$ i.e. consider U.D.L. only
$7\ m^2 < A < 54\ m^2$.

Therefore,

$q_L' = 0.7 - A/135$ (see Table 3.2)

$= 0.7 - 34.5/135 = 0.44$ kPa

$W_f' = [1.25(0.10) + 1.5(0.44)]\,(34.5) + 1.25(0.584)(11.5) = 35.5$ kN

$M_f' = 35.5(11.5)/8 = 51.0$ kN·m

$$M_u = \frac{\pi}{\omega L'}\sqrt{EI_yGJ + \left(\frac{E\pi}{L'}\right)^2 I_yC_w} \qquad \text{(Eq. 4.16)}$$

$$= \frac{\pi}{1.0(11.5)}\sqrt{\left[200(12.0)(77)(328) + \left(\frac{200\,\pi}{11.5}\right)^2(12.0)(468)\right] \times 10^{-3}}$$

$= 76.0$ kN·m

$\frac{2}{3} M_p = \frac{2}{3} Z_x F_y$ (W410×60 of 300W steel is Class 1 section in bending)

$= \frac{2}{3}(1.19)(300)$

$= 238$ kN·m > 76.0 kN·m. Therefore,

$M_r' = \phi M_u$ (Eq.4.14)

$= 0.9(76.0)$

$= 68.4$ kN·m > 51.0 kN·m OK

– construction stage 2 – concrete placement

$A > 8\ m^2$ i.e. consider U.D.L. only

$q_L = 2 q_L' = 0.88$ kPa (see Table 3.2)

$W_f = 1.5(0.88)(34.5) + 1.25(92.5) = 161$ kN

$M_f = 161(11.5)/8 = 231$ kN·m

$M_r = \phi Z_x F_y = 0.9(1.19)(300) = 321$ kN·m > 231 kN·m OK

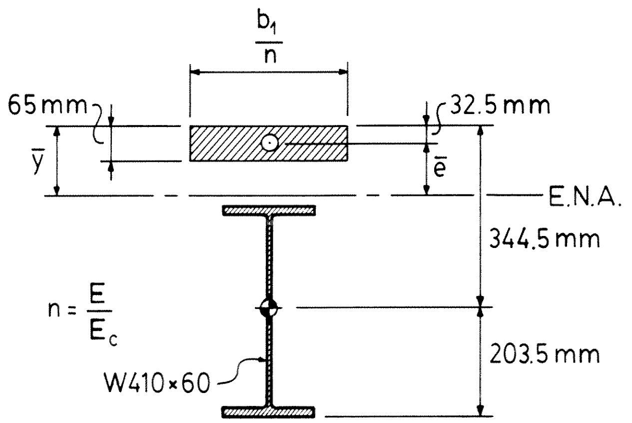


Figure 4.E4
Cross-Section of Composite Beam 'B1'
(Transformed into Elastic Steel Properties)

– Sectional properties of composite beam (see Fig. 4.E4)

a) Find moment of inertia of composite section, I_t (in steel unit)

$n = E/E_c = 200\,000/21\,210 = 9.43$

Section	Transformed Area, A (mm^2) in steel unit	Distance from top of slab, y (mm)	Ay mm^3	Ay^2 mm^4	I_{local} mm^4 in steel unit
Concrete	16 777	32.5	545 252	17.7×10^6	5.91×10^6
Steel	7 580	344.5	2 611 310	899.6×10^6	216×10^6
Total	24 357		3 156 562	917.3×10^6	221.9×10^6

$\bar{y} = \frac{3\,156\,562}{24\,357} = 129.6$ mm > 65 mm

i.e. all effective concrete is in compression

$I_t = \Sigma I_{local} + \Sigma A y^2 - \bar{y}^2 \Sigma A$

$= (221.9 + 917.3) \times 10^6 - (129.6)^2(24\,357) = 730 \times 10^6\ mm^4$

$S_t = \frac{I_t}{d + t_o - \bar{y}} = \frac{730\times10^6}{407+141-129.6} = 1.745 \text{ x } 10^6\ mm^3$

b) Find moment of inertia of composite section reduced to account for concrete creep, I_r. ($E_r = E_c/2.5$)

$n_r = E/E_r = 2.5(200\,000)/21\,210 = 23.6$

Section	Transformed Area, A (mm^2) in steel unit	Distance from top of slab, y (mm)	Ay mm^3	Ay^2 mm^4	I_{local} mm^4 in steel unit
Concrete	6 704	32.5	217 900	7.08×10^6	2.4×10^6
Steel	7 580	344.5	2 611 310	899.6×10^6	216×10^6
Total	14 284		2 829 210	906.7×10^6	218.4×10^6

$\bar{y} = \frac{2\,829\,210}{14\,284} = 198$ mm > 65 mm

$I_r = (218.4 + 906.7)\times10^6 - (198)^2(14\,284) = 565 \times 10^6\ mm^4$

– Deflection estimate

a) Camber requirement:
Deflection of unshored beam under fresh-concrete condition load,

$$\Delta_c = \frac{5W_cL^3}{384EI_x} = \frac{5(92.5)(11.5)^3}{384(200)(216)} \times 10^3 \text{ mm} = 42 \text{ mm}$$

Therefore **camber 40 mm** at mid span.

b) Shrinkage deflection

$$\bar{e} = \bar{y} - \frac{t_c}{2} = 129.6 - \frac{65}{2} = 97.1 \text{ mm}$$

$$\Delta_{sh} = \frac{\bar{e}\,\epsilon\,t_c\,b_1\,L^2}{8\,n\,I_t} \quad \text{(from Figure 4.E5)}$$

$$= \frac{97.1(0.0002)(65)(2\,434)(11.5)^2}{8(9.43)(730)}$$

$$= 7.4 \text{ mm} \quad \text{(Shrinkage strain, } \epsilon = 0.0002 \text{ assumed)}$$

c) Creep deflection
Total sustained load, W_s
$= 0.25W_L + 0.8W_p + W_{OD}$
$= 0.25(68.7) + 0.8(41.4) + 24.2 = 74.5$ kN

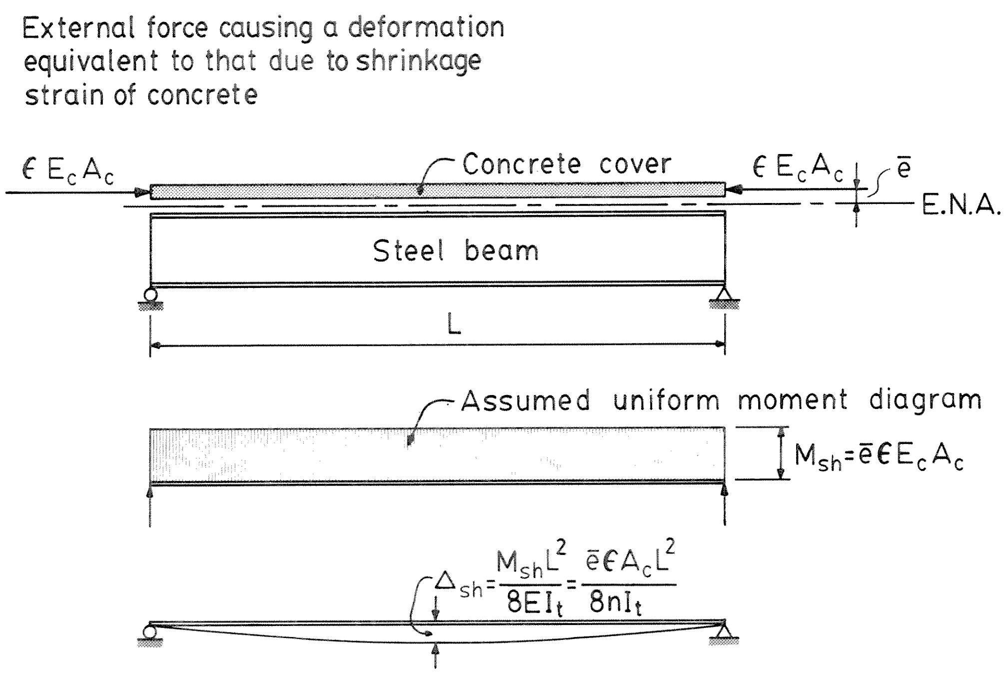


Figure 4.E5
Shrinkage Deflection of Composite Beams
(by analysing the structure as an eccentrically loaded column)

