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# VARIABLES AFFECTING THE SHEAR-BOND RESISTANCE OF COMPOSITE FLOOR DECK SYSTEMS

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## ABSTRACT

The 1½″ composite deck section is among the more popular floor systems used in the construction of steel buildings in North America. The shear-bond between the steel deck and the concrete normally controls the capacity of a composite floor slab. Shear-bond can, for the most part, be attributed to the presence of mechanical interlock that results from the use of embossments formed in the deck webs during the rolling process. However, the extent of shear resistance between the concrete and the steel can also vary depending on the deck profile, steel thickness and grade, coating, as well as the deck position, *i.e.* normal or inverted. In addition, the curing time of the concrete may influence the shear resistance of the composite slab. This paper describes the results of two research projects in which the effect of some of these variables on shear-bond capacity was evaluated.

## INTRODUCTION

The P-3606 (P-3615 in Canada) composite deck is one of the more popular products manufactured by the Canam Manac Group for the construction of floors in steel buildings. The deck panels are 1½″ (38 mm) deep with a nominal width of 36" (914 mm) and with flutes spaced at 6" (152 mm) on centre (Fig. 1). Deck panels are typically available in the following nominal thicknesses: 0.030" (0.76 mm), 0.036" (0.91 mm), 0.048" (1.21 mm), and with the following coatings: Z275, Z180 and ZF75 zinc, as well as paint or primer.

In design, the composite floor slab is assumed to act as a simply supported element spanning up to approximately 8 feet (2.4 m) between the steel frame members in the direction parallel to the deck flutes. In most scenarios normal weight concrete is poured directly onto the deck, which first acts as a form in bending, and eventually becomes the tension reinforcement for the slab when the concrete hardens. The typical slab thickness ranges between 3.5" (90 mm) and 6.5" (165 mm). Shear-bond between the deck and the concrete is provided through indentations or embossments rolled into the deck webs. In the P-3615 and P-3606 decks, the embossments protrude into the concrete side of the sheet when the deck is installed in the upright position. Friction and adhesion (chemical bond) between the concrete and the steel may also contribute to the composite action between the two elements, although these two mechanisms are generally ignored in calculations *(CSSBI, 1988b; ASCE, 1991)*.

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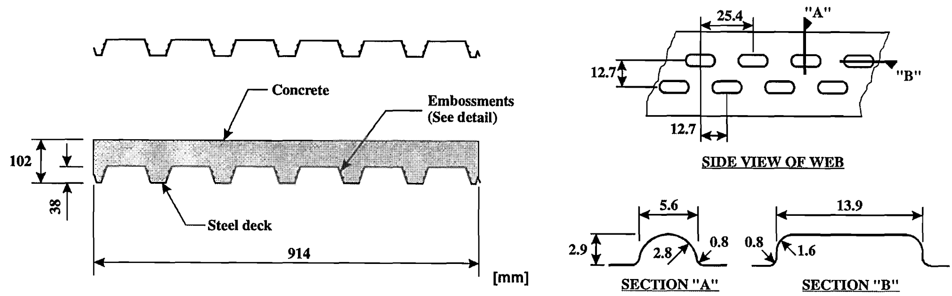



Fig. 1: Typical Composite Deck Cross-Section (102 mm slab shown with deck in upright position)

The characteristics of the steel deck, which may vary from one project to another, are the deck profile, the steel thickness, the steel finish or coating, the steel grade, and the deck position (normal or inverted). The deck can also be supplied without any indentations for roofs or with small circular holes in the web segments for acoustical purposes. The properties of the concrete slab may also vary: thickness of the slab, the density of the concrete, the compressive strength of the concrete, and the use of reinforcement.

