Composite Steel Beam Behaviour with Precast Rib Flooring

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Introduction
Floors consisting of precast ribs supported on structural steel beams and in situ structural topping are a common form of composite floor construction. In this article guidance is given on the modifications required to previously published design guidance for computing shear stud capacities to adapt them for use with precast rib flooring systems. In addition consideration will be given to steel beam top flange stability during construction using this type of flooring system. COBENZ 97 is a popular spreadsheet based programme used for steel composite beam design. Explanation is given to allow designers to use this programme to design composite floor systems utilising precast ribbed floor systems.

Shear Stud Capacity
Typically composite shear capacity expressions are based on tests in which concrete splitting is prevented before stud fracture or stud pull-out occurred (Hyland et al 2001). However, the performance of studs used in conjunction with a deep ribbed profile is affected by the relatively high concrete haunch depth, and the spacing of the deck ribs which provide a beneficial buttressing effect. With this type of decking system concrete splitting may occur before stud fracture. Long studs should be used in these circumstances as they suppress any tendency for stud pullout failure mechanisms.

Design equations have been published (Hyland et al 2001) for computing the inter-face shear capacities for use with deep profile metal deck systems. These equations are applicable with minor modification for use with precast ribbed flooring. Guidance is required to account for the sloped haunch geometry. For this situation (Oehler’s and Bradford, 1995) the shear capacity may be assessed by considering two rectangular concrete prisms of different widths \( (b_{ce,o} , b_{ce,i} ) \). These enclose the sloping portion of the haunch (Figure 1). The stud interface shear capacity will be intermediate between that of the inner and outer rectangular prisms and is a function of the haunch slope \( \theta \) (measured in degrees).

\[
(V_{split})_0 = (V_{split})_i + (V_{split})_o - (V_{split})_i \frac{\theta}{90} 
\]

(1)
$V_{\text{split}}$ is computed using equation (2).

$$V_{\text{split}} = \frac{0.6b_{ce}h_a f_{cb}}{k_d} + \frac{0.6d_{sc} f_{cb} \pi}{1 - \frac{h_a}{h_c}^2}$$

(2)

where:

- $b_{ce}$: Effective haunch width (mm)
- $d_{sc}$: Shear stud shank diameter
- $f_c$: Concrete cylinder strength (MPa)
- $f_{cb}$: Concrete splitting strength (MPa)
- $h_a = 1.8d_{sc}$: Effective bearing height (mm)
- $h_c = \min(4.5h_a, t_o)$: Effective haunch height (mm)
- $k_d = \frac{1}{\pi} \left(1 - \frac{d_{sc}}{b_{ce}}\right)^2$: Lateral force parameter
- $t_o$: Overall slab thickness (mm)
- $V_{\text{split}}$: Nominal stud splitting capacity (N/stud)

Once the stud shear capacity limited by splitting has been computed, equation 3 can then be used to derive the ultimate limit state stud capacity.

$$\phi_{sc} q_{fr} = \phi_{sc} \min \left( V_{\text{split}} / A_{sc} f_u, 0.8A_{sc} f_u \right)$$

(3)

where:

- $A_{sc}$: Shank area of stud (mm$^2$)
- $f_u$: Minimum tensile strength of stud (415 MPa)
- $\phi_{sc} 0.8A_{sc} f_u$: Stud shear capacity limited by steel fracture (N/stud)
- $\phi_{sc}$: Stud strength factor, taken as one in positive moment regions (table 13.1.2(1); NZS3404)
- $q_{fr}$: Stud design capacity (N/stud)

To ensure post splitting behaviour is reliable, transverse confinement reinforcement is required at the base of the studs. The minimum specified transverse reinforcement quantity ($A_{rt}$: section 13.4.10.4 NZS 3404) will ensure the post splitting stud shear strength is at least equal to the splitting strength, see equation 4.

$$A_{rt} \geq 430 \frac{s_{sc}^2 M^*}{\phi M_{tc}}$$

(4)

where:

- $s_{sc}$: Stud spacing (mm)
- $M^*$: Maximum design moment (kNm)
- $\phi M_{tc}$: Composite beam flexural design capacity

**Beam Top Flange Stability During Construction**

Precast ribs seated on the beam top flange do not typically provide reliable top flange restraint during construction. To avoid potential issues of top flange instability and high warping stresses in the bare steel section due to torsion, it is recommended that the ribs and steel beam are propped and that the beam is designed as a propped composite beam.
**COBENZ and Precast Ribbed Floors**

COBENZ 97 is a spreadsheet-based design tool for the design of composite steel beams with concrete floors. A user defined deck type must be selected for the floor system. The parameters required to define the ribbed floor system are input in the library worksheet. Guidance is given in the About COBENZ section found in the Licence worksheet to assist users with this task. The rib height variable \( (h_r) \) should include the timber planking depth as well as the rib height. To ensure the correct values have been input, the net topping thickness should be checked. In most instances for ribbed floors the topping depth is 75mm. The variable \( w_r \) can be adjusted to set the stud capacity in the work sheet to the value of \( \phi_{sc} q_r \) computed using equation 3. This will ensure the correct number of studs is determined.

**Worked Example**

Floor System (propped during construction)

Beams: 460UB82 9.5m long  
Slab: 200mm ribs, 25mm timber infill, 75mm topping, spanning 10m  
Studs: 19mm diameter 250mm long (studs must be \( \geq 40 \)mm above rib, section 13.3.2.2.1, NZS 3404)  
Concrete strength: \( f'_c = 30 \) MPa

**Design Loads**

SDL 0.75 kPa  
LL  1.5 kPa

**Step 1 – Stud Capacity**

For the purposes of the example the following parameters have been used:

\[
(b_{ce,o}=466mm; b_{ce,i}=120mm; \theta = 30 \text{ degrees})
\]

Based on these parameters, the following shear capacities are computed using equation 2; \( V_{\text{split},o} = 114 \)kN, \( V_{\text{split},i} = 54 \)kN. Substituting these values in equation 1 gives the following stud capacity limited by splitting of:

\[
(V_{\text{split},o}) = \left[V_{\text{split},i}\right]_o - \left(V_{\text{split}}\right)_i \left(\frac{0}{90}\right) = 54 + [114 - 54]\left(\frac{30}{90}\right) = 74 \text{kN}
\]

The design stud capacity is then determined using equation 3:

\[
\phi_{sc} q_r = \min\left(\phi_{sc} V_{\text{split},o}\right) \cdot \phi_{sc} 0.8A_{sc} f_{u} = \min(74,94) = 74 \text{kN}
\]

**Step 2 – COBENZ**

The precast floor data has been input into COBENZ using the custom floor option under deck type. As discussed in the COBENZ user information a high value of the moment of inertia \( I_d \) has been used to ensure the concrete factor is set to one. By setting the value of \( w_r \) to 767mm, the correct stud capacity \( \phi_{sc} q_r \) has been computed by the programme.

**Step 3 – Confining Reinforcement**

To achieve the required 50% composite action to provide adequate flexural strength to resist the design actions, a total of 43 studs must be provided. This equates to a stud spacing of approximately 9500/42=225mm. The quantity of confining steel required is found using equation 4.

\[
A_{rt} \geq 430 \frac{d_{sc}^2}{S_{sc}} \cdot \frac{M^*}{\phi M_{tc}} = 430 \cdot \frac{19.2^2}{225} \cdot \frac{759}{991} = 530 \text{mm}^2 / \text{m}
\]

A D12 rebar at 210 centres will meet this requirement.
References


SNZ, Steel Structures Standard; NZS 3404:1997 (Incorporating Amendment 1 and 2), Standards New Zealand, Wellington, 2007

Hyland, C.W.K., COBENZ 97, Hyland Consultants Ltd, Manukau City, 2006