Δ_{creep} = (beam deflection with creep effect)
– (instantaneous deflection of the transformed section)

$$\Delta_{creep} = \frac{5W_sL^3}{384E}\left(\frac{1}{I_r} - \frac{1}{I_t}\right) = \frac{5(74.5)(11.5)^3}{384(200)}\left(\frac{1}{565} - \frac{1}{730}\right) \times 10^3 = 3.0 \text{ mm}$$

d) Deflection due to live load and partition including long-term effects

$$\Delta = \frac{5(W_L + W_p)L^3}{384\,E\,I_e} + \Delta_{creep} + \Delta_{sh}$$

$$= \frac{5(68.7 + 41.4)(11.5)^3\,10^3}{384(200)(592)} + 3.0 + 7.4$$

$$= 28.8 \text{ mm} = L/399 < L/300 \quad \text{OK}$$

where I_e = effective moment of inertia
$= I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$ (Eq. 4.17)
$= [216 + 0.85\,(0.55)^{0.25}\,(730 - 216)] \times 10^6$ mm^4
$= 592 \times 10^6$ mm^4
where $I_s = I_x = 216 \times 10^6$ mm^4

– Unshored beam requirement

Moment due to specified fresh-concrete condition load acting on bare steel beam,

$$M_b = W_cL/8 = 92.5(11.5)/8 = 132.9 \text{ kN·m}$$

Moment due to all specified superimposed loads acting on composite beam (i.e. after concrete attains 75% f'_c),

$$M_t = (W_L + W_p + W_{OD})\,L/8 = (68.7 + 41.4 + 24.2)(11.5)/8 = 193.1 \text{ kN·m}$$

Combined stresses in bottom flange under specified loads become,

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{132.9}{1.060} + \frac{193.1}{1.745} = 236 \text{ MPa} < 0.9\,F_y \quad \text{(Eq. 4.18)}$$

Therefore shoring is not required.

W410×60 with 55% shear connection is satisfactory.

The total amount of computation presented above can be substantially reduced by using the Composite Beam and Girder Selection Tables provided. The selection of beam B1 with the aid of design tables is illustrated below:

Interior beam B1 – using design tables provided in Section 4.13

From Table 4.4, for **W410×60** with b_1 = 2 430 mm,

$M_{rc50\%}$ = 509 kN·m > 433 kN·m
V_r = 558 kN > 151 kN OK
$Q_{r100\%}$ = 1 610 kN

For 50% shear connection,

$$\text{Minimum number of studs} = 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,610}{74.3}\right) \text{, (from Table 2.1, } q_r = 74.3 \text{ kN)}$$

$= 21.7$ i.e a minimum of 22 studs per beam (**24 studs** per beam – detailed)

Since deck module is 406 mm, these studs can easily be accommodated on a one stud per rib basis, justifying their use as single-stud per rib connections (see Fig. 4.E12)

– Construction stages

$M_r' = 68.6$ kN·m (by interpolation - Table 4.4, $L' = 11\,500$ mm)
> 51.0 kN·m OK for deck placement stage.
$M_r = 321$ kN·m (Table 4.4)
> 231 kN·m OK for concrete placement stage.

– Deflection estimate

From Table 4.4
$d = 407$ mm $\quad I_x = 216\times10^6$ mm^4
$I_t = 730\times10^6$ mm^4 $\quad S_t = 1.74\times10^6$ mm^3

a) Camber requirement

$$\Delta_c = \frac{5(92.5)(11.5)^3}{384(200)(216)} \times 10^3 = 42 \text{ mm}$$

Therefore **camber 40** mm at mid span.

b) Shrinkage deflection

$$\bar{e} = d + t_d + \frac{t_c}{2} - \frac{I_t}{S_t} = 407 + 76 + \frac{65}{2} - \frac{730}{1.74} = 96.0 \text{ mm}$$

$$\Delta_{sh} = \frac{\bar{e}\,\epsilon\, t_c\, b_l\, L^2}{8\, n\, I_t} = \frac{96.0(0.0002)(65)(2\,430)(11.5)^2}{8(9.43)(730)} = 7.3 \text{ mm}$$

c) Deflection due to live load plus partitions including long term effects

$$\Delta = \frac{5(W_L + W_p)L^3}{384\, E\, I_e}(1.15 \text{ for creep}) + \Delta_{sh}$$

$$= \frac{5(68.7 + 41.4)(11.5)^3\, 10^3}{384(200)(592)}(1.15) + 7.3$$

$= 28.5$ mm $= L/404 < L/300$ OK

Note: value of $I_e = 592$ (x 10^6 mm^4), as shown before.

– Unshored beam requirement

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{132.9}{1.060} + \frac{193.1}{1.74} = 236 \text{ MPa} < 0.9F_y$$

Therefore shoring is not required.

W410×60 with 50% shear connection is satisfactory.

Interior beam B2

– Live load

$A = 3.0(9.0) = 27$ m^2
$RF_2 = 0.3 + \sqrt{9.8/27} = 0.90$
$W_L = 0.90(2.4)(27) = 58.3$ kN

– Dead loads

$W_c = (7.44 + 0.4)(9.0) = 70.6$ kN (w = 7.44 kN/m, 0.4 kN/m steel beam assumed)
$W_p = 1.2(27) = 32.4$ kN
$W_{OD} = 0.7(27) = 18.9$ kN

– Factored total load, moment and shear

$W_f = 1.5(58.3) + 1.25(70.6 + 32.4 + 18.9) = 240$ kN
$M_f = 240(9.0)/8 = 270$ kN·m
$V_f = 240/2 = 120$ kN

Try **W410×39** $L/d = 9000/410 = 22 < 30$ OK for a trial section.
$L/4 = 9000/4 = 2\,250$ mm (Governs)
Beam spacing $= 3\,000$ mm
$16t_o + b = 2\,400$ mm
For $b_l = 2\,250$ mm, $M_{rc50\%} = 349$ kN·m (Table 4.4)
> 270 kN·m OK
$Q_{r100\%} = 1\,330$ kN (Table 4.4)

$2.5\,t = 2.5\,(8.8) = 22$ mm > 19 mm.
Therefore no reduction in q_r, ($q_r = 74.3$ kN)

$$\text{Minimum number of studs} = 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,330}{74.3}\right)$$

$= 17.9$ i.e. **18 studs** per beam

(See stud layout, Fig. 4.E11; all single-stud per rib connections, therefore q_r OK)

$V_r = 448$ kN (Table 4.4)
> 120 kN OK

– Construction stage 1 - Deck placement

$A = 27$ m$^2 > 16$ m^2 Therefore consider U.D.L. only.
$q_L' = 0.5$ kPa (see Table 3.2)
$W_f' = [1.5(0.5)+1.25(0.10)](27)+1.25(0.4)(9.0)$ (0.4 kN/m steel beam assumed)
$= 28.1$ kN
$M_f' = 28.1(9.0)/8 = 31.6$ kN·m

Since no lateral support can be assumed, $L' = 9\ 000$ mm.
Hence,

$M_r' = 31.3$ kN·m (Table 4.4)
$\simeq 31.6$ kN·m say OK. (under designed by 1%)

– Construction stage 2 - Concrete placement

$q_L = 1.0$ kPa (see Table 3.2)
$W_f = 1.5(1.0)(27) + 1.25(70.6) = 129$ kN
$M_f = 129(9.0)/8 = 145$ kN·m
$M_r = 197$ kN·m (Table 4.4)
> 145 kN·m OK

– Deflection estimate

From Table 4.4,
$d = 399$ mm $I_x = 127 \times 10^6$ mm^4
$I_t \simeq 490 \times 10^6$ mm^4 (by interpolation)
$S_t \simeq 1.14 \times 10^6$ mm^3 (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{5(70.6)(9.0)^3}{384(200)(127)} \times 10^3 = 26 \text{ mm}$$

