Experimental study on the use of steel-decks for prefabricated reinforced concrete beams

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Abstract. This paper presents an experimental study on the use of steel-decks for concrete beams. The purpose of this research is to determine the beam’s capacity, and the load-displacement relationships due to the use of steel-decks. The failure mechanism was also studied, since the behavior differs significantly from conventional concrete members. For analysis purposes, two beam prototypes with steel-decks (GB1 and GB2), and two conventional concrete beams having the exact same material properties and dimensions (NB1 and NB2) functioning as control elements, were tested. Load was applied by a two-point loading system, creating a pure bending state. To monitor vertical deflections, two LVDTs were used. All precision instruments were connected to a data logger, and a computer. The results showed that the beams GB had a significant ultimate moment capacity increase, which is 2.3 times the control element NB. The main enhancement contribution is originated from the presence of the bottom steel-deck, which due to bonding to the concrete, functioned as additional tensile reinforcement. The deck also increased the member’s ductility performance by 1.3 times. Specimen GB2 underwent bond loss in the transition zone between the deck and the concrete, reducing the initial stiffness of the member.

1. Introduction

The revolutions and evolutions in the concrete industry today demand for innovations in productivity, effectivity and efficiency. Optimization leading to more effective and efficient construction methods in terms of time, execution, and cost will provide benefits for all parties.

The use of steel-decks as formwork for slabs and concrete floors have been widely used. The deck functions, besides as formwork, also as tensile reinforcement in the tension zone of the plate. The installation of steel deck formworks that do not require dismantling, and demand a minimum in scaffolding, will speed up the construction process and make the space below the construction operational faster. The steel deck formwork offers a very attractive alternative to the traditional scaffolding and framework that needs to be removed. In terms of cost efficiency, the addition of the permanent formwork will reduce the overall cost, and makes it an interesting choice. The research of Uji [1] demonstrated that the use of steel-decks as formworks can provide cost savings up till 28.12% when compared to traditional construction methods. In terms of time savings, the steel-decks can contribute to a 41.67% in time reduction.

The financial benefits of steel-decks forms would become more sophisticated when the structural improvements are analysed clearly. Kan et al. [2] conducted a study on the mechanical behavior and responses of steel-decks forms for light weight concrete beams. Their research concluded that the optimum load was achieved as a result of de-bonding in the interface between the steel-decks and the
concrete. The load-displacement curve then dropped abruptly, resulting in failure of the specimen. The presence of screws functioning as shear connectors influenced the ductility performance of the member, since the post peak curve was more pronounced for the specimens with screws. The use of screws is of crucial importance in maintaining the bond between the steel-decks and the concrete. A good bond will result in a compatibility mode, and then the steel and concrete will act as a composite member in carrying the load. Merryfield et al. [3] declared that the use of puddle-welds as shear connectors ensured a good bond, and the composite performance of the combined material improved significantly. At service loading, the actual vertical deflection of the beam was reduced to 70% of its allowed value, while the strain of reinforcing steel was lessened to only 20% of its yield strain. The main focus of the study was to determine the contribution of the steel-decks to load carrying capacity increase and the stiffness of the member, while the moment-curvature behavior was evaluated through the strain in the longitudinal tensile reinforcement and concrete in compression. This study focused on the potentials, and evaluation in behavior of a steel-decks formwork for a beam element, since up till now the beams of a steel-decks formwork plate or slab were constructed using a conventional casting system.

2. Methods
The response and behavior of the steel-decks formwork concrete beams GB1 and GB2 was studied by method of comparison to the control elements. These control elements were identical to the GB specimens in terms of their dimension, reinforcing steel configuration and concrete strength aspects. The GB1 and GB2 specimens were 200 mm by 300 mm. At the bottom, a corrugated sheet 1 mm thick was used as formwork, as well as tensile reinforcement [4]. The side panels were flat sheets having the same thickness as the bottom formwork. The panels were connected with a strip having a distance of 500 mm apart to ensure sufficient stiffness to maintain the design configuration of the beams’ cross section (figure 1). Connections between elements was provided by the use of self-drilling screws having a diameter of 4 mm as can be seen in figure 1b. These screws were positioned at a distance of 200 mm apart. The steel deck has a bilinear stress-strain behavior with a yield stress ($f_y$) of 300 MPa in combination with a yield strain ($\varepsilon_y$) of 0.0015. The Young’s modulus was recorded to be 200 GPa.

The control members NB1 and NB2 were 200 mm by 284 mm reinforced concrete beams, having 2D12 mm reinforcing steel in the tension as well as the compression zone. The steel had a yield stress ($f_y$) of 400 MPa in combination with a yield strain ($\varepsilon_y$) of 0.002. Further, the stress-strain behavior of these steel bars followed the standard yield pattern and a non-linear curve at strain hardening. The ultimate strength was 400 MPa and the Young’s modulus was 192 GPa. The mechanical properties of the steel-decks thus closely approached the properties of the reinforcing steel.