Failure of longitudinally unrestrained composite floor slab specimens subjected to gradually incremented gravity load generally occurs by longitudinal shear-bond failure. Design values associated with this failure mode can be obtained from a standardised full-scale two-point load bending test program *(CSSBI, 1988a; ASCE, 1991).* In lieu of performing standard two-point load bending tests to determine shear bond characteristics, pullout or pushoff tests have also been considered in the past *(Cheng et al., 1994; Daniels, 1988; Patrick and Bridge, 1994; Seleim and Schuster, 1982; Stark, 1978; Wright and Veljkovic, 1996; Schuurman and Stark, 2000; Veljkovic, 2000).* No pullout test procedure has yet been standardised for the determination of the shear-bond capacity for flexural members; nevertheless, this test approach was initially seen as more effective for comparing the bond capacity of various configurations of the same steel deck profile.

### *Objectives and Scope*

The main objective of this study was to evaluate the shear bond capacity of the P-3606 (P-3615) deck system. The scope of testing consisted of the following:

- Steel thickness (0.76 mm, 0.91 mm, and 1.21 mm)
- Steel grade (ASTM A653 Grade 230, and ASTM A366 with $F_y = 550$ MPa)
- Surface coating (ZF75, Z180, Z275, painted, pre-painted series 8000)
- Position of the deck (normal or inverted)
- Curing age of the concrete (1, 2, 4, 7, and > 28 days)
- Presence of electrical conduits in the slab

Note that the influence of the steel grade and the coating were studied in combination because some steel grades were only available in certain finishes. The findings provided in this paper were based on a total of 26 two-point bending slab tests and 50 pullout tests.

## TEST PROCEDURE AND SPECIMENS

All tests were performed in the Structural Engineering Laboratory at École Polytechnique of Montreal. Two different test procedures were used in the research projects: two-point bending tests on full-scale slab specimens and pullout tests on small-scale specimens. Ancillary testing was also conducted to determine the properties of the materials used in the fabrication of the specimens.

### *Flexural Slab Tests*

The CSSBI S2-85 *(1988a)* and ANSI/ASCE 3-91 *(1991)* documents contain provisions for the testing of composite slabs using a two-point bending procedure (Fig. 2). The results obtained from such a test program can be used in design to determine the shear-bond resistance of a composite slab *(CSSBI, 1988b; ASCE, 1991)*. A series of bending tests was performed for each deck profile, surface coating, embossment pattern, and concrete type. In the research project, the shear bond parameters $k_5$ and $k_6$ were determined using a test series of four slab specimens with only one steel thickness. In all cases, the slab thickness and shear span of the specimens were selected such that two specimens gave maximum shear strength and two specimens were at minimum shear strength as required by the CSSBI S2-85 and ANSI/ASCE 3-91 standards.

$$V_t = bd\left[\frac{k_5}{L'} + k_6\right] \quad (1)$$

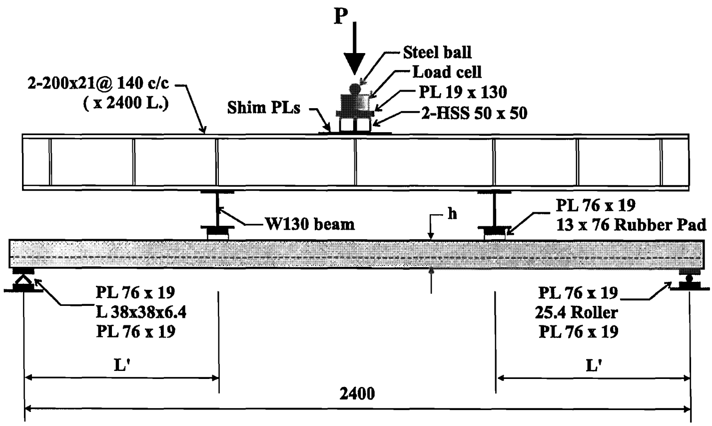



Fig. 2: Bending Test Set-Up (P-3606 shown)

Test specimens were 2500 mm (P-3615) or 2600 mm (P-3606) long with a nominal width of 914 mm (P-3615) or 950 mm (P-3606). The centre-to-centre span of the specimens was 2400 mm and the shear span, L', was varied. Reaction support was provided by 76 × 19 × 914 mm steel plates that distributed the load at opposite ends of the span. A two-point load system was used where forces were transferred to the slab by means of W-section beams as shown in Fig. 2. A 76 × 19 × 914 mm steel plate and neoprene pad were placed between the slab cross beams so that even loading of the specimen was achieved. The load was applied using a displacementcontrolled 250 kN actuator. The instrumentation used for the test set-up included a 200 kN capacity load cell, two displacement transducers located on either side of the slab at mid-span to measure the deflection, as well as a displacement transducer at either end of the specimen to measure relative slip between the steel deck and concrete.

