23 COMPOSITE FLOORS - I

1.0 INTRODUCTION

Traditional steel - concrete composite floors consist of rolled or built-up structural steel beams and cast in-situ concrete floors connected together using shear connectors in such a manner that they would act monolithically (Fig.1). The principal merit of steel-concrete composite construction lies in the utilisation of the compressive strength of concrete slabs in conjunction with steel beams, in order to enhance the strength and stiffness of the steel girder.

More recently, composite floors using profiled sheet decking have become very popular in the West for high rise office buildings. Composite deck slabs are particularly competitive where the concrete floor has to be completed quickly and where medium level of fire protection to steel work is sufficient. However, composite slabs with profiled decking are unsuitable when there is heavy concentrated loading or dynamic loading in structures such as bridges. The alternative composite floor in such cases consists of reinforced or pre-stressed slab over steel beams connected together to act monolithically.
A typical composite floor system using profiled sheets is shown in Fig.2. There is presently no Indian standard covering the design of composite floor systems using profiled sheeting.

Designing a reinforced concrete slab or pre-stressed concrete slab in composite construction is not different from any conventional R.C. or pre-stressed structures; hence, this is not discussed any further here. In this chapter, concrete floors using profiled decks are treated in depth. The structural behaviour of these floors is similar to a reinforced concrete slab, with the steel sheeting acting as the tension reinforcement. The main structural and other benefits of using composite floors with profiled steel decking are:

- Savings in steel weight are typically *30%* to *50%* over non-composite construction
- Greater stiffness of composite beams results in shallower depths for the same span. Hence lower storey heights are adequate resulting in savings in cladding costs, reduction in wind loading and savings in foundation costs.
- Faster rate of construction.

The steel decking performs a number of roles, such as:
- It supports loads during construction and acts as a working platform
- It develops adequate composite action with concrete to resist the imposed loading
- It transfers in-plane loading by diaphragm action to vertical bracing or shear walls
- It stabilises the compression flanges of the beams against lateral buckling, until concrete hardens.
- It reduces the volume of concrete in tension zone
- It distributes shrinkage strains, thus preventing serious cracking of concrete.