The 284 mm height of the NB cross section was a conversion of the 300 mm GB beam’s height, targeting a constant section area. The conversion accommodated the depth of the corrugated, part measuring 40 mm (figure 1). The concrete had a volumetric material mix proportion of 1 : 2 : 3 for the cement, fine and coarse aggregates respectively, while the water-cement-ratio was set at 0.5. The cylindrical compression strength at 28 days was measured to be 19.8 MPa. This simplified mix design...
The beam was simply supported and had a length of 2.440 mm, the supports were distanced 2.280 mm apart. The two-points loading system had a distance of 760 mm as can be seen in figure 2. The load was recorded by a load cell type CLC-500kN with a capacity of 500 kN while the vertical displacements were monitored by two Linear Variable Displacement Transducer (LVDT) type SDP-100C. A acrylic base plate with sufficient stiffness was attached at the base of the beam at mid-span, as reading point for the LVDT’s (figure 2a). To obtain the strain behavior of reinforcing steel and steel-decks, strain gauges type FLA 6-11 were attached to the reinforcing steel bars in tension, and the steel-decks at mid-point. The compression strain of the concrete was recorded using the PL 60-11 strain gauges situated at the top of the section in the longitudinal direction of the beam (figure 2b).

A monotonic increment load was applied to the beam till failure, and all the data was stored. The load data was further analyzed and transferred to the moment-deflection relationship, while the strain information was used to generate the curvature development as a function of the load increase. The bending moment area underneath the loads had a constant positive bending moment.

\[
M = \left( \frac{1}{2} \right) \left( \frac{1}{3} \right) P L = \frac{1}{6} P L
\]  

(1)

where P is the load recorded by the load cell in kN and L is the beam’s clear span 2.28 meters in length. This two-point loading system ensures a pure state of bending. As for the curvature determination, the strain in the concrete in compression \( \varepsilon_c \) and the strain in the tension reinforcement \( \varepsilon_s \) were used (figure 3). The strain in the steel deck \( \varepsilon_D \) functioned as controlling strain. The curvature was calculated using simple mathematics as:

\[
\phi = \frac{\varepsilon_c + \varepsilon_s}{d}
\]

(2)
3. Data analyses and discussion

3.1 Moment-displacement response and curvature

The response of the load in kN was converted to the bending moment capacity of the section, since this provided a better understanding to the overall strength of the beam under consideration (figure 4). The control elements NB1 and NB2 showed a typical under-reinforced concrete member behavior in bending. The initial stiffness of the member was high and was calculated to be 11 kN/mm.

The initial stages exhibited a linear response, and the moment-displacement trajectory followed a straight path. The first crack in the concrete tension zone was identified at 6 kN-m, the curve underwent an unmistakable deviation in stiffness. This decrease in member stiffness was due to a reduction in moment of inertia of the section. Visual observation to the behavior of the beam showed that this first crack occurred at mid-span. The moment-displacement pattern tends to become non-linear. The load was increased, and at 18 kN-m the tensile reinforcement yielded. The curve diverged significantly and was distinguished by a yield plateau. The recordings of strain gauges on the reinforcing steel confirmed...
that the steel yielded. The strain hardening of these steel bars resulted in a slight capacity increase, and the average of ultimate moment capacity of the member was determined at 20 kN-m.

The composite beams demonstrated a different moment-displacement response. The initial stiffness of the member GB1 and GB2 were remarkably higher when compared to the NB beams. The GB1 specimen had an initial stiffness of 35 kN/mm, while specimen GB2 had a stiffness of 24 kN/mm. The steel-decks formwork increased the initial stiffness of the member 2.1 ~ 3.1 times. Closer analysis revealed that the major contribution to this stiffness enhance was originated from the side steel panels, especially during the period that the bond between these side-panels and the concrete was intact. As load increased, this bond was lost, and the concrete element underwent substantial slipping with respect to the steel-decks formwork, as can be seen in figures 5a and 5b. Prior to this, the stiffness dropped gradually, resulting in the non-linear behavior at later stages of loading.

