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Composite action of structural steel beams and precast concrete slabs

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1. Introduction

This paper considers the innovative composite behaviour between structural steel beams and precast concrete planks for multi-storey buildings. Whilst it is common practice to use precast concrete planks in Australian building construction, the benefits of composite behaviour have not yet been fully realised with these systems, (National Precast Concrete Association of Australia, 2003). The main advantages of this form of construction are that the precast concrete units and the steel frames are both derived from manufacturing technology rather than a site based activity, and share the quality control, accuracy and reliability of factory production and thus lead to savings associated with reduced construction times. This paper considers the issues that need to be resolved in order to establish composite behaviour. The paper considers various structural forms including hollow-core (Figure 1) or solid precast concrete construction manufactured in Australia and considers the behaviour and design of these systems for both service and ultimate loads.



Figure 1 Precast concrete steel structural system

In Australia, the New South Wales Government White Paper Construct (New South Wales Government 1998) has placed an emphasis on seizing opportunities to build a better construction industry. The central tenet of this document is the promotion of ecologically sustainable development for government building projects. The theme of ecologically sustainable development as it applies to building construction involves the use of construction materials which can be reused as well as those materials which can improve the adverse affects on the environment. In particular the ability to eliminate and minimise wet trades from a construction site has a greater ability to satisfy this premise. The use of steel frames with precast concrete floors is thus a very innovative method in addressing the New South Wales Government White Paper which seeks to serve as a benchmark for the construction industry through inspiring a paradigm shift in construction. In Australia steel framed construction only represents a market share somewhere between 5-10 % of commercial buildings. Nevertheless, for multi-storey building construction it is certainly the exception and not the rule to use steel frames. Multi-storey and tall building structures tend to be dominated by reinforced concrete construction and for long spans prestressed concrete is quite commonly used. Whereas in steel frames the use of composite slabs utilising profiled steel decking supported and made composite with structural steel beams has definitely been the most widely used. Composite steel and precast concrete floor systems have the advantage of being able to span large distances during construction and eliminate the need for secondary steelwork for both the construction and in-situ conditions. A typical floorplan of a building is illustrated in Figure 2,

which shows primary steelwork spanning East-West, with the precast concrete floors spanning North-South. Figure 3 illustrates typical profiles which can be used for precast concrete floors, with hollowcore and solid slabs.