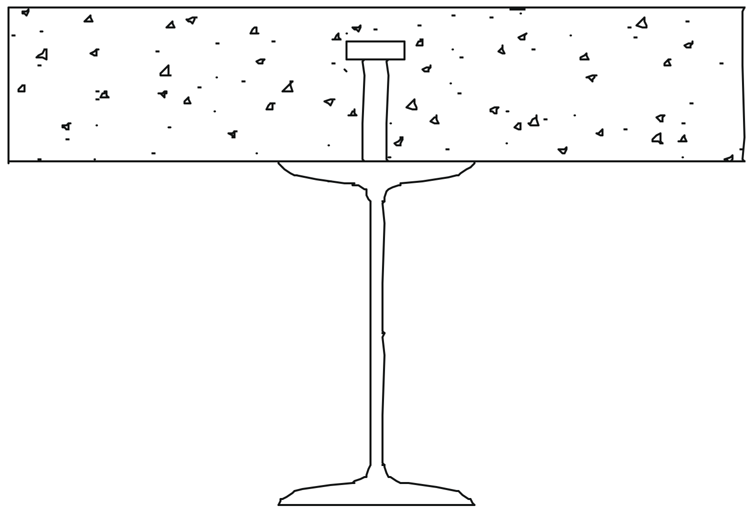


Fig. 1 Steel beam bonded to concrete slab with shear connectors

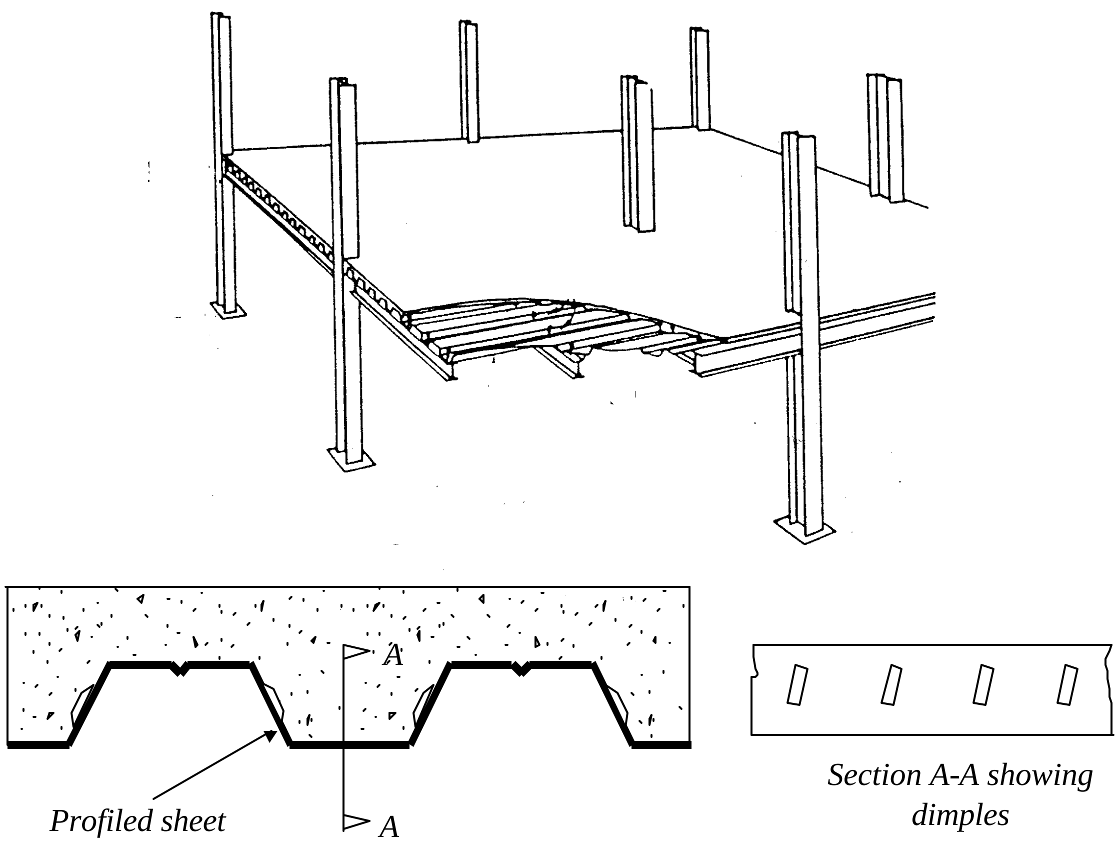


Fig. 2 Composite floor system using profiled sheets

Care has to be taken in the construction of composite floors with profiled decking to prevent excessive *'ponding'*, especially in the case of long spans. The profiled sheet deflects considerably requiring additional concrete at the centre that may add to the concreting cost. Thus, longer spans will require propping to eliminate substantial deflection or need significant quantities of concrete. Fig. 3 shows ponding of the profiled deck.

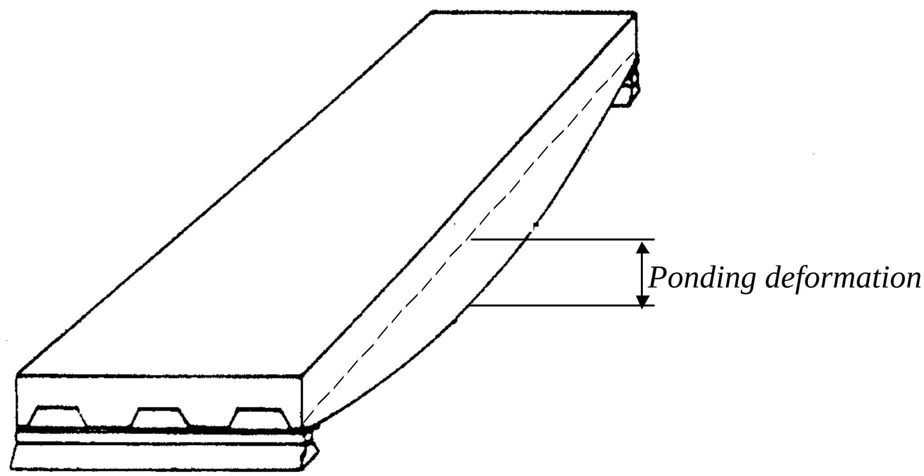


Fig. 3 Ponding in profiled decking, due to the weight of concrete

2.0 THE STRUCTURAL ELEMENTS

Composite floors with profiled decking consists of the following structural elements along with in-situ concrete and steel beams:

- Profiled decking
- Shear connectors
- Reinforcement for shrinkage and temperature stresses

Connections between the structural steel elements are generally designed as “simple” i.e. not moment resisting. Stud shear connectors are welded through the sheeting on to the top flange of the beam. Insulation requirements for fire usually control the slab thickness above the profile. Thickness values between *65* and *120 mm* are sufficient to give a fire rating of up to *2* hours. Lightweight concrete is popular, despite its slightly higher initial cost, because of the consequent reduction in weight and enhanced fire-insulation properties.

2.1 Profiled sheet decking

The steel deck is normally rolled into the desired profile from *0.9 mm* to *1.5 mm* galvanised coil. It is profiled such that the profile heights are usually in the range of *38-75 mm* and the pitch of corrugations is between *150 mm* and *350 mm*. Generally, spans of the order of *2.5 m* to *3.5 m* between the beams are chosen and the beams are designed to span between *6 m* to *12 m*. There are two well-known generic types of profiles.

