Composite Steel Beam Behaviour with Precast Rib Flooring

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composite steel beams, precast ribbed flooring, shear stud capacity

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

Detailing of the Shear Connection

There are a number of detailing requirements to ensure dependable composite action can be achieved for precast ribs supported on structural steel beams. These requirements have been taken from Eurocode, British and Australian Standards for composite beam design. Refer to figures 1 and 2. The sides of the haunch must lie outside a line drawn at 45° from the outside edge of the shear stud. The nominal concrete cover from the side of the haunch to the shear stud must be not less than 50mm. Transverse reinforcing bars must be provided in the haunch at not less than 40mm clear below the head of the shear stud.

The shear stud height must be greater than four times the stud diameter. The shear stud need not extend above the precast rib height.

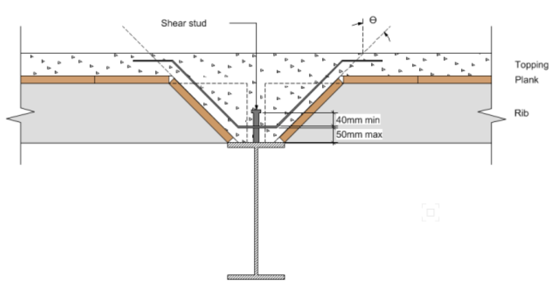


Figure 1: Cross section – composite steel and precast ribbed floor

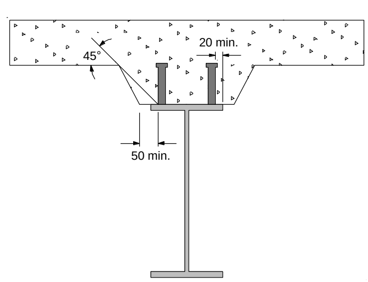


Figure 2: Haunch Detailing

Shear Stud Capacity

The shear stud capacity is determined using the provisions of the *Steel Structures Standard*, NZS 3404 clause 13.3.2.1 for shear studs in a solid slab. This is only applicable if the detailing requirements of above are used.

Longitudinal Shear

Providing that the shear connectors may be considered ductile (defined by a characteristic slip capacity of 6 mm), the longitudinal shear force at the shear connection may be assumed to be redistributed between connectors such that a rigid plastic distribution may be assumed in design. The assumption of a rigid plastic longitudinal shear force permits the designer to assume that each shear connector is equally loaded. To ensure that the composite beam is able to develop sufficient rotation such that the moment resistance may be calculated by means of rigid plastic theory, the longitudinal shear resistance of the concrete flange must be kept larger than the longitudinal resistance of the shear connection. Shear connectors that consist of 19 mm diameter headed studs, which conform to AS/NZS 1554.2, may be considered to be ductile. Otherwise, the composite beam and shear connection should be designed using elastic theory.

The shear stud capacity is determined using the provisions of NZS 3404 clause 13.3.2.1 for a shear stud in a solid slab. This is only appropriate where the above detailing provisions are adhered to.

The potential longitudinal shear planes are shown in figure 3. The longitudinal resistance of the concrete is given in NZS 3404 clause 13.4.10.2.

To ensure the longitudinal resistance of the concrete is larger than the longitudinal resistance of the shear studs, transverse reinforcement is required in the haunch near the base of the shear studs.

The transverse reinforcement at the base of the studs also ensures post splitting behaviour is reliable. Where the side cover to the stud is less than 10 times the stud diameter NZS 3404 clause 13.4.10.4 require a check that the amount of transverse reinforcement is sufficient to ensure shear stud strength is maintained in the event of cracking along the line of the shear studs.

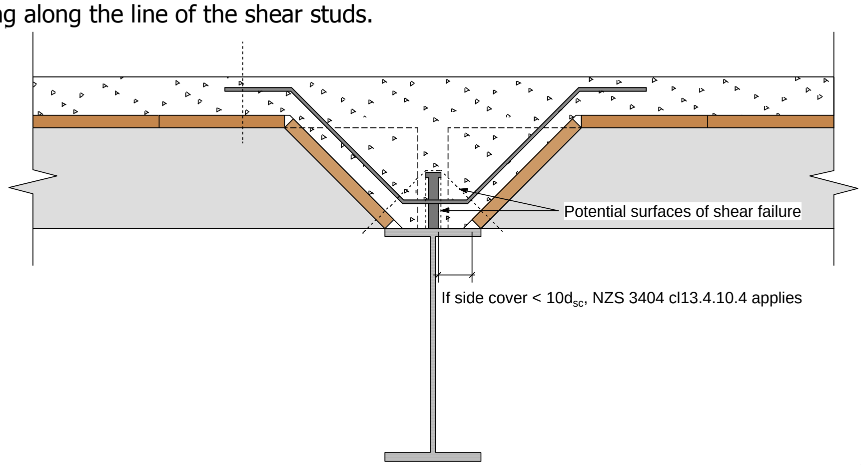


Figure 3: Typical potential surfaces of shear failure (topping reinforcement not shown)

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.