Table 1: Nominal Bending Test Specimen Information

| *Specimen Number* | *Steel Thickness (mm)* | *Coating* | *Position of Deck* | *f'c*[1] *(MPa)* | *Shear Span L' (mm)* | *Slab Thickness (mm)* |
|---|---|---|---|---|---|---|
| P3606-22-N-850 | 0.76[2] | Z180 | Normal | 26.6 | 850 | 90 |
| P3606-22-N-700 | 0.76 | Z180 | Normal | 26.2 | 700 | 115 |
| P3606-22-N-550 | 0.76 | Z180 | Normal | 26.6 | 550 | 140 |
| P3606-22-N-400 | 0.76 | Z180 | Normal | 26.6 | 400 | 165 |
| P3606-18-N-850 | 1.21[3] | Z275 | Normal | 26.6 | 850 | 90 |
| P3606-18-N-700 | 1.21 | Z275 | Normal | 26.2 | 700 | 115 |
| P3606-18-N-550 | 1.21 | Z275 | Normal | 26.6 | 550 | 140 |
| P3606-18-N-400 | 1.21 | Z275 | Normal | 26.6 | 400 | 165 |
| P3606-22-I-850 | 0.76[2] | Z180 | Inverted | 26.6 | 850 | 90 |
| P3606-22-I-700 | 0.76 | Z180 | Inverted | 26.2 | 700 | 115 |
| P3606-22-I-550 | 0.76 | Z180 | Inverted | 26.6 | 550 | 140 |
| P3606-22-I-400 | 0.76 | Z180 | Inverted | 26.2 | 400 | 165 |
| P3606-18-I-850 | 1.21[3] | Z275 | Inverted | 26.6 | 850 | 90 |
| P3606-18-I-700 | 1.21 | Z275 | Inverted | 26.2 | 700 | 115 |
| P3606-18-I-550 | 1.21 | Z275 | Inverted | 26.6 | 550 | 140 |
| P3606-18-I-400 | 1.21 | Z275 | Inverted | 26.6 | 400 | 165 |
| P3615-22-N-600a | 0.76[4] | ZF75 | Normal | 34.6 | 600 | 102 |
| P3615-22-N-750 | 0.76 | ZF75 | Normal | 34.6 | 750 | 102 |
| P3615-22-N-400a | 0.76 | ZF75 | Normal | 34.6 | 400 | 127 |
| P3615-22-N-500 | 0.76 | ZF75 | Normal | 34.6 | 500 | 127 |
| P3615-22-N-400b[5] | 0.76 | ZF75 | Normal | 21.6 | 400 | 102 |
| P3615-22-N-600b[5] | 0.76 | ZF75 | Normal | 21.6 | 600 | 102 |
| P3615-22-I-600 | 0.76[4] | ZF75 | Inverted | 21.6 | 600 | 102 |
| P3615-22-I-750 | 0.76 | ZF75 | Inverted | 21.6 | 750 | 102 |
| P3615-22-I-400 | 0.76 | ZF75 | Inverted | 21.6 | 400 | 127 |
| P3615-22-I-500 | 0.76 | ZF75 | Inverted | 21.6 | 500 | 127 |

[1]Concrete strength on day of testing  [2]$F_y$ = 326 MPa $F_u$ = 404 MPa $\varepsilon_{50}$ = 29%
[3]$F_y$ = 299 MPa $F_u$ = 385 MPa $\varepsilon_{50}$ = 32%  [4]$F_y$ = 329 MPa $F_u$ = 379 MPa $\varepsilon_{50}$ = 38%
[5]Electrical PVC conduits (25.4 mm) in slab (transverse to flutes at 300 mm from ends and parallel to middle and two edge flutes