- Dovetail profile
- Trapezoidal profile with web indentations

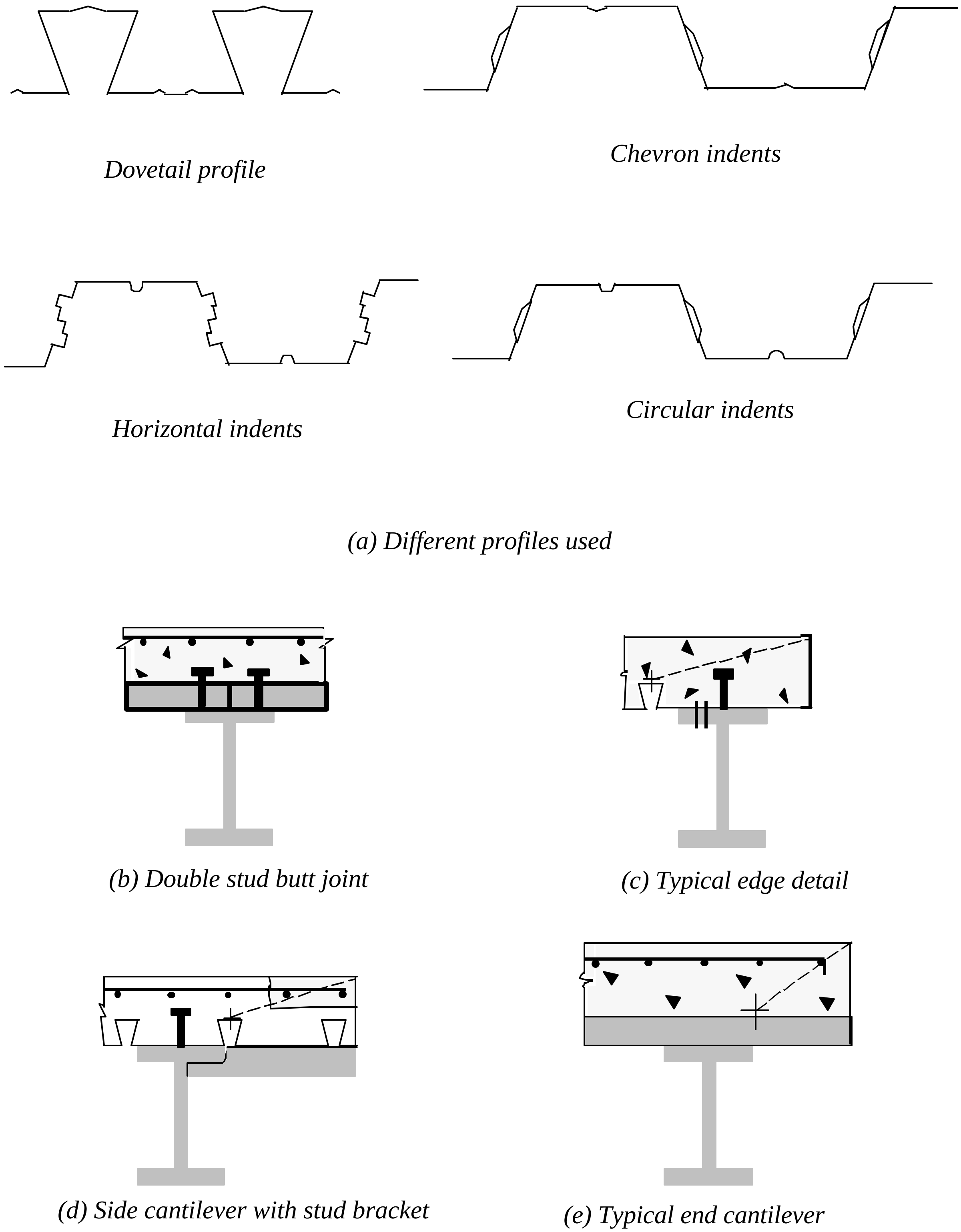


Fig. 4 Deck profiles and typical details

Their shapes are generally chosen as a compromise between enhancing the bond at the steel-concrete interface and providing stability while supporting wet concrete and other construction loads. Indentations and protrusions into the rib mobilise the bearing resistance in addition to adhesion and also provide the shear transfer in composite slabs. The shear resistance capacity will depend on the parameters like re-entrant angle, lug spacing, lug width etc. Fig. 4 shows typical examples of deck profiles and the details of their attachments to the steel beams.

2.1.1 Profiled sheeting as permanent form work

Construction stage: During construction, the profiled steel deck acts alone to carry the weight of wet concrete, self weight, workmen and equipments. It must be strong enough to carry this load and stiff enough to be serviceable under the weight of wet concrete only. In addition to structural adequacy, the finished slab must be capable of satisfying the requirements of fire resistance.

Composite Beam Stage: The composite beam formed by employing the profiled steel sheeting is different from the one with a normal solid slab, as the profiling would influence its strength and stiffness. This is termed *'composite beam stage'*. In this case, the profiled deck, which is fixed transverse to the beam, results in voids within the depth of the associated slab. Thus, the area of concrete used in calculating the section properties can only be that depth of slab above the top flange of the profile. In addition, any stud connector welded through the sheeting must lie within the area of concrete in the trough of the profiling. Consequently, if the trough is narrow, a reduction in strength must be made because of the reduction in area of constraining concrete. In current design methods, the steel sheeting is ignored when calculating shear resistance; this is probably too conservative.

Composite Slab Stage: The structural behaviour of the composite slab is similar to that of a reinforced concrete beam with no shear reinforcement. The steel sheeting provides adequate tensile capacity in order to act with the concrete in bending. However, the shear between the steel and concrete must be carried by friction and bond between the two materials. The mechanical keying action of the embossments is of great importance. This is especially so in open trapezoidal profiles, where the embossments must also provide resistance to vertical separation. The predominant failure mode is one of shear bond rupture that results in slip between the concrete and steel.

2.1.2 Design method

As there is no Indian standard covering profiled decking, we refer to Eurocode 4 (EC4) for guidance. The design method defined in EC4 requires that the slab be checked firstly for bending capacity, assuming full bond between concrete and steel, secondly for shear bond capacity and, finally, for vertical shear. The analysis of the bending capacity of the slab may be carried out as though the slab was of reinforced concrete with the steel deck setting as reinforcement. However, no satisfactory analytical method has been developed so far for estimating the value of shear bond capacity. Depending upon the test data available, the loads at the construction stage often govern the allowable span rather than at the composite slab stage.

2.2 Shear connectors

Shear connectors are steel elements such as studs, bars, spiral or any other similar devices welded to the top flange of the steel section and intended to transmit the horizontal shear between the steel section and the cast in-situ concrete and also to prevent vertical separation at the interface. This topic is discussed in detail in the chapter titled composite beams - I

2.3 Reinforcement for shrinkage and temperature stresses

In buildings, temperature difference in the slabs is negligible; thus there is no need to provide reinforcement to account for temperature stresses. The effect of shrinkage is considered and the total shrinkage strain for design may be taken as *0.003* in the absence of test data.

3.0 BENDING RESISTANCE OF COMPOSITE SLAB

The structural properties of profiled sheet along with reinforcement provided and concrete with a positive type of interlock between concrete and steel deck is the basis of a composite floor. Some loss of interaction and hence slip may occur between concrete - steel interface. Such a case is known as *'Partial interaction'*. Failure in such cases occurs due to a combination of flexure and shear.

The width of the slab 'b' shown in Fig. 5(a) is one typical wavelength of profiled sheeting. But, for calculation purpose 'b' is taken as *1.0 m*. The overall thickness is h_t and the depth of concrete above main flat surface h_c. Normally, h_t is not less than *80 mm* and h_c is not less than *40 mm* from sound and fire insulation considerations.

The neutral axis normally lies in the concrete in case of full shear connection; but in regions of partial shear connection, the neutral axis may be within the steel section. The local buckling of steel sections should then be considered. For sheeting in tension, the width of embossments should be neglected. Therefore, the effective area 'A_p' per meter and height of centre of area above bottom 'e' are usually based on tests. The plastic neutral axis e_p is generally larger than e.