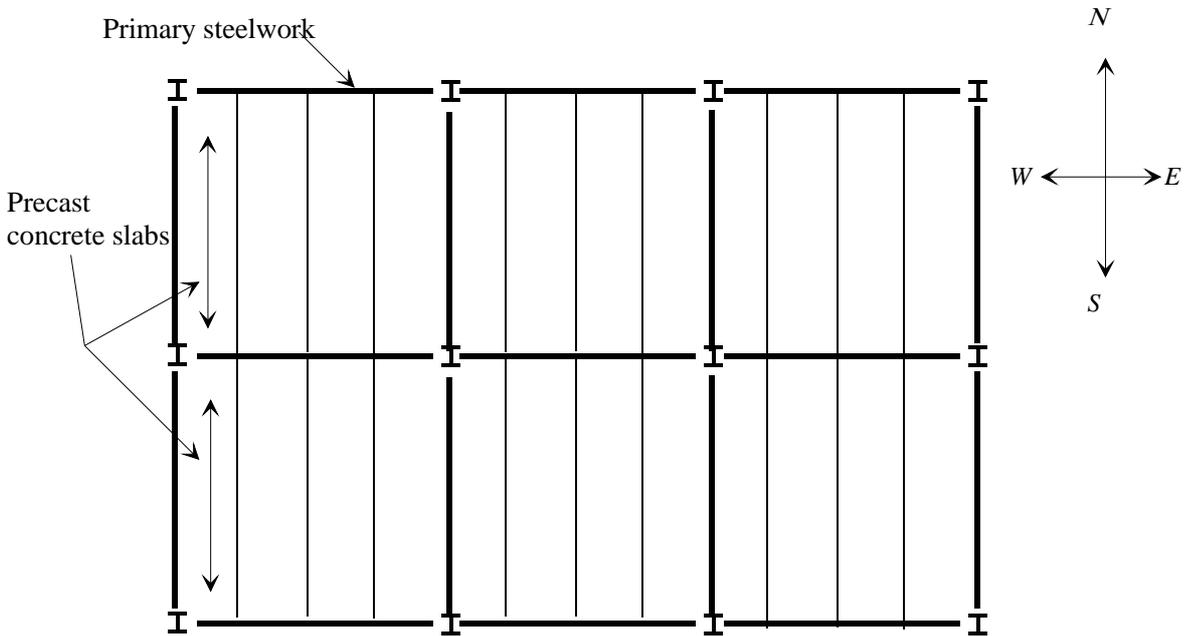


Figure 2 Typical composite steel and precast concrete floor system

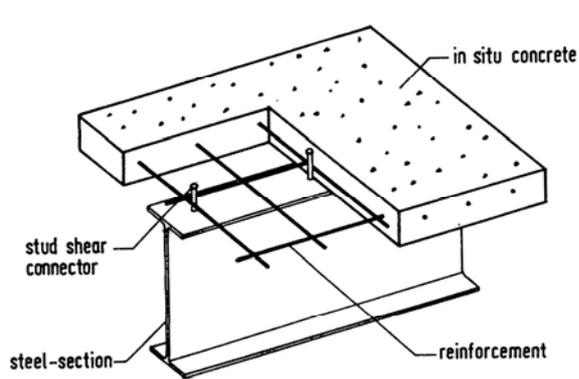


(a) Hollowcore precast units

(b) Solid precast units

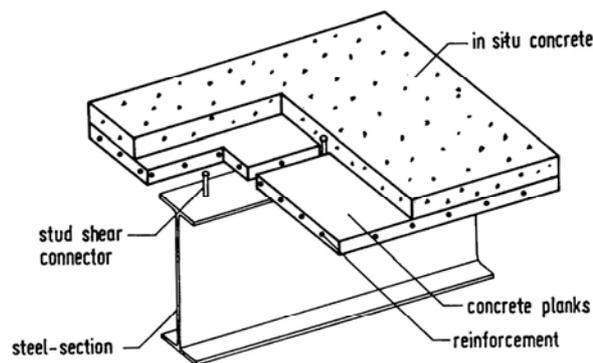
Figure 3 Typical Australian pretensioned precast concrete slabs, hollowcore and solid

Figure 4 illustrates the various types of composite steel-concrete beam systems that can be categorised, according to Uy and Liew (2001). This paper is principally concerned with the systems illustrated in (b) and (c). The system shown in Figure 4 (b) can be developed with pretensioned precast concrete planks, similar to the diagram in Figure 3. These pretensioned precast concrete planks are then overlaid with an in-situ concrete slab. Figure 4 (c) illustrates a system, whereby the precast element is a full depth element. These have more recently been made by pretensioned hollowcore units as illustrated in Figures 1 and 3.



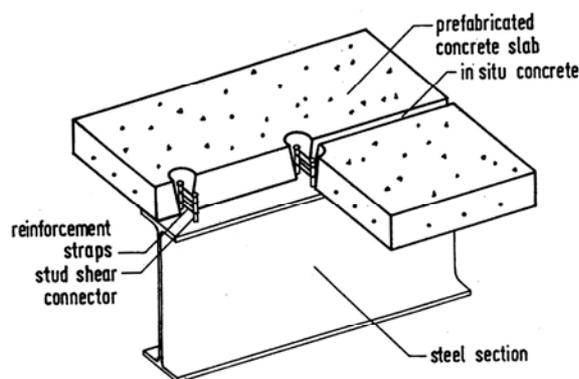
Composite beam with in situ concrete slab

(a)



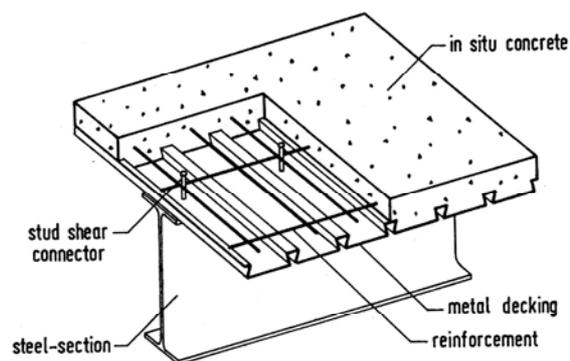
Precast reinforced concrete planks with in situ concrete topping slab