Therefore **camber 25** mm at mid span.

b) Shrinkage deflection

$$\bar{e} = 399 + 76 + \frac{65}{2} - \frac{490}{1.14} = 77.7 \text{ mm}$$

$$\Delta_{sh} = \frac{77.7(0.0002)(65)(2\ 250)(9)^2}{8(9.43)490} = 5.0 \text{ mm} \qquad \text{(see Fig. 4.E5)}$$

c) Deflection due to live load plus partition including long term effects

$$\Delta = \frac{5(58.3+32.4)(9)^3\ 10^3}{384(200)\ I_e}(1.15) + 5.0$$

$$= 17.8 \text{ mm} = L/506 < L/300 \qquad \text{OK}$$

where $I_e = I_s + 0.85\ (p)^{0.25}\ (I_t - I_s)$
$= [127 + 0.85\ (0.5)^{0.25}\ (490 - 127)] \times 10^6$
$= 386\ (\times 10^6 \text{ mm}^4)$

– Unshored beam requirement

$M_b = 70.6(9.0)/8 = 79.4$ kN·m
$M_t = (58.3 + 32.4 + 18.9)(9.0)/8 = 123$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{79.4}{0.634} + \frac{123}{1.14} \qquad \text{(from Table 4.4, } S_x = 0.634 \times 10^6 \text{ mm}^3)$$

$$= 233 \text{ MPa} < 0.9\ F_y$$

Therefore, shoring is not required and W410×39 with 50% shear connection is satisfactory.

Spandrel Beam SB

– Design loads

$L = 11\ 500$ mm slab overhang $= 250$ mm
$A = (1.5+0.250)(11.5) = 20.1$ m^2
$RF_2 \simeq 1.0$
$W_L = 2.4(20.1)(1.0) = 48.2$ kN
$W_c = [1.034(2.4)(1.75)+0.7](11.5) = 58.0$ kN (0.7 kN/m steel beam assumed)
$W_w = 3.8(11.5) = 43.7$ kN (spandrel wall load)
$W_p + W_{OD} = (1.2 + 0.7)(1.75)(11.5) = 38.2$ kN

– Factored total loads, moment and shear

$W_f = 1.5(48.2) + 1.25(58.0 + 43.7 + 38.2) = 247.2$ kN
$M_f = 247.2\ (11.5)/8 = 355$ kN·m
$V_f = 247.2/2 = 124$ kN

Try **W460×67** $L/d = 11\ 500/460 = 25 < 30$ OK as a trial section
$b = 190$ mm (Table 4.4)

$0.10(11.5) = 1\ 150$ mm
$6(141) = 846$ mm (Governs)
Half clear spacing $= 0.5(3\ 000 - 95 - 89) = 1\ 408$ mm

$b_1 = 155 + 190 + 846 = 1\ 190$ mm (say) (See Fig. 4.E6)

$M_{rc50\%} = 519$ kN·m (by interpolation from Table 4.4)
> 355 kN·m OK

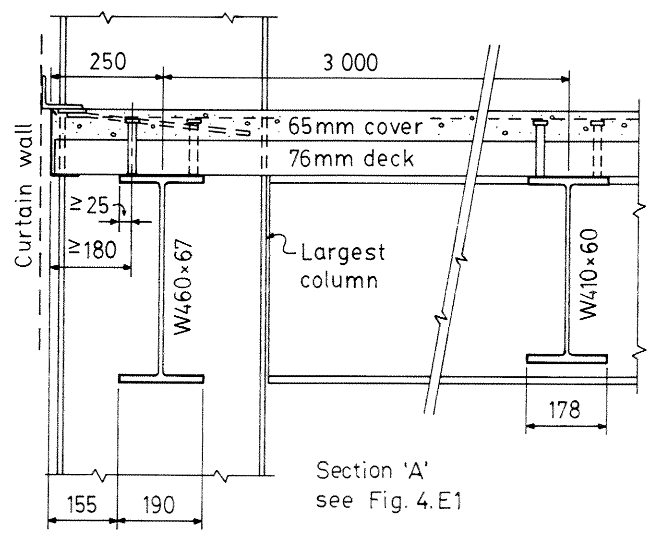


Figure 4.E6
Spandrel Beam Cross-Section and Slab Overhang Detail

$Q_{r100\%}$ = 789 kN (by interpolation from Table 4.4)

2.5t = 2.5(12.7) = 31.8 mm > 19 mm, and
edge distance = 155+25 = 180 mm > 141 mm.

Hence no reduction in stud capacity (from Fig. 2.4, q_r = 74.3 kN).

Minimum number of studs = $(2)\left(\frac{50\%}{100\%}\right)\left(\frac{789}{74.3}\right) = 10.6$ i.e. **12 studs** per beam (or 57% connection)

V_r = 688 kN (Table 4.4)
> 124 kN OK

– Construction stage 1 – Deck placement

A = 20.1 m^2 > 16 m^2 Therefore consider U.D.L. only.
A = 20.1 m^2 < 27 m^2
q_L' = 0.5 kPa (see table 3.2)
W_f' = [1.5(0.5)+1.25(0.10)](20.1)+1.25(0.7)(11.5) = 27.7 kN
M_f' = 27.7 (11.5)/8 = 39.8 kN·m
No lateral support is assumed before deck is welded to beam, L′ = 11 500 mm
M_r' = 87.2 kN·m (by interpolation, Table 4.4)
> 39.8 kN·m OK

– Construction stage 2 – Concrete placement

q_L = 1.0 kPa
W_f = 1.5(1.0)(20.1)+1.25(58.0) = 102.7 kN
M_f = 102.7(11.5)/8 = 148 kN·m
M_r = 405 kN·m (Table 4.4)
> 148 kN·m OK

– Deflection estimate

From Table 4.4
d = 454 mm $I_x = 300 \times 10^6$ mm^4
$I_t \simeq 776 \times 10^6$ mm^4 (by interpolation)
$S_t \simeq 1.99 \times 10^6$ mm^3 (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{5(58.0)(11.5)^3}{384(200)(300)} \times 10^3 = 19 \text{ mm}$$

Camber 20 mm at mid span.

b) Instantaneous deflection due to cladding mass

$$\Delta_w = \frac{5(43.7)(11.5)^3}{384(200)(776)} \times 10^3 (1 + 15\% \text{ for deck profile} + 15\% \text{ for partial connection})$$

$$= 7.2 \text{ mm} < 12 \text{ mm} \quad \text{OK}$$

c) Shrinkage deflection

$$\bar{e} = 454 + 76 + \frac{65}{2} - \frac{776}{1.99} = 173 \text{ mm}$$

$$\Delta_{sh} = \frac{173(0.0002)(65)(1\ 190)(11.5)^2}{8(9.43)(776)} = 6.0 \text{ mm} \quad \text{(see Fig. 4.E5)}$$

d) Deflection due to all superimposed loads including long term effects

$$\Delta = \frac{5(48.2+43.7+38.2)(11.5)^3\ 10^3}{384(200)\ I_e}(1.15) + 6.0$$
$$= 28.7 \text{ mm} = L/400 < L/360 \quad \text{OK}$$

where $I_e = I_s + 0.85\ (p)^{0.25}\ (I_t - I_s)$
$= [300 + 0.85\ (0.57)^{0.25}\ (776 - 300)]\ 10^6$
$= 652\ (\times 10^6\ mm^4)$

– Unshored beam requirement

$S_x = 1.32 \times 10^6$ mm^3 (Table 4.4)
M_b = 58.0(11.5)/8 = 83.4 kN·m
M_t = (48.2+43.7+38.2)(11.5)/8 = 187 kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{83.4}{1.32} + \frac{187}{1.99} = 157 \text{ MPa} < 0.9\ F_y$$

Therefore shoring is not required.

W460×67 with 50% shear connection is satisfactory.