All steel deck panels were produced and cut to length by Canam Manac, with the composite specimens cast in the horizontal position while the deck was supported throughout on the floor, as specified in the CSSBI S2-85 provisions. No steel welded wire fabric nor any other reinforcing steel was included in the slab. Prior to pouring of the concrete, the surface of the deck was cleaned with an acetone-based solvent. The concrete was placed manually and vibration was performed using a needle-type vibrator. The top surface of the slab was finished using a metal trowel, and no curing or sealing agent was used. The specimens were air cured at room conditions until testing. The P-3615 specimens were cast in the laboratory and formwork was removed after one day, whereas the P-3606 specimens were cast at the Canam Manac plant in Boucherville Canada, and then after the concrete had cured were shipped to the university still in their formwork. A listing of the test specimens can be found in Table 1.

Two tests were completed to study the influence on the shear bond capacity of electrical conduits in the slab. A total of five 25.4 mm o.d. PVC conduits were placed in each specimen (Fig. 3). A full-length conduit running parallel to the deck was placed in the central and outer flutes, and full width conduits were placed perpendicular to the flutes at 300 mm from either end of the specimen.

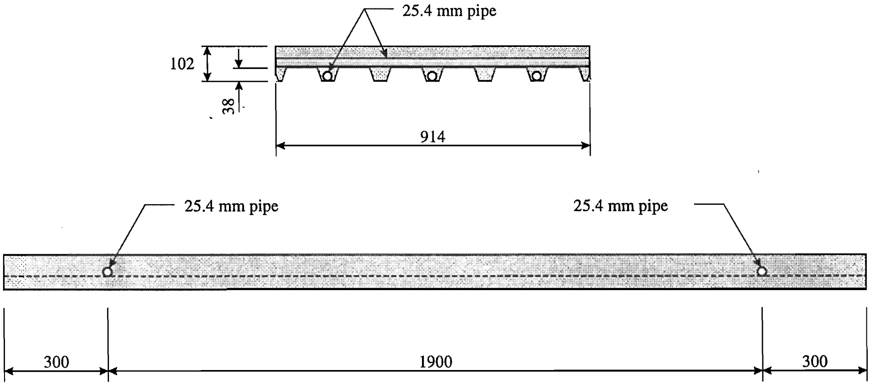



Fig. 3. Position of the Electrical Conduits in Test Specimens

### *Pullout Tests*

Pullout tests were used in the study of P-3615 deck specimens to examine the influence of various steel deck characteristics on the shear bond capacity at the steel-concrete interface. No standardised pullout test procedure has yet been established for the purpose of obtaining shear-bond coefficients for use in determining the flexural capacity of a composite slab. In this study, it was decided to use a pullout test procedure that replicates as closely as possible the conditions that prevail in the two-point bending tests. This approach was followed under the assumption that the findings of the pullout tests could be applied in predicting the flexural behaviour of composite slabs.

A schematic drawing of the test set-up is provided in Fig. 4. Each test specimen consisted of a concrete block sandwiched between two steel deck sections to create a symmetrical loading condition. The deck sections were placed on the outside of the test specimen to contain and to facilitate pouring of the concrete. The specimen was tested horizontally with the longitudinal loading system selected to best reproduce the conditions in an actual composite slab in bending. The deck was subjected to uniform tension applied at its centroid, whereas the concrete was loaded in compression, reproducing the typical stress situation found in bending members. A vertical load was applied to the specimen end by means of a 6 kN concrete mass, similar to the load used by Wright and Veljkovic *(1996),* to provide containment and to reproduce the end conditions of a typical simply supported slab. In comparison with the bending tests, the 6 kN load corresponds to an end reaction of 18.3 kN (= 6 kN $\times$ 914 mm / 300 mm) per unit deck panel width.