(b)



Composite beam floor using prefabricated concrete elements

(c)



Composite beam with in situ concrete slab on trapezoidal metal decking

(d)

Figure 4 Composite beam types, (Uy and Liew, 2001)

2. Previous research

In the last thirty years the steel construction industry has been transformed in the UK, such that the use of structural steel in multi-storey building construction (greater than 2 storeys) now is greater than 75 %. This is a trebling of the original figure of thirty years ago. Two very important contributors to this have been the development of composite construction and the use of prefabricated and precast technology in buildings. Lam et al. (1998 and 2000) have undertaken extensive research on the composite behaviour of precast concrete slabs on steel beams. This research has resulted in the publication of an industry wide design guideline prepared by the Precast Flooring Federation and the Steel Construction Institute, (Hicks and Lawson, 2003) and a suggested amendment to the current standard for composite design to allow for the effects of the infill concrete has been developed based on the experiments by Lam et al. (1998) and recently presented nationally in a paper by Lam and Uy (2003).

3. Pertinent design issues for steel and precast concrete systems

Design rules for steel-concrete composite beams have been developed initially for systems with solid reinforced concrete slabs as illustrated in Figure 4 (a). More recently international standards have incorporated design rules to account for profiled steel sheeting as illustrated in Figure 4 (d). There has been less attention to considering the design issues associated with precast concrete slabs. This paper attempts to identify the areas where special attention is required and where guidance may be provided in the absence of extensive research to date. The two most important limit states which structural engineers are concerned with include:

- Strength; and
- Serviceability

The issues associated with using precast concrete planks for these cases will be outlined in this paper and some preliminary design suggestions will be made.

4. Strength limit states of composite beams

This section briefly outlines the fundamental behaviour of steel-concrete composite beams and then considers special attention which may be required to be considered with the use of precast concrete slabs with in-situ concrete provided for the longitudinal shear connection.

4.1 Flexural strength of composite beams

The flexural strength of a composite steel-concrete beam as outlined by Oehlers and Bradford (1995) is dependent on three principal parameters, the concrete, steel and shear connectors. Three possible scenarios are then obtainable, which are outlined herein:

(a) Plastic neutral axis in the concrete slab (full shear connection, $\beta=1.0$)

When the concrete slab is stronger than the steel beams, the plastic neutral axis will lie within the concrete slab. For the case when the plastic neutral axis lies within the concrete slab, the ultimate flexural strength is determined from a simple couple as shown in Figure 5.

$$M_u = TL = CL \quad (1)$$

where $T = Ag.f_y$
and $C = 0.85fc b_{eff} d_n$

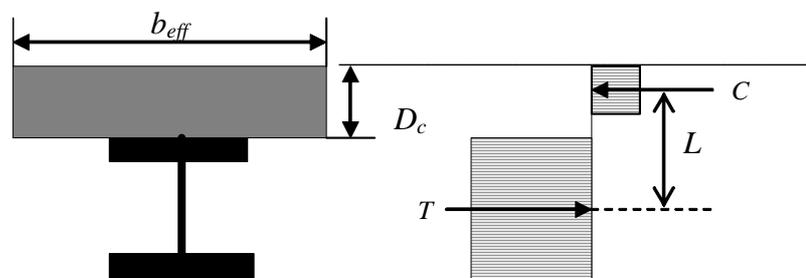


Figure 5 Ultimate flexural moment, plastic neutral axis in concrete slab ($\beta=1.0$)

(b) Plastic neutral axis in the steel beam (full shear connection, $\beta=1.0$)

When the steel beam is stronger than the concrete slab, the plastic neutral axis for the beam with full shear connection will lie within the steel beam. For this case it is more convenient to sum moments about the centroid of the tension force as illustrated in Figure 6.