Interior Girder G

– Live Load

A $\simeq$ 2(3.0)(11.5+9.0)/2 = 61.5 m^2*
RF_2 = $0.3 + \sqrt{9.8/61.5} = 0.70$
P_{L1} = 0.70(2.4)(31.2) = 52.4 kN
P_{L2} = 0.70(2.4)(17.3) = 29.1 kN

*For an asymmetrically loaded girder, the tributary area for live load reduction seems less well defined. A conservative approach is adopted here.

– Dead Loads

P_{c1} $\simeq$ (92.5 + 70.6)/2 + 1.034(2.4)(0.10)(4.5) = 82.7 kN
P_{c2} $\simeq$ 92.5/2 = 46.2 kN
W_c = 0.8(9.0) = 7.2 kN (0.8 kN/m steel girder assumed)
P_{p1} $\simeq$ (41.4 + 32.4)/2 + 1.2(0.10)(4.5) = 37.4 kN
P_{p2} = 41.4/2 = 20.7 kN
P_{OD1} $\simeq$ (24.2 + 18.9)/2 + 0.7(0.10)(4.5) = 21.9 kN
P_{OD2} = 24.2/2 = 12.1 kN

– Factored total loads, moment and shear

$P_{f1} = 1.5(52.4) + 1.25(82.7 + 37.4 + 21.9) = 256$ kN
$P_{f2} = 1.5(29.1) + 1.25(46.2 + 20.7 + 12.1) = 142$ kN
$W_f = 1.25(7.2) = 9.0$ kN
$V_f = [(9.0)P_{f2} + (3.0+6.0)P_{f1} + 4.6W_f]/9.2$
$= [(9.0)(142) + (9.0)(256) + 4.6(9.0)]/9.2 = 394$ kN
$M_f = 3.2(394) - 3.0(142) - (3.2)^2(1.25)(0.8)/2 = 830$ kN·m

Try W530×92

$16t_o + b$	=	2 470 mm (Table 4.4)
Average girder spacing	=	10 250 mm
$L/4 = 9\,200/4$	=	2 300 mm (Governs b_1)

From Table 4.4, for $b_1 = 2\,300$ mm,

$M_{rc50\%} = 880$ kN·m (by interpolation)
> 830 kN·m OK
$V_r = 969$ kN > 394 kN OK
$Q_{r100\%} = 1\,530$ kN (by interpolation)

$2.5\ t = 2.5(15.6) = 39$ mm > 19 mm (19 mm studs used)
Therefore, no reduction in shear stud capacity

i.e. $q_r = 74.3$ kN (Table 2.1)

$$\text{Minimum number of studs} = 2\left(\frac{50\%}{100\%}\right)\left(\frac{1\,530}{74.3}\right) = 20.6$$

Use no less than **22 studs** per girder or 11 studs (53% connection) on each side of point of maximum moment (see Fig. 4.E7). The studs required in shear span 2 (11 studs) can be uniformly distributed whereas the stud distribution between the point of maximum moment and the end support at the column depends on the number of studs required in shear span 1 (and other factors that will be dealt with later under steel deck and shear stud layouts).

Number of studs required in shear span 1,

$n' = n(M_{f1} - M_r)/(M_f - M_r)$ (Eq. 2.10)

$= \frac{20.6}{2}\left(\frac{803 - 637}{830 - 637}\right)$ ($M_r = 637$ kN·m, Table 4.4)

$= 8.9$ i.e. 9 studs required

Therefore at least 2 studs are required between two point loads, P_{f1}.

– Construction stage

$A = 61.5\ m^2 > 54\ m^2$ i.e. $q_L = 0.6$ kPa (Table 3.2)
$P_{f1} = 1.5(0.6)[3.0(5.75)+3.1(4.5)] + 1.25(82.7) = 131$ kN
$P_{f2} = 1.5(0.6)(3.0)(5.75) + 1.25(46.2) = 73.3$ kN

$$M_f = [9.0(P_{f1} + P_{f2}) + 4.6\ W_f]\frac{3.2}{9.2} - 3.0\ P_{f2} - (3.2)^2\ (1.25)(0.8)/2$$

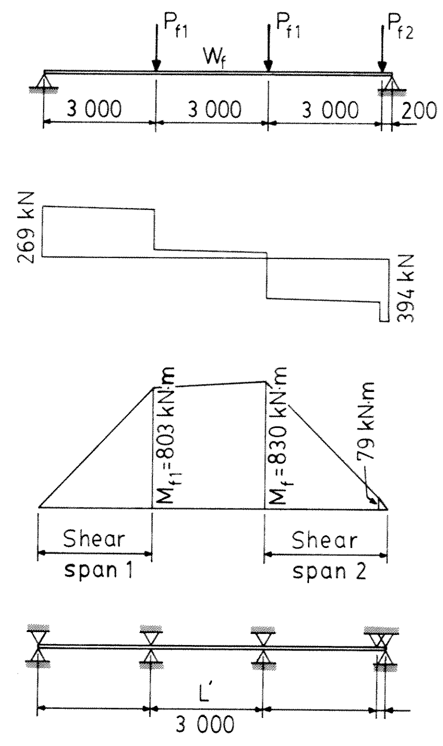


Figure 4.E7
Shear and Moment Diagrams of Floor Girder 'G'

$$= [9.0(131+73.3) + 4.6(9.0)]\frac{3.2}{9.2} - 3.0\ (73.3) - 5.12$$

$= 429$ kN·m

For $L' = 3\,000$ mm, $M_r = 633$ kN·m (Table 4.4)
> 429 kN·m OK

– Deflection estimate

From Table 4.4, $d = 533$ mm $I_x = 552 \times 10^6\ mm^4$
$I_t = 1\,510 \times 10^6\ mm^4$ (by interpolation)
$S_t = 3.12 \times 10^6\ mm^4$ (by interpolation)

a) Camber requirement

$$\Delta_c \simeq \frac{L^3}{48EI_x}\left\{\Sigma\left[P\left(\frac{3a}{L} - \frac{4a^3}{L^3}\right)\right] + \frac{5}{8}\ W_c\right\}$$

where a = distance from each load point to the nearest support

$$\Delta_c \simeq \frac{L^3}{48EI_x}\left\{P_{c1}\left[\frac{3(3.0)}{L} - \frac{4(3.0)^3}{L^3}\right] + P_{c1}\left[\frac{3(3.2)}{L} - \frac{4(3.2)^3}{L^3}\right]\right.$$

$$\left. + P_{c2}\left[\frac{3(0.2)}{L} - \frac{4(0.2)^3}{L^3}\right] + \frac{5}{8}W_c\right\}$$

$$= \frac{(9.2)^3 \times 10^3}{48(200)(552)}\left\{82.7(1.715) + 46.2(0.0652) + 0.625(7.2)\right\} = 22 \text{ mm}$$

Camber 20 mm at mid span.

b) Shrinkage deflection

$$\bar{e} = 533 + 76 + \frac{65}{2} - \frac{1\,510}{3.12} = 158 \text{ mm}$$

$$\Delta_{sh} = \frac{158(0.0002)(65)(2\,300)(9.2)^2}{8(9.43)(1\,510)} = 3.5 \text{ mm}$$

c) Deflection due to live load plus partition including long term effects

$$\Delta = \frac{L^3}{48EI_e}\left\{(P_{L1}+P_{p1})(1.715)+(P_{L2}+P_{p2})(0.0652)\right\}(1.15) + \Delta_{sh}$$

$$= \frac{(9.2)^3(10^3)}{48(200)\,I_e}\left\{(52.4+37.4)(1.715)+(29.1+20.7)(0.0652)\right\}(1.15)+3.5$$

$= 15.3$ mm $= L/601 < L/300$ OK

where $I_e = I_s + 0.85\,(p)^{0.25}\,(I_t - I_s)$

$$= \left[552 + 0.85\,(0.53)^{0.25}\,(1\,510 - 552)\right] 10^6$$

$= 1\,247\ (\times 10^6\ \text{mm}^4)$

– Unshored girder requirement

$$M_b = \left[9.0(P_{c1}+P_{c2})+4.6W_c\right]\frac{3.2}{9.2} - 3.0\,P_{c2} - (3.2)^2(0.8)/2$$

$$= \left[9.0(82.7+46.2)+4.6(7.2)\right]\frac{3.2}{9.2} - 3.0(46.2) - 4.10 = 272 \text{ kN·m}$$

$$M_t = 9.0\,(P_{L1}+P_{L2}+P_{p1}+P_{p2}+P_{OD1}+P_{OD2})\,\frac{3.2}{9.2} - 3.0(P_{L2}+P_{p2}+P_{OD2})$$

$$= 9.0(52.4+29.1+37.4+20.7+21.9+12.1)\,\frac{3.2}{9.2} - 3.0(29.1+20.7+12.1)$$

$= 358$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{272}{2.07} + \frac{358}{3.12} = 246 \text{ MPa} < 0.9\,F_y$$ ($S_x = 2.07 \times 10^6$ mm³, Table 4.4)

Therefore shoring is not required.