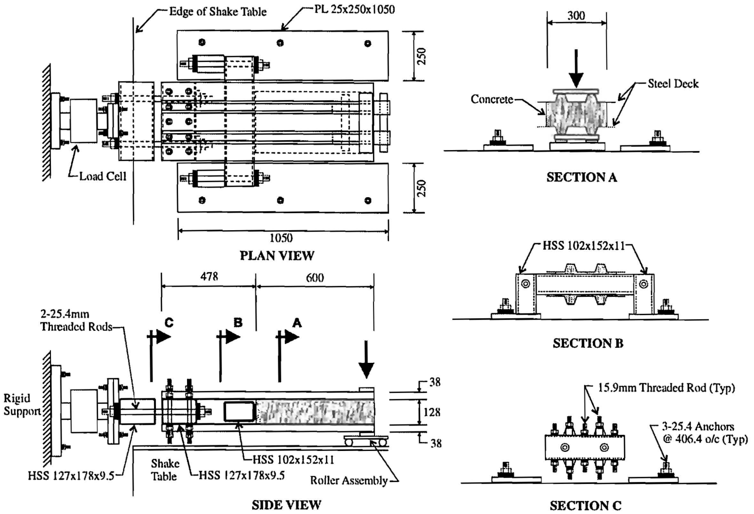



Fig. 4: Pullout Test Set-Up

A 102×152×11 HSS section formed part of a frame assembly that was bolted to the floor of an MTS shake table. The deck sections were attached to another HSS system that transferred load through a load cell to a rigid support. Displacement of the shake table caused the HSS section to bear against the concrete creating shear forces between the concrete and steel deck. A 300 mm wide concrete section was used in order to include a total of four webs of each deck so that an average shear distribution could be obtained. The 600 mm length was chosen to represent the shear span found for the upper range 2.4 m design span used for the 38 mm deck. The 128 mm thick concrete section corresponds to twice the thickness of the concrete cover found for the common 102 mm (4") (total depth) composite deck slab. Data was recorded by means of a 250 kN load cell attached to the rigid frame, and three displacement transducers. The relative slip between the concrete and steel deck was measured for both the top and bottom of the test specimen. Overall displacement between the rigid support and the table was also monitored and used for displacement control of the shake table.

The deck was fabricated and cut to size by Canam Manac and shipped to École Polytechnique for assembly and testing. Formwork was provided by means of channel spacers bolted to either side of the deck sections. The specimens were secured in the vertical position with the tension-loaded end of the deck pointing downward. A plywood insert was cut to fit the interior dimensions of the specimen to create a smooth bearing surface. Concrete was poured for all specimens simultaneously and vibrated accordingly. The specimens were left at room conditions and allowed to air cure. The channel spacer sections were kept in place during installation of the test specimens and then removed prior to testing. It must be noted that the direction of rolling for the deck could not be identified, and therefore is was not possible to ensure that this characteristic was the same for all test specimens. The embossments are not entirely symmetric, and hence their mechanical shear bond resistance when loaded in the rolling direction may differ from that when loaded in the opposite direction.

The influence of the curing age of the concrete was examined through analysis of the ZF75 coated 0.91 mm deck specimens. Duplicate pullout tests and cylinder tests were carried out at 1, 2, 4, 7, and ≈ 55 days. A listing of all pullout tests and material properties can be found in Tables 2-3. The steel deck units that were used to fabricate the specimens were samples taken out of the regular production sequence at the Canam Manac plant in Boucherville, Canada. All ZF75, Z275 coated, and series 8000 sheet steels were specified as ASTM A653 Grade 230 *(1994)* products, whereas the painted sheet steels fall under ASTM A366 *(1993)* with $F_y = 550$ MPa.