The simple plastic theory of flexure is used for analysis of these floors for checking the design at Limit State of collapse load. Eurocode assumes the equivalent ultimate stress of concrete in compression as $0.85(f_{ck})_{cy}/g_c$ where $(f_{ck})_{cy}$ is the characteristic cylinder compression strength of concrete. However, IS 456: 1978 uses an average stress of $0.36 (f_{ck})_{cu}$ accommodating the value of g_c and considering $(f_{ck})_{cu}$ as characteristic ***cube strength of concrete***. IS code is on the conservative side. Comparison of two values is shown in the appendix. Note that in this chapter f_{ck} refers to cube strength of concrete

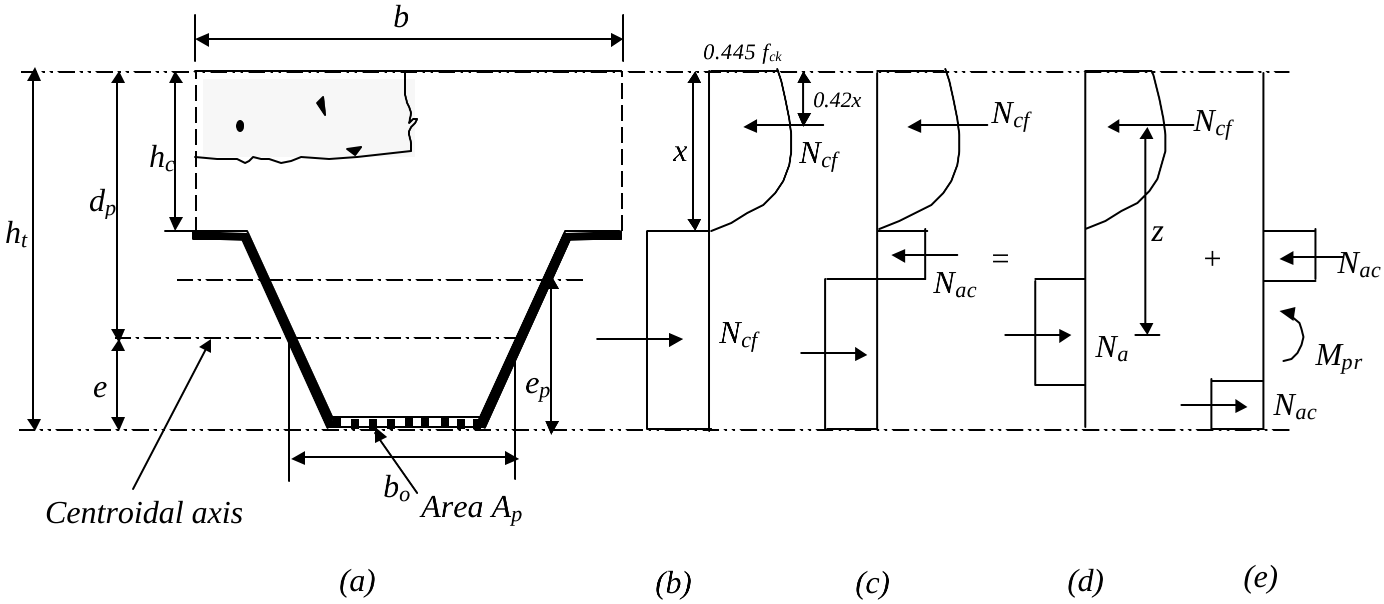


Fig. 5 Resistance of composite slab to sagging bending

3.1 Neutral axis above the sheeting [Fig. 5(b)]

Full shear connection is assumed. Hence, compressive force N_{cf} in concrete is equal to steel yield force N_{pa}.

$$N_{cf} = N_{pa} = \frac{A_p \; f_{yp}}{g_{ap}} \qquad (1)$$

$$N_{cf} = 0.36 \, f_{ck}.b.x$$

where A_p = Effective area per meter width

f_{yp} = Yield strength of steel

g_{ap} = Partial safety factor (1.15)

The neutral axis depth x is given by

$$x = \frac{N_{cf}}{b \; (0.36 \; f_{ck})} \qquad (2)$$

This is valid when $x \leq h_c$, i.e. when the neutral axis lies above steel decking.

$$M_{p.Rd} = N_{cf} (d_p - 0.42 \, x) \qquad (3)$$

Note that centroid of concrete force lies at $0.42 \, x$ from free concrete surface.

$M_{p.Rd}$ is the design resistance to sagging bending moment.

3.2 Neutral axis within sheeting and full shear connection [Fig. 5(c)]

$$N_{cf} = \left(bh_c \times 0.36\ f_{ck}\right) \quad (4)$$

The compression of concrete within rib is neglected. The force N_{cf} is less than N_{pa}. The tensile force in sheeting is split into N_a (equal to compressive force N_{cf}) plus N_{ac}.

$$N_a = N_{cf}$$

and the remaining force N_{ac} such that the total tensile force is $N_{ac} + N_a$. The equal and opposite force N_{ac} provide resisting moment M_{pr}. Note this M_{pr} will be less than M_{pa}, the flexural capacity of steel sheeting. The relationship between $\frac{M_{pr}}{M_{pa}}$ and $\frac{N_{cf}}{N_{pa}}$ is shown in Fig. 6(*a*) in the dotted line. For design this can be approximated by line ADC that can be expressed as

$$M_{pr} = 1.25 M_{pa}\left[1 - \frac{N_{cf}}{N_{pa}}\right] \le M_{pa} \quad (5)$$

The moment of resistance is given by

$$M_{p.Rd} = (N_{cf})z + M_{pr} \quad (6)$$

Sum of resistance is shown in Fig. 5(*d*) and Fig. 5(*e*), which is equal to the resistance shown in Fig. 5(*c*).

The lever arm z can be found by examining the two extreme cases. For case (i) where $N_{cf} = N_{pa}$ or $N_{cf}/N_{pa} = 1.0$, $N_{ac} = 0$ and hence $M_{pr} = 0$.

$$M_{p.Rd} = N_{pa}\,(d_p - 0.42\,h_c) \quad (7)$$

Hence, $z = d_p - 0.42h_c = h_t - e - 0.42\,h_c$ (8)

This is indicated by point F in Fig. 6(*b*).