Girder's longitudinal shear requirement will be dealt with after steel deck layout is planned.

Spandrel Girder SG

– Live loads

$A = 6.0(11.5)/2 + 9.0(0.25) = 36.8\ \text{m}^2$
$RF_2 = 0.3 + \sqrt{9.8/36.8} = 0.82$
$P_L = 0.82(2.4)(3.0)(11.5)/2 = 34.0$ kN
$W_L = 0.82(2.4)(9.0)(0.25) = 4.43$ kN

– Dead loads

Assuming 0.7 kN/m due to steel girder and 3.5 kPa due to slab overhang

$W_c = 3.5(9.0)(0.25)+0.7(9.0) = 14.2$ kN
$P_c = 92.5/2 = 46.3$ kN
$W_w = 3.8(9.0) = 34.2$ kN
$W_{OD}+W_p = (1.2+0.7)(9.0)(0.25) = 4.28$ kN
$P_{OD}+P_p = (41.4+24.2)/2 = 32.8$ kN

– Factored total load, moment and shear

$P_f = 1.5(34.0)+1.25(46.3+32.8) = 150$ kN
$W_f = 1.5(4.43)+1.25(14.2+34.2+4.28) = 72.5$ kN
$M_f = 150(9.0)/3+72.5(9.0)/8 = 532$ kN·m
$V_f = 150+72.5/2 = 186$ kN

Try **W460×74**

$b = 190$ mm (Table 4.4)
$6(141) = 846$ mm (Governs slab projection to the interior floor from edge of beam flange)

$0.10(9\,000) = 900$ mm
Half clear spacing of girders > 846 mm (by inspection)
$b_1 = 846 + 190/2 + 250 = 1\,190$ mm (250 mm = exterior slab overhang from centre of beam; see Fig. 4.E8)

From Table 4.4, $M_{rc50\%} = 565$ kN·m (by interpolation) > 532 kN·m OK
$V_r = 733$ kN > 186 kN OK
$Q_{r100\%} = 789$ kN (by interpolation)
$2.5t = 2.5(14.5) = 36$ mm > 19 mm; edge distance $= 180$ mm > 141 mm.
Therefore, no reduction in shear stud capacity, i.e. $q_r = 74.3$ kN (Table 2.1)

Minimum number of studs $= 2\left(\frac{50\%}{100\%}\right)\left(\frac{789}{74.3}\right) = 10.6$ i.e **12 studs** per girder 57% connection)

– Construction stage

$A = 36.8\ \text{m}^2 > 27\ \text{m}^2$
$q_L = 1.4 - A/67.5 = 0.85$ kPa (see Table 3.2)

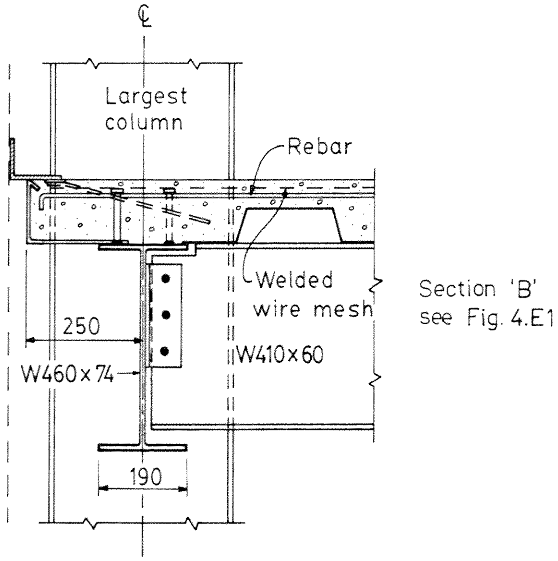


Figure 4.E8
Spandrel Girder 'SG' Cross-Section

$P_f = 1.5(0.85)(3.0)(11.5)/2 + 1.25(46.3) = 79.9$ kN
$W_f = 1.5(0.85)(9.0)(0.25) + 1.25(14.2) = 20.6$ kN
$M_f = 79.9(9.0)/3 + 20.6(9.0)/8 = 263$ kN·m

For $L' = 3\ 000$ mm, $M_r = 433$ kN·m > 263 kN·m (Table 4.4) OK

– Deflection estimate

From Table 4.4
$d = 457$ mm $I_x = 333 \times 10^6\ mm^4$
$I_t = 833 \times 10^6\ mm^4$ (by interpolation)
$S_t = 2.16 \times 10^6\ mm^3$ (by interpolation)

a) Camber requirement

$$\Delta_c = \frac{23(46.3)(9.0)^3}{648(200)(333)} \times 10^3 + \frac{5(14.2)(9.0)^3}{384(200)(333)} \times 10^3 = 20 \text{ mm}$$

Camber 20 mm at mid span.

b) Instantaneous deflection due to cladding mass

$$\Delta_w = \frac{5(34.2)(9.0)^3}{384(200)(833)} \times 10^3 \ (1.0 + 15\% \text{ for partial connection})$$

$= 2.2 \text{ mm} < 12 \text{ mm}$ OK

c) Shrinkage deflection

$$\bar{e} = 457 + 76 + \frac{65}{2} - \frac{833}{2.16} = 180 \text{ mm}$$

$$\Delta_{sh} = \frac{180(0.0002)(65)(1\ 190)(9.0)^2}{8(9.43)(833)} = 3.6 \text{ mm}$$

d) Deflection due to all superimposed load including long term effects

$$\Delta = \left[\frac{23(34.0+32.8)(9.0)^3\ 10^3}{648(200)\ I_e} + \frac{5(4.43+34.2+4.28)(9.0)^3\ 10^3}{384(200)\ I_e}\right](1.15) + 3.6$$

$= 21.1 \text{ mm} = L/426 < L/360$ OK

where $I_e = I_s + 0.85\ (p)^{0.25}\ (I_t - I_s)$

$= \left[333 + 0.85\ (0.57)^{0.25}\ (833 - 333)\right] 10^6\ mm^4$

$= 702\ (\times 10^6\ mm^4)$

– Unshored girder requirement

$M_b = 46.3(9.0)/3 + 14.2(9.0)/8 = 155$ kN·m
$M_t = (34.0 + 32.8)(9.0)/3 + (4.43+34.2+4.28)(9.0)/8 = 249$ kN·m

$$\frac{M_b}{S_x} + \frac{M_t}{S_t} = \frac{155}{1.46} + \frac{249}{2.16} \qquad (S_x = 1.46 \times 10^6\ mm^4, \text{ Table 4.4})$$

$= 221 \text{ MPa} < 0.9\ F_y$

Therefore shoring is not required.

W460×74 with 50% shear connection is satisfactory.