$$M_u = C_c L_c + C_s L_s \quad (2)$$

where $C_c = 0.85 f_c b_{eff} D_c$

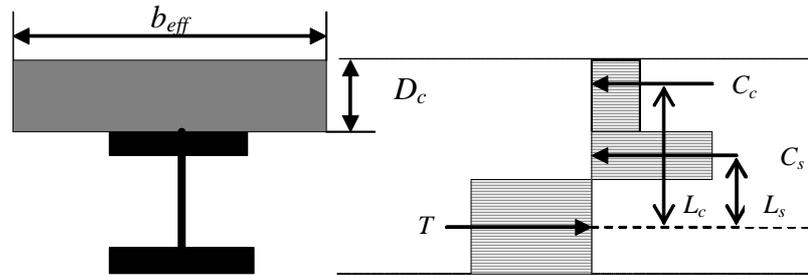


Figure 6 Ultimate flexural moment, plastic neutral axis in steel beam ($\beta=1.0$)

(c) Partial shear connection ($\beta < 1.0$)

For the case of partial shear connection of composite beams, the shear connection is the weakest element. Again, summing the moments on a convenient point on the cross-section will yield the ultimate flexural moment of the beam as illustrated in Figure 7.

$$M_u = C_c L_c + C_s L_s \quad (3)$$

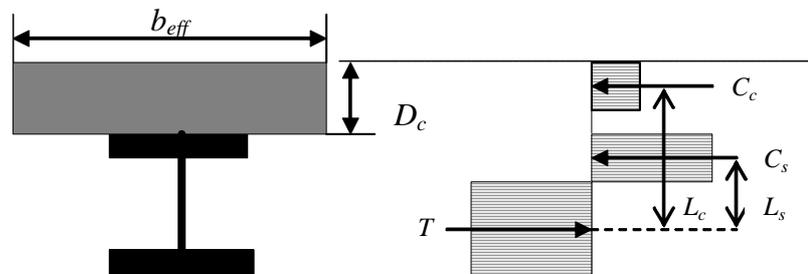


Figure 7 Ultimate flexural moment, partial shear connection ($\beta < 1.0$)

Existing International Standards that deal with the flexural strength of composite beams include the AISC-LRFD, (American Institute of Steel Construction, 1993), AS2327.1 (Standards Australia 1996 and 2003), BS5950: Part 3 (British Standards Institution, 1990) and Eurocode 4 ((British Standards Institution, 1994). Whilst some of these standards have a closed form solution for the flexural strength determination, it is best left in a more general form in terms of stress blocks as shown in Figures 5 to 7 and for individuals to refer to the individual regional standards to determine the strength equations and to apply the relevant load and capacity reduction factors. The most general manner in which to assess the flexural strength of a composite beam is as in Equation 4.

$$M^* \leq \phi M_u \quad (4)$$

Where M^* is the design bending moment at a cross-section and M_u is the ultimate moment of the cross-section and where ϕ is a capacity reduction factor which accounts for material inconsistencies and issues associated with ductility.

Existing rules for the use of precast planks in the UK, based on the research of Lam et al. (1998 and 2000) and presented in a design context by Hicks and Lawson (2003) only allow case (b) to be used. The reasons for this are to ensure that the neutral axis is always below the steel-concrete interface, so that at the ultimate limit state for flexure, there is no tension in the precast plank. This is to ensure that the precast plank is not subjected to tension. Reasons for this are obvious when using pre-tensioned concrete planks, as any tension in the plank could render the plank with some obvious safety problems if pretensioned wires lost their bond in the transverse direction due to the longitudinal tension created by primary bending.

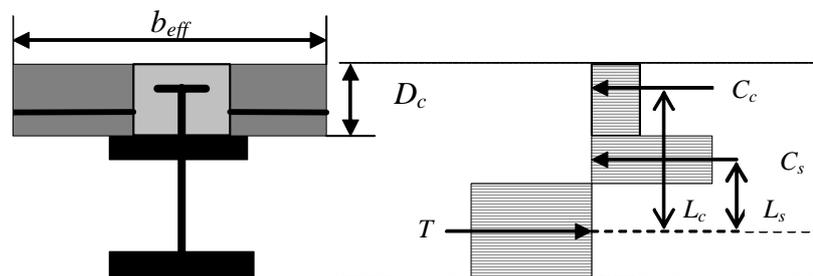


Figure 8 Ultimate flexural moment of precast concrete plank on steel beam with in-situ longitudinal shear connection

Figure 8 shows the method of calculation for the flexural strength of steel-concrete composite beams when precast planks are used with in-situ concrete provided for the longitudinal shear connection. Two important considerations are produced when such a situation arises.