For case (ii), on the other hand

N_{cf} @ $0;\ N_a = 0.$

$M_{pr} = M_{pa}$. The neutral axis is at a height e_p above the bottom. Then

$$z = h_t - 0.42h_c - e_p \quad (9)$$

This is represented by point E. Thus the equation to the line EF is

$$z = h_t - 0.42\, h_c - e_p \frac{(e_p - e)N_{cf}}{N_{pa}} \tag{10}$$

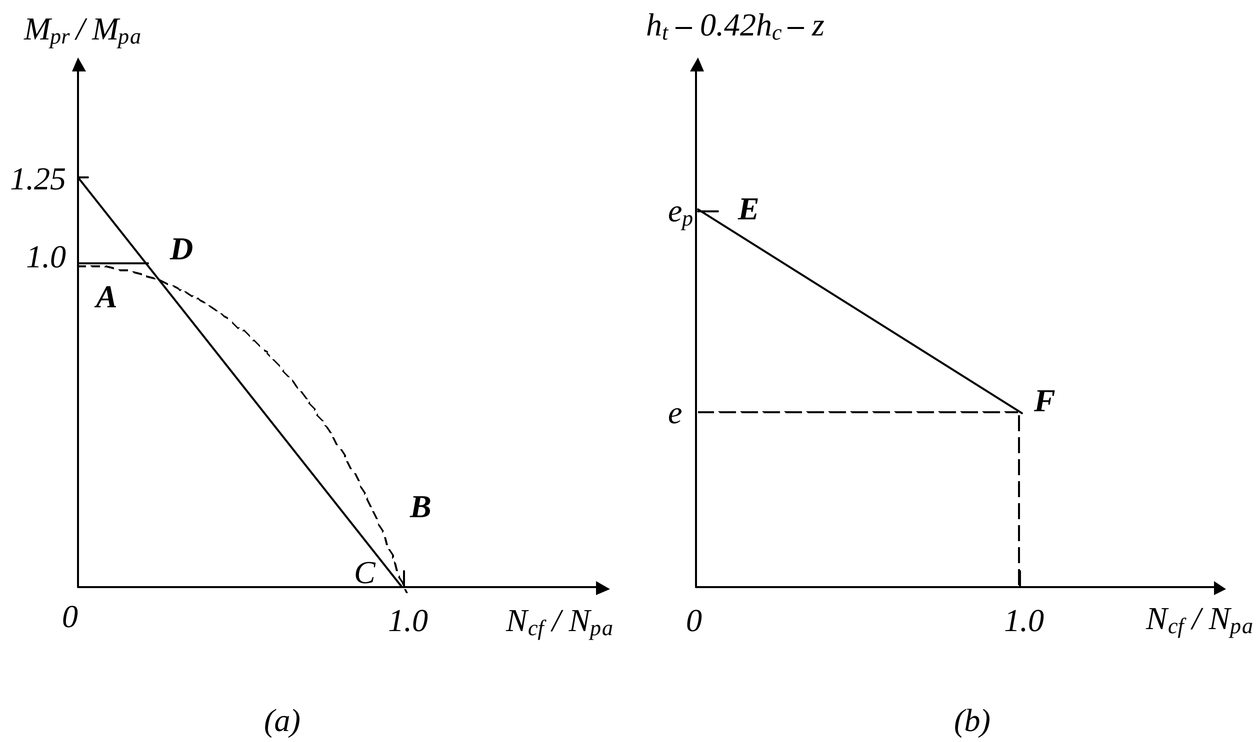


Fig. 6 Resistant moment of profiles

3.3 Partial shear connection ($N_c < N_{cf}$)

In this case, the compressive force in the concrete N_c is less than N_{cf} and depends on the strength of shear connection and the stress blocks are as shown in Fig. 5(*b*) for the slab (with N_c in place of N_{cf}) and Fig. 5(c) for sheeting.

The depth of stress block is,

$$x = \frac{N_c}{b\,(0.36\, f_{ck})} \le h_c \tag{11}$$

In this case, equations 5, 6 and 10 get modified by substituting

$N_c \quad = \quad N_{cf}$
$N_{cf} \quad = \quad N_{pa}$
$x \quad = \quad h_c$

Thus,

$$z = h_t - 0.42x - e_p \frac{(e_p - e)N_c}{N_{cf}} \tag{12}$$

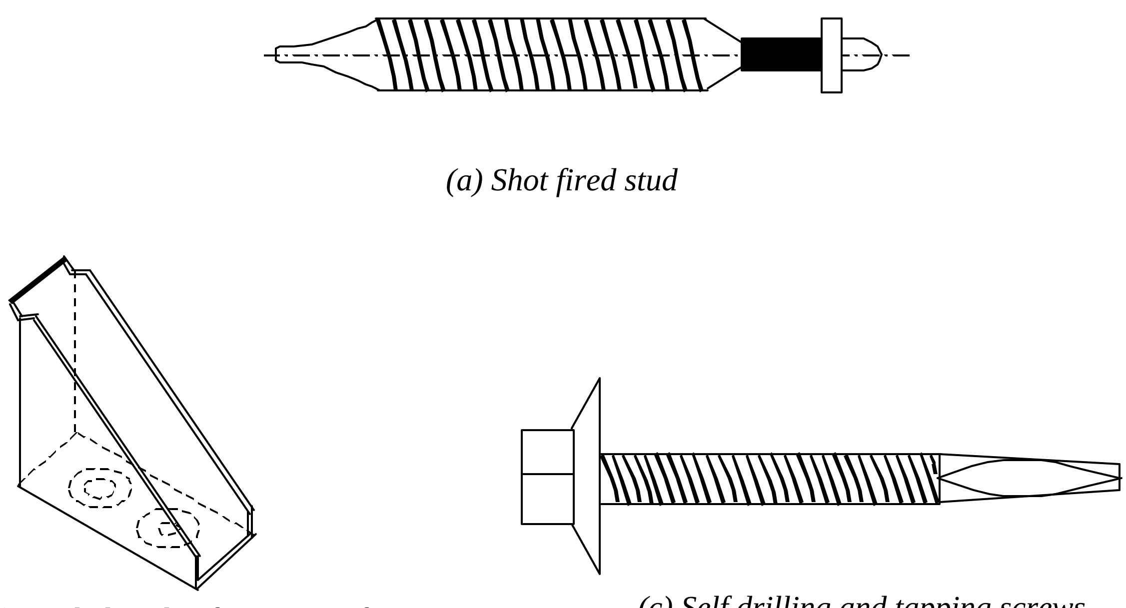
$$M_{pr} = 1.25M_{pa}\left[1 - \frac{N_c}{N_{cf}}\right] \le M_{pa} \tag{13}$$

$$M_{p.Rd} = N_c z + M_{pr} \tag{14}$$

4.0 SHEAR RESISTANCE OF COMPOSITE SLAB

The shear resistance of composite slab largely depends on connection between profiled deck and concrete. The following three types of mechanisms are mobilised:

(i) Natural bond between concrete and steel due to adhesion
(ii) Mechanical interlock provided by dimples on sheet and shear connectors
(iii) Provision of end anchorage by shot fired pins or by welding studs (Fig. 7) when sheeting is made to rest on steel beams.



(a) Shot fired stud

(b) Angle bracket for two pin fixing

(c) Self drilling and tapping screws

Fig. 7 Connector details

Natural bond is difficult to quantify and unreliable, unless separation at the interface between the sheeting and concrete is prevented. Dimples or ribs are incorporated in the sheets to ensure satisfactory mechanical interlock. These are effective only if the embossments are sufficiently deep. Very strict control during manufacture is needed to ensure that the depths of embossments are consistently maintained at an acceptable level. End anchorage is provided by means of shot-fired pins, when the ends of a sheet rests on a steel beam, or by welding studs through the sheeting to the steel flange.

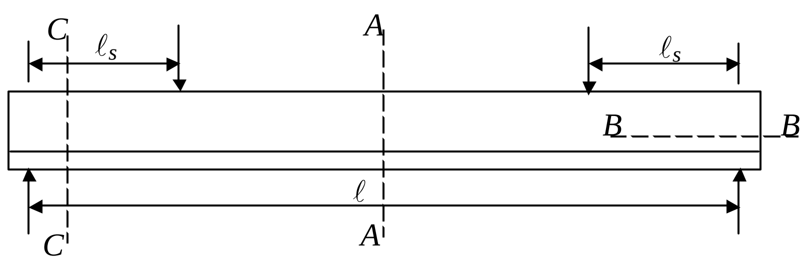
Quite obviously the longitudinal shear resistance is provided by the combined effect of frictional interlock, mechanical interlock and end anchorage. No mathematical model could be employed to evaluate these and the effectiveness of the shear connection is studied by means of load tests on simply supported composite slabs as described in the next section.

4.1 Resistance to longitudinal shear

If the shear connection is partial, slip occurs between decking and concrete. The effectiveness of the shear connection is tested using an *m-k* shear bond test. The test is described below. The failure of the beam is initiated by one of the following three modes [See Fig. 8].

(i) Flexure
(ii) Shear at support
(iii) Shear bond mode

Note that ℓ_s is the shear span and ℓ is effective span.



A-A Flexure mode
B-B Shear bond mode
C-C Vertical Shear

Fig. 8 Failure modes and critical sections

4.1.1 Evaluation of shear capacity of profiled sheets using m – k test

Specifications for tests to evaluate shear capacity of profiled sheets are given in Eurocode 4 and BS 5950: Part 4. The influence of bond is minimised in the standard test, by the application of several thousand cycles of repeated loading up to *1.5* times the service load, before loading to failure. The length of each shear span (ℓ_s) is usually (*span /4*) for uniformly distributed loading. The span is typically 3 metres. The evaluation of shear span is illustrated below.

Illustration of evaluation of shear spans:

For uniformly distributed load on a span ℓ, the length ℓ_s is taken as $\ell/4$. The principle that is used when calculating ℓ_s for other loading is now illustrated by an example.

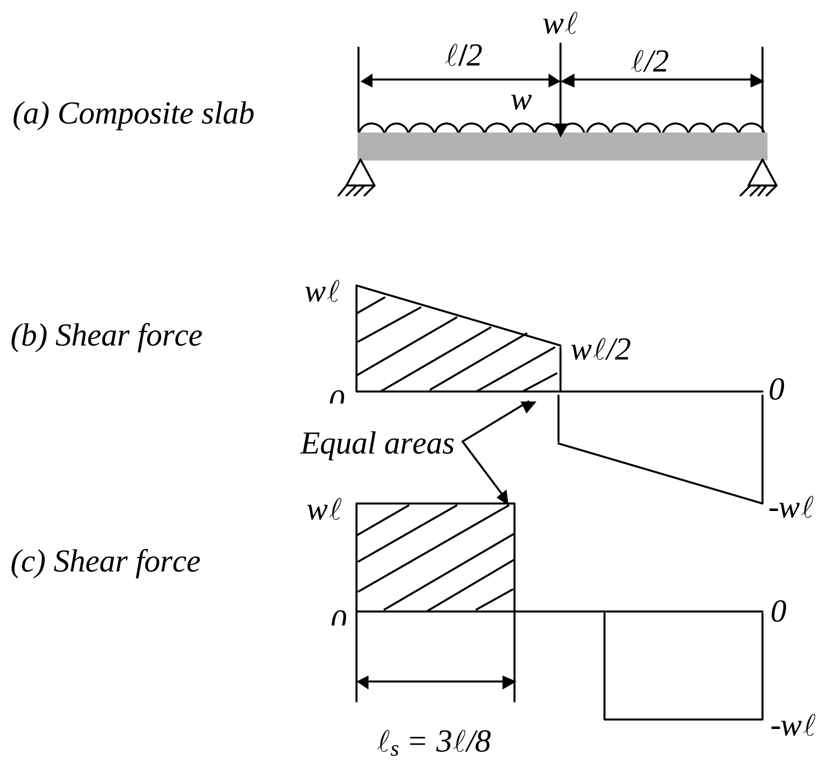


Fig. 9 Calculation of ℓ_s for composite slab

The composite slab shown in Fig. 9(a) has a distributed load w per unit length and a centre point load $w\ell$, so the shear force diagram is as shown in Fig. 9(b). A new shear force diagram is constructed for a span with two point loads only, and the same two end reactions, such that the areas of the positive and negative parts of the diagram equal to those of the original diagram. This is shown in Fig. 9(c), in which each shaded area is $3w\ell^2/8$. The positions of the point loads define the lengths of the shear spans. Here, each one is $3\ell/8$.