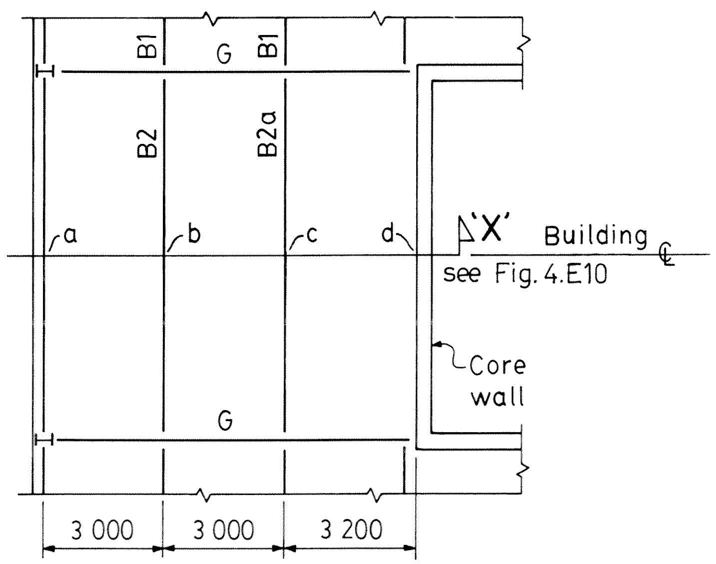


Figure 4.E9
Layout of Floor Bay at End of Service Core

Slab overhang

Select cold formed screed flash (bent plate) from data given in Fig. 4.18. For length of slab overhang, $x = 155$ mm and $t_o = 140$ mm, screed flash of 2.67 mm nominal thickness will limit tip deflection to less than 3 mm (see Fig. 4.18 for welding requirement). Negative reinforcing bars are required for cantilever action in the direction transverse to girder (see Fig. 4.E8).

Beam B2a (floor topography)

Since beam B2a is parallel and adjacent to a core wall, its mid-span deflection with respect to the floor elevation at the core wall, a rigid support, must be investigated. In this case deflection limitation may govern the design of beam B2a including the amount of camber required. The final elevation of the beam's mid-span (identified as 'c' in Fig. 4.E9) is a function of the simply supported beam deflection plus the girder deflection at the beam supports whereas the floor elevation at the core wall (identified as 'd' in Fig. 4.E10) remains virtually unchanged. The variation of support stiffnesses at 'c' and 'd' illustrates the fact that the true tributary area on beam B2a is less than the 'tributary' area assumed (based on half the deck span at either side of beam B2a); thus, actual total deflection at 'c' (beam B2a) should be less than that shown in Fig. 4.E10.

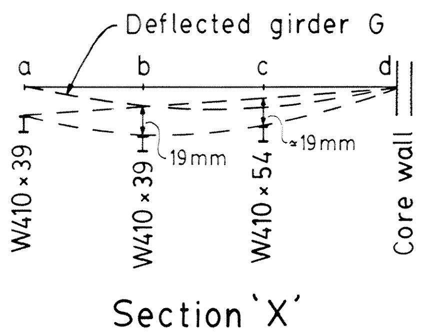


Figure 4.E10
Final Elevation of Deck-Slab with respect to Support Elevation at Core Wall

Steel deck layout and shear stud distribution (wide-rib deck)

Steel deck layout is an important part of hollow-composite construction. A proper layout reduces wastage, cutting and erection time. The actual shear stud quantity also depends on steel deck layout. The minimum number of shear studs required for each composite member that was determined previously was computed based on the assumption that no more than one stud per deck flute is allowed. Quite often some flutes must accommodate more than one shear stud due to a limited number of wide flutes available per beam span. A steel deck layout and shear stud distribution plan is shown in Figs. 4.E11 and 4.E12. This layout allows one stud per flute for all beams except for **beam B2a**. In the case of beam B2a,

$$Q_{r50\%} = \frac{1\,490}{2} = 745 \text{ kN}$$

$$\text{Minimum number of single studs} = 2\left(\frac{745}{74.3}\right) = 20.1$$

i.e. 11 single studs per half span, but number of flutes available per half span = 10. Try 9 single groups plus 1 double stud group per half span.

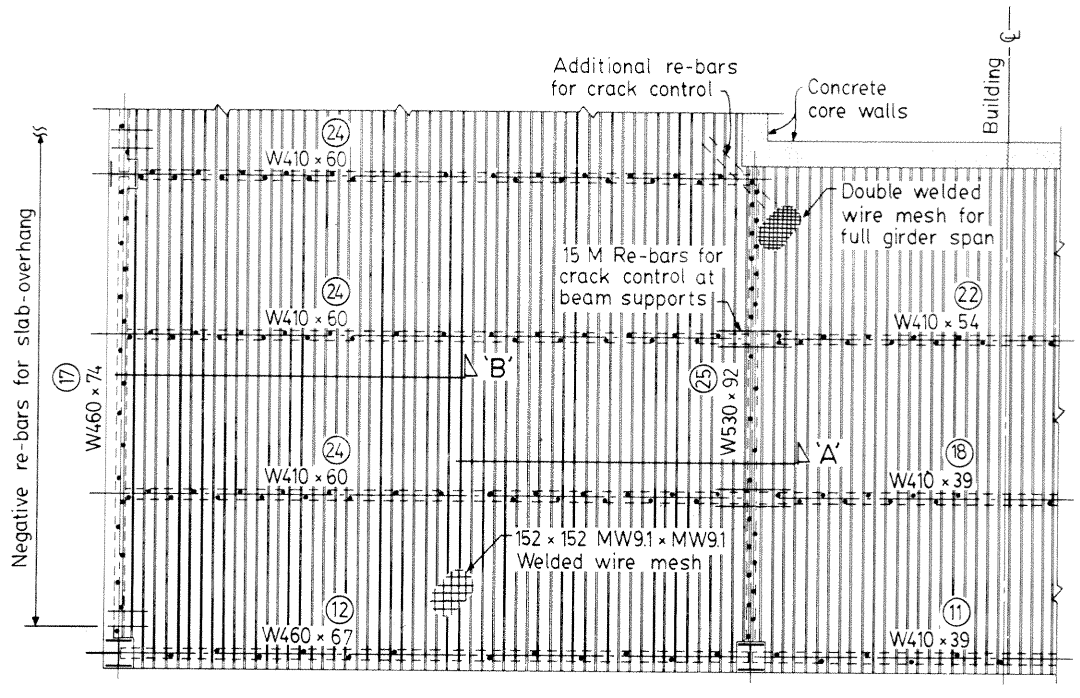


Figure 4.E11
Steel Deck and Shear Stud Layout (Non-Cellular Configuration)

$$q_{rN} = \frac{0.85}{\sqrt{N}}\left(\frac{H-t_d}{t_d}\right)\left(\frac{W_{rib}}{t_d}\right) q_r \qquad \text{(Eq. 2.10)}$$

$$q_{r2} = \frac{0.85}{\sqrt{2}}\left(\frac{120-76}{76}\right)\left(\frac{180}{76}\right)(74.3) = 61.2 \text{ kN per stud}$$

$$Q_r = \Sigma\, q_r = 9(74.3) + (61.2)(2)$$
$$= 791 \text{ kN} > 745 \text{ kN} \qquad \text{OK}$$

Alternatively B2a may be designed as non-composite member for strength consideration.
$M_r = 283$ kN·m $> M_f = 270$ kN·m
For deflection, no less than 25% connection is used to create composite action (use 12 studs per beam)

Vertical separation and mechanical ties

Shear stud distribution and weld pattern along beam B1 are illustrated in Fig. 4.E12. The deck is welded to each steel member before shear studs are installed. These arc spot welds also serve to prevent vertical separation between the deck and the steel shape after concrete is cast while the composite steel deck holds the concrete by means of its embossment. The weld spacing must also satisfy the fire-resistance rating requirements of the floor assembly.

When a deck flute is not aligned over a girder, the deck is cut out to admit shear studs (see details in Figures 4.E8 and 4.E13). In the absence of steel deck additional shear studs are provided near the girder mid-span to serve as mechanical ties. Average spacing of mechanical ties should not exceed 600 mm in accordance with Clause 17.3.7 of S16.1.

Owing to the presence of either a stiffener (embossment) or a side-lap joint in the middle of each flute, each stud must be installed asymmetrically to the flute and should be placed on the side closer to the nearest beam support whenever possible (see Section 2.4).