1. If hollowcore units are used the determination of the effective width needs to account for the fact that the concrete in the longitudinal direction is unable to carry stress throughout the entire depth; and
2. The use of in-situ concrete for the longitudinal shear connection in systems where hollowcore and precast planks are used needs to consider that generally lower grade concrete is used for the in-situ concrete.

The following two sections detail the above two issues.

4.2 Effective width of composite beams

The effective width, b_{eff} of the concrete flange for positive bending in AS2327.1-1996 and 2003 for a solid slab in a beam for a regular floor system is determined as the minimum of the following

$$b_{eff} = \min\left(\frac{L_{ef}}{4}, b, b_{sf} + 16D_c\right) \quad (5)$$

Research conducted by Lam et al (1998 and 2000) and presented nationally by Lam and Uy (2003) has suggested that an effective width method can be used to further reduce the slab width to account for the loss in compression transfer which occurs in hollowcore units. The effective width has been suggested as

$$b_{eff} = \left(\frac{\sqrt{f_{cu}}}{40} \frac{32\phi}{500} \frac{f_f}{460} 1000 \right) + 2.5g \quad (6)$$

where f_{cu} is the insitu concrete strength
 ϕ is the diameter of the transverse reinforcement
 f_y is the characteristic strength of the reinforcement
 g is the gap between the ends of the precast units

The effective width determined from Equation 6 however can not exceed that determined from Equation 5.

4.3 Shear connector behaviour

Shear connectors may exist in quite a few varieties, which include headed shear studs, steel angles and high strength friction grip bolts. However, it is the headed shear stud connectors, which have seen the greatest application, and these will be outlined herein. In the design of the shear connection in composite structures, the designer is mainly interested in the strength, which each stud can transfer in shear. Empirical relationships for the shear resistance of headed shear studs exist in various international codes of practice. The Australian Standard, AS2327.1 (AS2327.1-1996 and 2003), represents the strength of the shear connectors by the lesser of one of these two following expressions

$$f_{vs} = 0.63d_{bs}^2 f_{uc} \quad (7)$$

$$f_{vs} = 0.31d_{bs}^2 \sqrt{f'_{cj} E_c} \quad (8)$$

where d_{bs} is the diameter of the shank of the stud
 f_{uc} is the ultimate strength of the material of the stud
 f'_{cj} is the characteristic cylinder strength of the concrete
 E_c is the mean value of the secant modulus of the concrete

Equation 7 represents the strength of the shear stud if it fails by fracture of the weld collar, whereas Equation 8 represents concrete cone failure surrounding the stud. The design shear resistance of studs in Eurocode 4 for the same failure modes is given by the following equations 9 and 10.

$$P_{Rd} = 0.8 f_u (\pi d^2 / 4) / \gamma_{Mv} \quad (9)$$

$$P_{Rd} = 0.29 \alpha d^2 (f_{ck} E_{cm})^{1/2} / \gamma_{Mv} \quad (10)$$

where d is the diameter of the shank of the stud

f_u	is the ultimate strength of the material of the stud
f_{ck}	is the characteristic cylinder strength of the concrete
E_{cm}	is the mean value of the secant modulus of the concrete
h	is the overall height of the stud
γ_{Mv}	is a partial safety factor, taken as 1.25 for the ultimate limit state
α	$= 0.2[(h/d) + 1]$ for $3 \leq h/d \leq 4$ $= 1.0$ for $h/d > 4$.