The expected mode of failure in a test depends on the ratio of (ℓ_s) to the effective depth (d_p) of the slab. The test specimen of breadth *"b"* should include four or five complete wavelengths of sheeting. The total cross sectional area of the sheeting is A_p. Fig. 10 shows arrangement for m - k test.

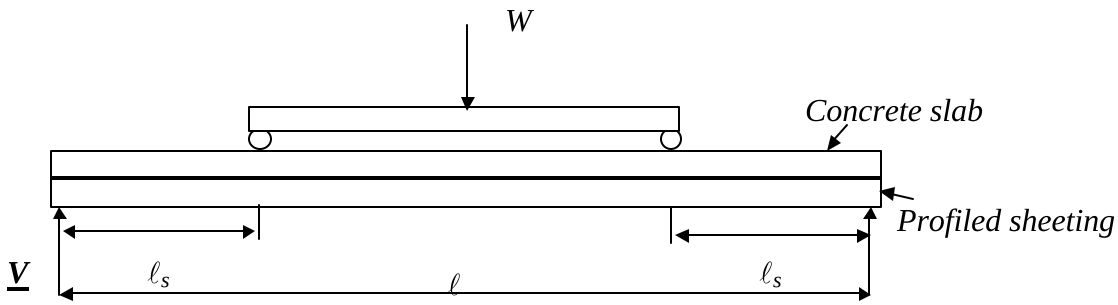


Fig. 10 A typical test arrangement

In typical Eurocode 4 tests, the results are plotted on a diagram with axis V/bd_p and $A_p/b\ell_s$ (See Fig. 11)

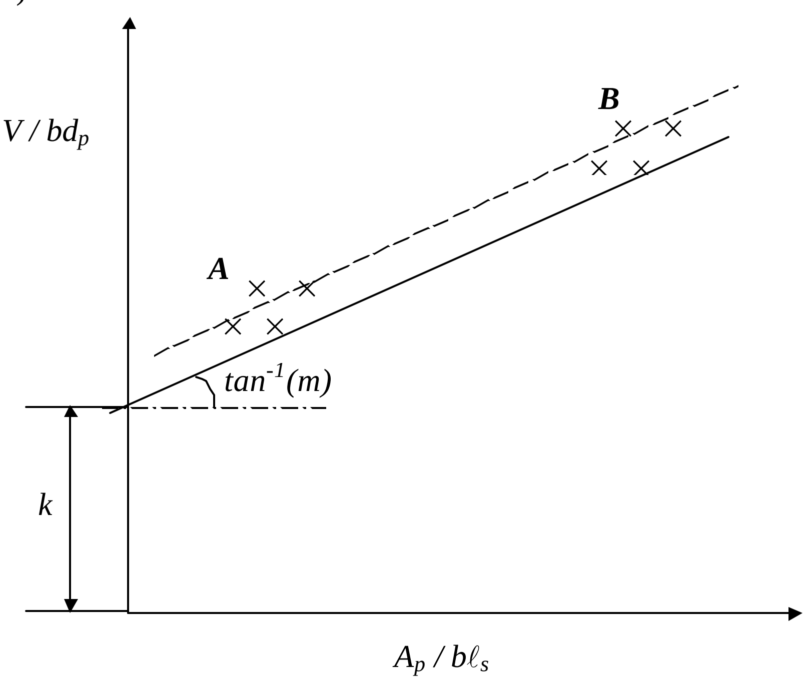


Fig. 11 m-k test details

The empirical constants m and k are determined from prototype slab tests to failure and are calculated from the slope and intercept of a regression line of Fig. 11. Tests are carried out under two or four point loads to stimulate a uniform load. The regression line is to be lowered by *15%* if less than eight slab tests are performed over a range of spans. Physically *"m"* is a broad measure of the mechanical interlock and k represents the friction load. The conceptual background to these tests is described below. At high values of ℓ_s/d_p, flexural failure occurs. The maximum bending moment (M_u) is given by

$$M_u = V.\ell_s \tag{15}$$

where, V is the maximum vertical shear.

Flexural failure is modelled by simple plastic theory with all the steel at its yield stress (f_{yp}). Concrete is stressed to average compressive stress of $0.36\ (f_{ck})_{cu}$, where $(f_{ck})_{cu}$ is the cube strength of concrete. The lever arm is approximately equal to d_p.

$$M_u \text{ is proportional to } A_p f_{yp} d_p \tag{16}$$

From equation (*15*),

$$\frac{V}{bd_p} = \frac{M_u}{bd_p \cdot \ell_s} \text{ is proportional to } \frac{A_p \cdot f_{yp}}{b\ell_s} \tag{17}$$

In tests f_{yp} is not varied. Hence flexural failure should show as a line through the origin.

At low values of (ℓ_s/d_p), vertical shear failure occurs. The mean vertical shear stress on the concrete is roughly (V/bd_p). Longitudinal shear failure occurs at intermediate values (ℓ_s/d_p) and be on the line

$$\frac{V}{bd_p} = m\left(\frac{A_p}{b\ell_s}\right) + k \tag{18}$$

A typical set of tests consists of two groups of three or four each; one of these has ℓ_s/d_p values chosen in such a manner that the results be near the point *A* in Fig. 11. Second group is chosen with a lower ℓ_s/d_p values such that the results lie near the point *B*. Values of *m* and *k* are found for a line drawn below the lowest result in each group, at a distance that allows for the scatter of test data. The behaviour is controlled by the two parameters of the straight line, namely

- *m* - the slope of the line
- *k* - the intercept of *y*-axis.