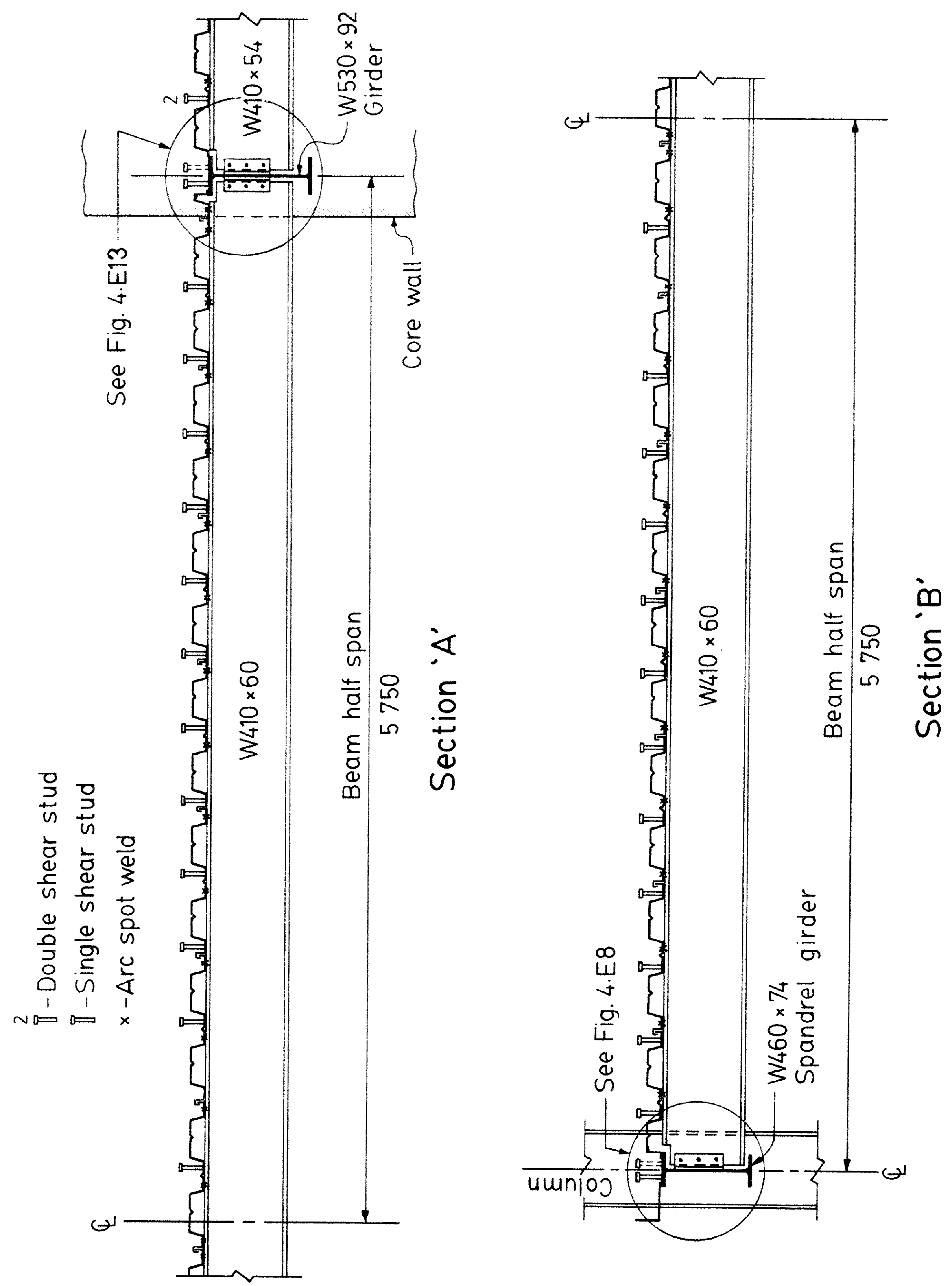


Figure 4.E12
Detailed Sections of Deck and Shear Stud Layout (Non-Cellular Configuration)

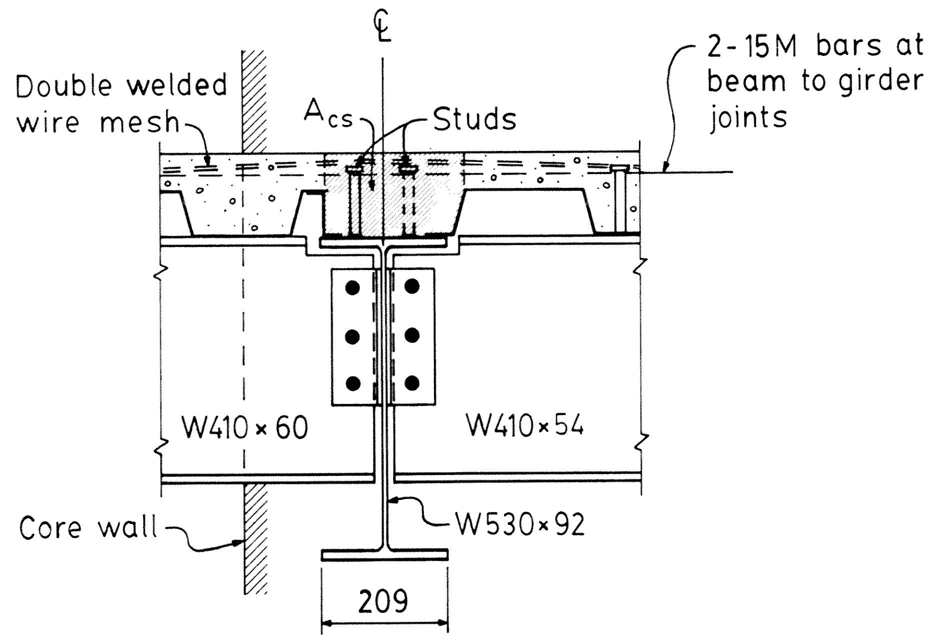


Figure 4.E13
Interior Girder 'G' Cross-Section

Longitudinal shear in composite girders

a) Interior girder G: Check if **two** layers of 152 × 152 MW9.1 × MW9.1 welded wire mesh satisfy the design criterion outlined in Section 4.9. Since shear studs are more closely spaced in shear span 2 (l_{sh} = 3 200 mm), shear span 2 governs.

$Q_r = 1\ 530/2 = 765$ kN

$\rho = \dfrac{(9.0)}{152(65)} = 0.00091$ (credit 1 layer of mesh for longitudinal shear resistance)

v_u	$= 0.8\ \rho\ f_y + 2.76$	(Eq. 4.20)
	$= 0.8\ (0.00091)(400) + 2.76$	(f_y = 400 MPa)
	$= 3.05$ MPa $< 0.3\ f'_c$	($0.3\ f'_c$ = 6.0 MPa)
V_u	$= 2\ l_{sh}\ t_c\ v_u$	(Eq. 4.19)
	$= 2(3\ 200)(65)(3.05) \times 10^{-3} = 1\ 270$ kN	
A_{cs}	$= 209(140) = 29\ 300\ \text{mm}^2$	(see Fig. 4.E15)

$\phi_v(V_u + 0.85\ f'_c\ A_{cs}) = 0.60[1\ 270 + 0.85(0.020)(29\ 300)]$ (Eq. 4.23)
$= 1\ 060$ kN > 765 kN OK

b) Spandrel girder SG: Resistance to longitudinal shear is found adequate.

Cellular Floor

Figures 4.E14 and 15 illustrate a layout of a cellular floor system. Typical cross-sectional and pictorial views of cellular steel deck and trench header duct are also shown in Fig. 1.4. The presence of these underfloor distribution features have affected the structural design in several major areas:

a) The presence of cellular decks may reduce the number of wide-ribs per beam, depending on the deck manufacturer, and hence may result in double stud application on some interior beams.

b) The trench header duct truncates the effective slab width of the adjacent beam. As a result, a heavier steel beam section may become necessary, but requiring fewer shear studs.

c) The trench duct also crosses over the main girder and reduces the girder's composite shear span. This necessitates closer spacing of shear studs and additional longitudinal shear reinforcement. The proximity of the trench duct to the end of the girder normally permits the girder to resist bending and shear forces as a non-composite section.

d) Since the trench duct displaces concrete over the steel deck, the steel deck alone must resist the total bending moment and shear at the trench duct location as a non-composite section. A thicker deck may have to be provided locally (see Figs. 4.E14 and 15).