Research conducted by Lam et al (1998 and 2000) and presented nationally by Lam and Uy (2003) has suggested that the design shear resistance of an automatically welded headed stud with a normal weld collar in a hollowcore unit should be determined from

$$P_{RD} = 0.8f_u(\pi d^2 / 4) / \gamma_v \quad (11)$$

or

$$P_{RD} = 0.29\alpha\beta\epsilon d^2 \sqrt{\omega f_{cp} E_{cp}} / \gamma_v \quad (12)$$

whichever is smaller, where

d	is the diameter of the shank of the stud;
f_u	is the specified ultimate tensile strength of the material of the stud ;
β	is a factor which takes into account the gap width g (mm) and is given as $0.5 (g/70 + 1) \leq 1.0$, and $g \geq 30$ mm;
ϵ	is a factor which takes into account the diameter ϕ of transverse high tensile tie steel (grade 460) and is given by $0.5 (\phi/20 + 1) \leq 1.0$, and $\phi \geq 8$ mm;
ω	is a transverse joint factor = $0.5(w/600 + 1)$, w = width of hcu
f_{cp}	is the average concrete cylinder strength = $0.8 \times$ average cube strength of the insitu and precast concrete;
E_{cp}	is the average value of elastic modulus of the insitu and precast concrete.

The partial safety factor γ_v should be taken as 1.25 for the ultimate limit state. The existing expressions for the Australian Standards in Equations 6 and 7 could thus be rewritten, to incorporate the same factors which the expressions in Equations 11 and 12 account for. Thus, expressions for the shear capacity used in Australian construction to account for the reductions in capacity experienced in hollowcore units can be expressed as the lesser of Equations 13 and 14.

$$f_{vs} = 0.63d_{bs}^2 f_{uc} \quad (13)$$

$$f_{vs} = 0.31d_{bs}^2 \beta \epsilon \sqrt{\omega f_{cp}' E_c} \quad (14)$$

5. Serviceability limit states of composite beams

The behaviour of composite steel-concrete beams under service loads generally requires some consideration of the composite behaviour between the steel and the concrete. The nuances associated with using precast concrete planks with steel beams will be explained herein.

Deflections of simply supported composite beams need to incorporate both the effects of creep and shrinkage in addition to the loading effects. These time dependent effects are taken into account by generally transforming the concrete slab to an equivalent area of steel using a modular ratio. The modular ratio should include the effects of the disparate elastic moduli as well as the effects of creep of concrete. The effects of the use of two different concrete moduli also further complicates the issue when using precast concrete and cast in-situ concrete. The concrete is considered to be fully effective, however the effects of shear lag need to be determined using an effective breadth relationship. For the determination of deflections in AS2327.1 (Standards Australia, 1996 and 2003) the modular ratio is determined for immediate deflections using the value (E_s/E_c), whilst for long term deflections a modular ratio of 3 is suggested. The effective second moments of area of composite beams for immediate and long term deflections respectively are calculated in the Australian Standards, AS2327.1 (Standards Australia, 1996 and 2003) as

$$I_{eti} = I_{ti} + 0.6(1 - \beta_m)(I_s - I_{ti}) \quad (15)$$

$$I_{elt} = I_{tl} + 0.6(1 - \beta_m)(I_s - I_{tl}) \quad (16)$$

where I_{ti} and I_{tl} are the transformed second moments of area of a composite beam under immediate and long term loads, β_m is the level of shear connection and where I_s is the second moment of area of the steel section alone. Now, even though the current Australian Standard allows for the use of partial shear connection, it is recommended for precast pre-tensioned units that full shear connection is used ($\beta_m = 1.0$) so that there is no tension generated in the base of the slab unit. The reasons once again are to ensure that longitudinal tension in the slab in the direction of the beam span does not cause any deleterious effect to the bond between the pretensioned wires in the slab in the transverse direction to the steel beam.

6. Conclusions and further research

This paper has introduced the concept of using precast concrete planks in combination with structural steel beams to determine composite action for floor systems. Current Australian practices have been highlighted and previous research to address this issue overseas have been summarised. The pertinent design issues which need to be considered for these systems, for the strength and serviceability limit states have been outlined and modifications to existing Australian design procedures have been suggested.

Whilst this paper has highlighted some suggested procedures for the design of these systems, there are still quite a few outstanding issues that need to be considered. For example, the design of the longitudinal shear connection for solid precast planks when lower strength in-situ concrete is used, needs some consideration. Furthermore, whilst UK practices suggest that partial shear connection is not able to be used, some research needs to be conducted to consider the behaviour of these systems if partial shear connection results. Furthermore, exploration of these systems with continuity in the direction of the steel beam could lead to some further advantages and requires further research.

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