![Figure 5. Side-panel bond loss and bond-slip.](image)

The path of the curves then deviated substantially, and this was diagnosed as the stage at which the tensile reinforcement yielded. The moment capacity was calculated to be 45 kN-m and 44 kN-m for specimens GB1 and GB2 respectively. The use of the steel deck formwork enhanced the load carrying capacity 2.5 times. The first crack could not be detected, not numerically nor visually, since the concrete view was blocked by the presence of the steel-decks. Table 1 shows the values as obtained by the experimental test results. The control elements NB1 and NB2 had almost identical outcomes, and the average of these were taken for evaluation purposes.

| Specimen | M_y (kN-m) | GB/NB ratio | M_u (kN-m) | GB/NB ratio | Stiffness K (kN/mm) | GB/NB ratio |
|----------|------------|-------------|------------|-------------|---------------------|-------------|
| GB1      | 45         | 2.5         | 46         | 2.3         | 35                  | 3.1         |
| GB2      | 44         | 2.4         | 44         | 2.2         | 24                  | 2.1         |
| NB_av    | 18         | 1.0         | 20         | 1.0         | 11                  | 1.0         |

When examining the data at failure it was also noticed that the elements GB1 and GB2 had a much greater ductility when compared to the conventional beam. The data are presented in table 2. Since ductility of a member is customary expressed in the ratio between the deflection at ultimate \( \delta_u \) and the yield deflection \( \delta_y \), the ductility can be calculated. The GB specimens had ductility coefficient of 1.3 times the conventional sections.
Table 2. Ductility behavior.

| Specimen | $\delta_y$ (mm) | $\delta_u$ (mm) | $\mu_S$ |
|----------|-----------------|-----------------|---------|
| GB1      | 11              | 58              | 5.3     |
| GB2      | 10              | 52              | 5.2     |
| NB        | 7               | 27              | 3.9     |

The moment-curvature data underlined the conclusions obtained from the moment-displacement curves. The specimens GB had a much stiffer section behavior as compared to the conventional concrete elements NB. These conventional concrete beams also demonstrated a clear yield plateau prior to failure of the element. The first cracking of the members was distinguished by an unmistakably increment in curvature.

3.2 Failure mechanism

The conventional specimens NB failed due to extensive cracking in the tension area of the member. The cracks were initiated at the extreme concrete fibers in tension, and propagated vertically towards the neutral axes. At advanced loading stages, the cracks multiplied along the area in between the loading points, and widened significantly. No shear failure was detected, and prior to yielding, the concrete in compression collapsed due to crushing of the material (figure 6).

Figure 6. Failure mechanism of controlling elements.

![Failure mechanism of controlling elements.](image)

Figure 7. Bond-slip mechanism. [4]

The failure mechanism of the steel-decks beams differed from the failure mode of the conventional beams. The most distinguishing factor was the bond deterioration between the steel-decks and the concrete. The shear stresses disparities in the interface of the corrugated sheet and the concrete were carried by the shear connectors, situated at a distance of 160 mm apart (figure 7).

There were no shear connectors place at the side-panels, and the loss in bond resulted in a decrease in moment inertia of the section. The corrugated sheet acted as additional tensile reinforcement, and the capacity of the members increased accordingly [4,5]. To evaluate the failure behavior of the GB specimens, the formwork was stripped and the members visually inspected (figure 8). The crack pattern deviates from the pattern of the conventional beams NB. All cracks were situated beneath the point of impact of the load, suggesting that the concrete in this area suffered extensive bearing stresses that...
weaken the section. A decrease in moment of inertia then initiated the propagation of tensile cracks towards the neutral axis.

**Figure 8.** Failure mechanism of steel-decks specimens.

When the steel bars in tension yielded, cracks in the concrete widened, and the shear stresses in the interface reached a maximum [6]. The shear connectors in this area at a distance of 1/3 of the span from the support collapsed, and the beam failed. Close examination to the shear connectors underlined this outcome. The shear connector in the mid-section were straight while the ones in the high shear stress area were bent outward to the direction of the beam (figure 9).

**Figure 9.** Shear connector failure mode

### 4. Conclusion and remarks

The steel-deck formwork has an advantage in that it provides a speed and clean construction environment. The formwork doesn’t require dismantling so that the structure can be directly used upon reaching of the concrete design strength. The system also doesn’t demand scaffoldings so that even during construction work, the space underneath the beam can be used.

The corrugated sheet when utilized with shear connectors, contribute to the composite action of the section. In positive bending mode, this sheet provides additional tensile reinforcement to the beam. Test results demonstrated a 2.3 times ultimate moment capacity increase when compared to an identical conventional concrete member with the exact same material properties and reinforcing configurations.

The initial stiffness of the member was enhanced significantly, thus grating a better serviceability level at service loading conditions. At ultimate, the steel-decks formwork element exhibits a much higher ductility ratio compared to the conventional beam.

The performance of shear connectors is determining to the composite action between the steel-decks and the concrete. Care should be taken in designing of these shear connectors especially in the areas
subjected to high shear forces. Also, the side-panels do not contribute to the load carrying capacity of the member, so in design these sheets can be reduced in thickness. However, the bond between these side-panels and the concrete should be improved to ensure the high stiffness of the member under service loading conditions. These panel should also be sufficiently stiff to guarantee the final dimensions, so that no deformations occur.

5. References
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