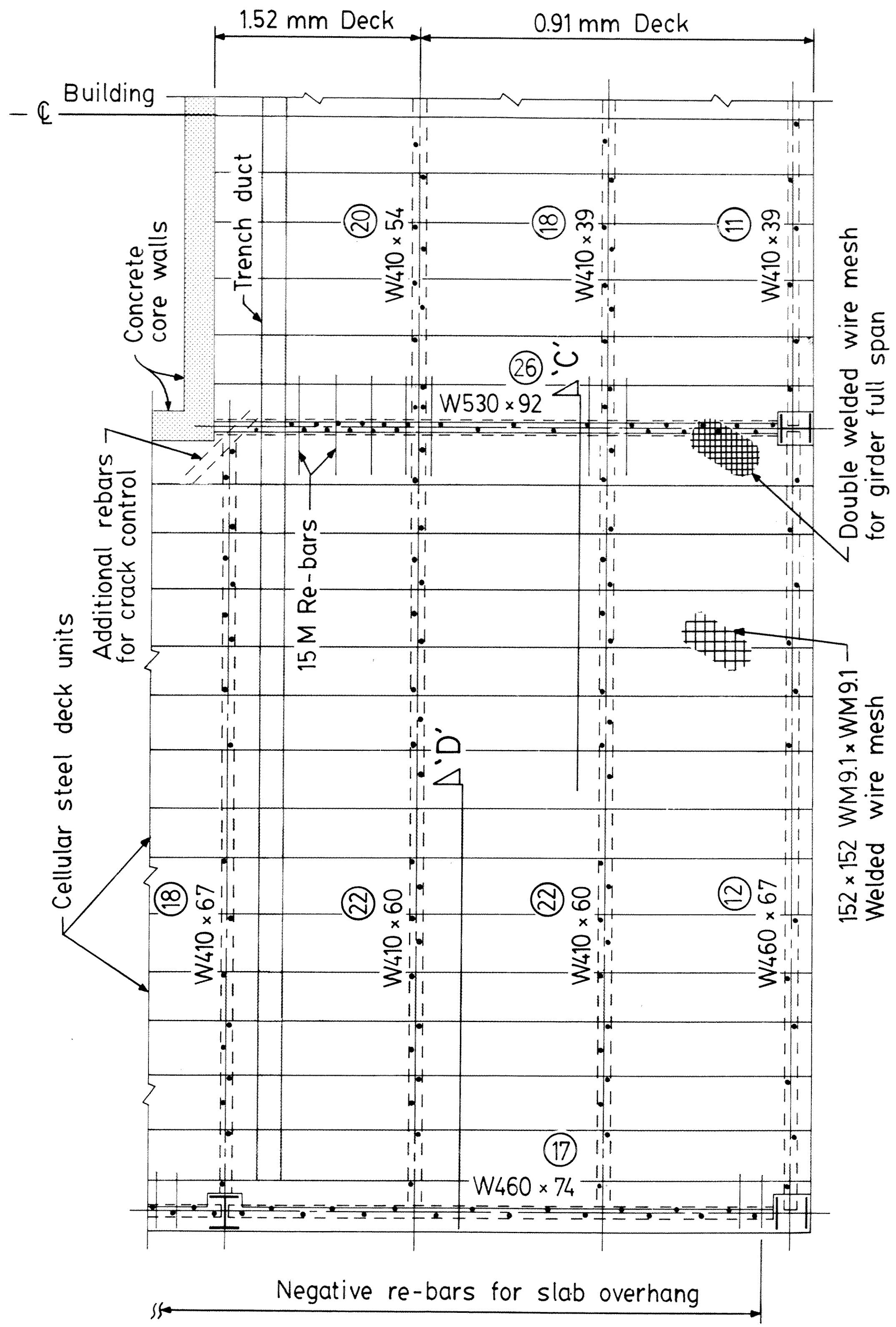


Figure 4.E14
Steel Deck and Shear Stud Layout
(Cellular Configuration)

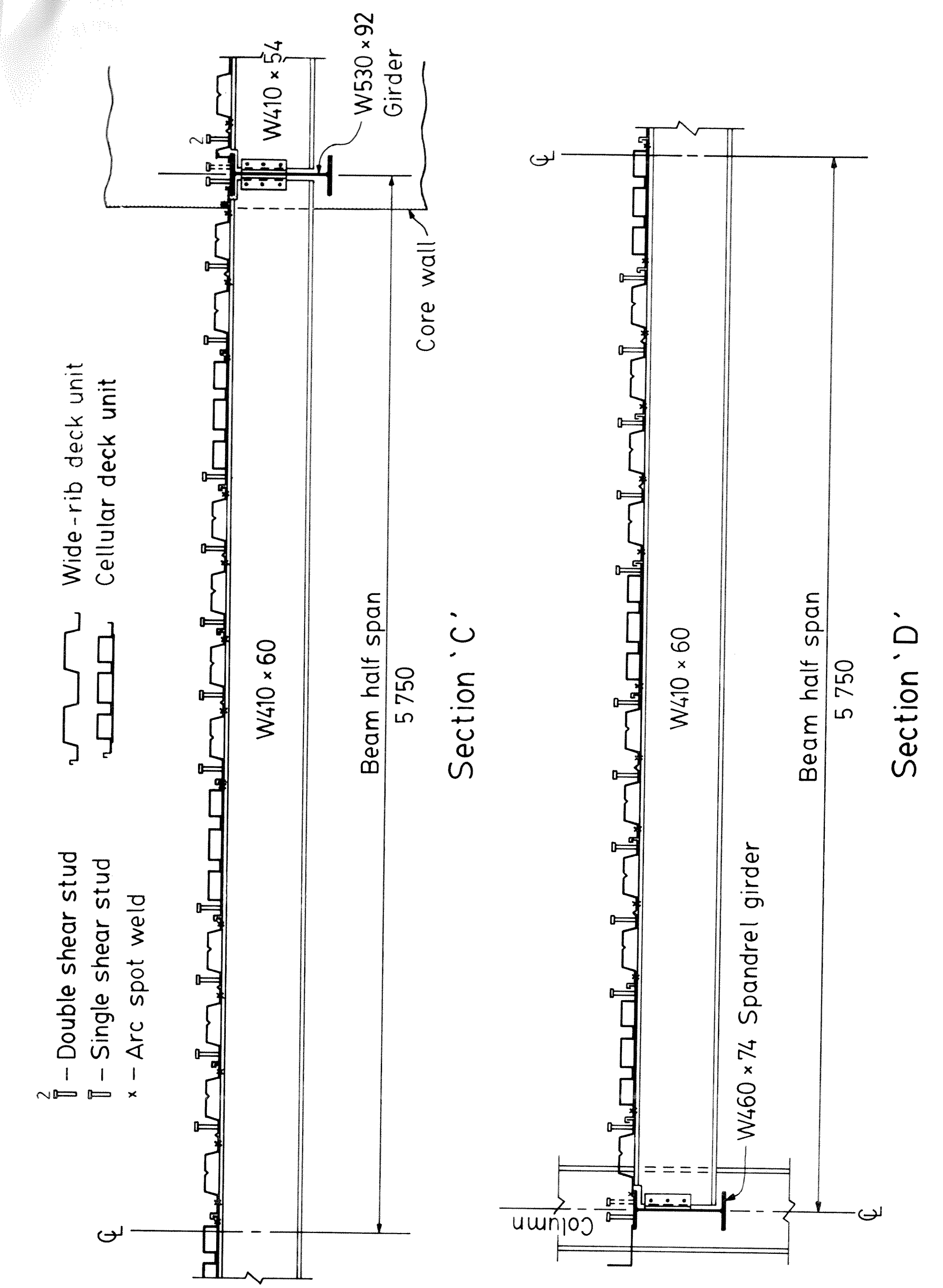


Figure 4.E15
Detailed Sections of Deck and Shear Stud Layout (Cellular Configuration)

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