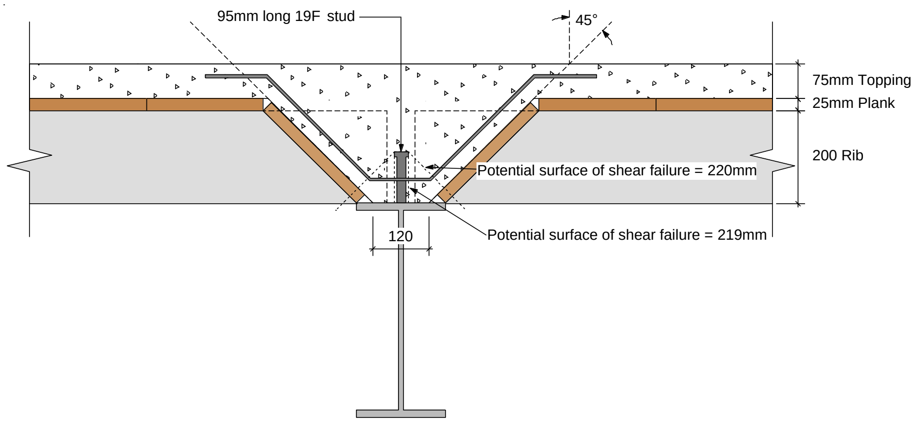
COBENZ and Precast Ribbed Floors

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 95mm long, f_u=415 MPa
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.
Base haunch width = 120mm
Haunch angle = 45 degrees

For headed studs with h_{sc}/d_{sc}>4 the nominal stud capacity is determined using NZS 3404 equation 13.3.2.1.

$$q_r = \alpha_{dc} 0.13\sqrt{f_c^{'}} A_{sc} f_u \leq 0.8 f_u A_{sc}$$

For a solid slab $\alpha_{dc} = 1$

$$q_r = \min(84, 94) = 84\ kN$$

The design stud capacity is

$$\phi_{sc} q_r = 1.0 \times 84 = 84\ kN$$

Step 2 – COBENZ

The precast floor data has been inputted into COBENZ using the custom floor option under deck type. The variable w_r is adjusted to ensure the correct stud capacity $\phi_{sc} q_r$ has been computed by the programme.

To achieve the NZS 3404 cl 13.4.6.1 minimum degree of shear connection of 50%, a total of 38 studs must be provided. This equates to a stud spacing of approximately 9500/38=250mm.

Step 3 – Design Longitudinal Shear

The design longitudinal shear (V*l) is determined using the equation from NZS 3404 cl 13.4.10.1.

$$V_l^* = \sum \phi_{sc} q_r - 0.85 \phi_{sc} f_c^{'} A_{cs} - \phi A_{rl} f_{yr}$$

If the presence of any longitudinal reinforcement and concrete cross section area under compression between shear planes is ignored the equation simplifies to the following:

$$V_l^* = \sum \phi_{sc} q_r$$

There is a total of 19 studs over half the beam span. This results in a design longitudinal shear of

$$V_l^* = 19 \times 84 = 1596\ kN$$

Step 4 – Longitudinal Shear Resistance

The design shear resistance for normal density concrete along any potential shear surfaces is given by equation from NZS 3404 cl 13.4.10.2

$$V_r = \left(0.80\phi A_{rt} f_{yr} + 2.76\phi_c A_{cv}\right) \leq 0.50\phi_c f_c^{'} A_{cv}$$

To suppress longitudinal shear failure, $V_l^* <= V_r$. The equation for design shear resistance is rearranged to determine the amount of transverse reinforcement required.

$$A_{rt} \geq \frac{V_l^* - 2.76\phi_c A_{cv}}{0.80\phi f_{yr}}$$

In the haunch area the potential shear failure surfaces is shown in figure 3. The shear surface at right angles to the haunch is 220mm/m. The shear surface immediately around the stud is 219mm/m.

The minimum transverse reinforcement requirements apply as:

$$2.76\phi_c A_{cv} = 2.76\,MPa \times 0.6 \times 219mm \times 4.75m = 1723\,kN > V_l^*$$

The design shear resistance equation in NZS 3404 cl 13.4.10.2 comes from limits proposed by Mattock et al, 1974. The equation is only valid if the reinforcement contribution to the shear resistance is greater than 1.38 MPa. i.e.

$$\rho f_y \geq 1.38MPa$$

where :

ρ = reinforcement percentage

Therefore minimum hanger reinforcement required

$$A_{rt,\min} = \frac{1.38MPa}{500MPa} \times \frac{219mm}{2} \times 1000 = 302\,mm^2\ /\ m$$

Step 5 – NZS 3404 clause 13.4.10.4 Check

The quantity of confining steel required is found using the equation in cl 13.4.10.4, NZS 3404. φM_{rc} is taken from COBENZ.

$$A_{rt} \geq 430\frac{d_{sc}^2}{s_{sc}}\frac{M^*}{\varphi M_{rc}} = 430.\frac{19^2}{250}.\frac{759}{991} = 476mm^2\ /\ m\ .$$

Therefore hanger reinforcement required is

$A_{rt} = \left(302, 476\right)_{\max} = 476mm^2\ /\ m$ A D12 rebar at 230 centres will meet this requirement.

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