The specifications require that all tests have to be in longitudinal shear. Typically the failure is initiated when a crack occurs in concrete under one of the load points, associated with loss of bond along the shear span. If this leads to failure of the slab - the shear connection is classified as "brittle", as they occur suddenly. These are penalised EC4 by *20%* reduction in design resistance. When the eventual failure load exceeds the load causing the first end slip by more than *10%*, the failure is classed as "ductile".

Because of the rather complex nature of the prescribed tests, manufacturers of profiled sheets generally provide *m* and *k* values based on tests carried out by independent laboratories.

4.2 Resistance to vertical shear

The resistance to vertical shear is mainly provided by the concrete ribs. For open profiles b_o [Fig. 5(*a*)] should be taken as effective width. The resistance of a concrete slab with ribs of effective width b_0 at a spacing of b is

$$V_{v.Rd} = (b_0/b)d_p\, \tau_{Rd}\, k_v\, (1.2 + 40\rho) \text{ per unit width} \qquad (19)$$

where d_p is the depth to the centroidal axis

τ_{Rd} is the basic shear strength of concrete

k_v allows higher shear strength for shallow members

$k_v = (1.6 - d_p) \geq 1$ with d_p in m

ρ allows a small contribution due to shearing

$$\rho = A_p / b_0\, d_p < 0.02 \qquad (20)$$

A_p = effective area of shearing within width b_0

5.0 SERVICEABILITY CRITERIA

The composite slab is checked for the following serviceability criteria:

(i) Cracking
(ii) Deflection
(iii) Fire endurance

5.1 Cracking

The crack width is calculated for the top surface in the negative moment region using standard methods prescribed for reinforced concrete. The method is detailed in the next chapter. Normally crack width should not exceed *3 mm*. IS 456: 2000 gives a formula to calculate the width of crack. Provision of *0.4 %* steel will normally avoid cracking problems in propped construction and provision *0.2 %* of steel is normally sufficient in un-propped construction. If environment is corrosive it is advisable to design the slab as continuous and take advantage of steel provided for negative bending moment for resisting cracking during service loads.

5.2 Deflection

The IS 456: 2000 gives a stringent deflection limitation of $\ell/350$ which may be un-realistic for un-propped construction. The Euro code gives limitations of $\ell/180$ or *20 mm* which ever is less. It may be worth while to limit span to depth ratio in the range of *25* to *35* for the composite condition, the former being adopted for simply supported slabs and the later for continuous slabs. The deflection of the composite slabs is influenced by the slip-taking place between sheeting and concrete. Tests seem to be the best method to estimate the actual deflection for the conditions adopted.

5.3 Fire endurance

The fire endurance is assumed based on the following two criteria:

- Thermal insulation criterion concerned with limiting the transmission of heat by conduction
- Integrity criterion concerned with preventing the flames and hot gases to nearby compartments.

It is met by specifying adequate thickness of insulation to protect combustible materials. *R* (time in minutes) denotes the fire resistance class of a member or component. For instance, *R60* means that failure time is more than *60* minutes. It is generally assumed that fire rating is *R60* for normal buildings. Reader can refer to reference 1 for further details.

6.0 CONCLUSION

This chapter described various behavioural aspects to be considered for composite floors using profiled sheeting. The design equations and application to simply supported and continuous slabs are included in the next chapter.

7.0 REFERENCES

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APPENDIX

ULTIMATE STRESS OF CONCRETE

Eurocode assumes ultimate stress of concrete as $0.85\ (f_{ck})_{cy}/\gamma_c$ where $(f_{ck})_{cy}$ is the characteristic cylinder compression strength of concrete and γ_c is partial safety factor that is equal to *1.50*. They adopt rectangular stress block with dimensions is shown in Fig. 11. For the sake of comparative study cylinder strength is changed to cube strength, $(f_{ck})_{cu}$. The ratio of cylinder strength to cube strength is adopted as *0.80*.

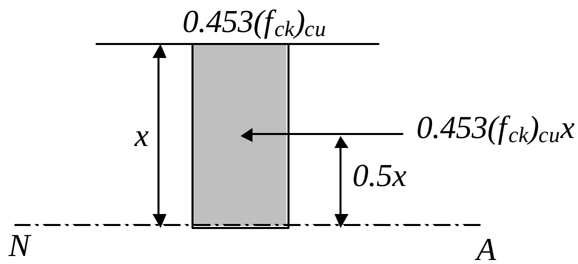

Fig. 11 Stress block for concrete strength according to Eurocode

$0.85(f_{ck})_{cy}/\gamma_m = 0.8 * 0.85(f_{ck})_{cu}/1.5 = 0.453(f_{ck})_{cu}$

$Moment\ capacity = 0.453\ (f_{ck})_{cu}\ x * 0.5x = 0.227\ (f_{ck})_{cu}\ x^2$

In contrast, IS: 456 - 2000 assumes parabolic stress block as shown in Fig. 12. The dimensions of the stress block are also shown in Fig. 12.

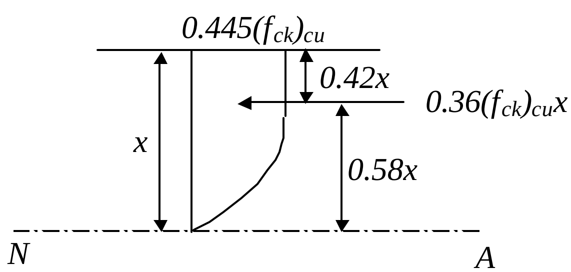

Fig. 12 Stress block for concrete strength according to IS: 456 - 2000

$Moment\ capacity = 0.36\ (f_{ck})_{cu}\ x * 0.58x = 0.209\ (f_{ck})_{cu}\ x^2$

IS code is *7 - 8 %* conservative compared to Eurocode, it is accounted for difference in quality control of the concrete at site. If, designer is confident about quality control of the concrete he can make proper choice between the two and arrive at an economic design.