Table 2: Pullout Test Specimen Information

| ***Specimen Number.*** | ***Steel deck*** | | | | ***Concrete curing (days)*** | ***f'c[1] (MPa)*** | ***Coil # or Requisition #*** |
|---|---|---|---|---|---|---|---|
| | ***t (mm)*** | ***Coating*** | ***Pos.*** | ***Web*** | | | |
| 3.1a/3.1b | 0.76 | ZF75 | N[a] | E[c] | ≥ 28 | 34.8 | 44854 / 44854 |
| 3.2a/3.2b | | | | F[d] | ≥ 28 | 34.8 | 44854 / 44854 |
| 3.3a/3.3b | | | I[b] | E | ≥ 28 | 34.8 | 287961 / 287961 |
| 3.4a/3.4b | | Z275 | N | E | ≥ 28 | 34.8 | 287960 / 45016 |
| 3.5a/3.5b | | | | F | ≥ 28 | 34.8 | 287962 / 287962 |
| 3.6a/3.6b | | | I | E | ≥ 28 | 34.8 | 45016 / 287960 |
| 3.15a/3.15b | 0.91 | ZF75 | I | E | ≥ 28 | 34.8 | 44798 / 44798 |
| 3.20a/3.20b | | Z275 | N | F | ≥ 28 | 21.4/34.8 | 44846 / 44846 |
| 3.21a/3.21b | | | I | E | ≥ 28 | 34.8 | 44846 / 44846 |
| 3.24a/3.24b | 1.21 | ZF75 | N | E | ≥ 28 | 34.8 | 287969 / 287969 |
| 3.25a/3.25b | | | I | E | ≥ 28 | 34.8 | 287969 / 287969 |
| 3.26a/3.26b | | Z275 | N | E | ≥ 28 | 34.8 | 287968 / 287968 |
| 3.27a/3.27b | | | I | E | ≥ 28 | 34.8 | 287968 / 44345 |
| 3.7a/3.7b | 0.76 | 8000 | N | E | ≥ 28 | 21.4 | 289173 / 289173 |
| 3.8a/3.8b | | Painted | N | E | ≥ 28 | 21.4 | 44915 / 45571 |
| 3.9a/3.9b | 0.91 | ZF75 | N | E | 1 | 5.6 | 287965 / 287965 |
| 3.10a/3.10b | | | | | 2 | 11.1 | 287965 / 44798 |
| 3.11a/3.11b | | | | | 4 | 14.8 | 287965 / 287965 |
| 3.12a/3.12b | | | | | 7 | 18.0 | 287965 / 287965 |
| 3.13a/3.13b | | | | | ≥ 28 | 21.4 | 287965 / 287965 |
| 3.14a/3.14b | | | | F | ≥ 28 | 21.4 | 44798 / 44798 |
| 3.19a/3.19b | | Z275 | | E | ≥ 28 | 21.4/34.8 | 45025 / 45373 |
| 3.22a/3.22b | | 8000 | | | ≥ 28 | 21.4 | 289174 / 289174 |
| 3.23a/3.23b | | Painted | | | ≥ 28 | 21.4 | 45209 / 45209 |
| 3.28a/3.28b | 1.21 | Painted | | | ≥ 28 | 21.4 | 45247 / 45247 |

[1]Concrete strength on day of testing
[a] Normal deck position [b] Inverted deck position [c] Embossments in web [d] No embossments in web

Table 3: Coil Material Properties

| *Coil #* | *Coating* | *t (mm)* | *$F_y$ (MPa)* | *$F_u$ (MPa)* | *$\varepsilon_{50}$ (%)* |
|---|---|---|---|---|---|
| 44345 | Z275 | 1.21 | 330 | 350 | 35 |
| 44798 | ZF75 | 0.91 | 343 | 400 | 35.9 |
| 44846 | Z275 | 0.91 | 314 | 375 | 36.7 |
| 44854 | ZF75 | 0.76 | 258 | 324 | 40.9 |
| 44915 | Painted | 0.76 | 641 | 708 | na |
| 45016 | Z275 | 0.76 | 337 | 399 | 29.6 |
| 45025 | Z275 | 0.91 | 319 | 376 | 33.5 |
| 45209 | Painted | 0.91 | 750 | 776 | 2.9 |
| 45247 | Painted | 1.21 | 646 | 684 | 5.0 |
| 45373 | Z275 | 0.91 | 301 | 388 | 28.6 |
| 45571 | Painted | 0.76 | 688 | 709 | 2.7 |
| *Requisition #* | *Coating* | *t (mm)* | *$F_y$ (MPa)* | *$F_u$ (MPa)* | *$\varepsilon_{50}$ (%)* |
| 287960 | Z275 | 0.76 | 309 | 389 | 31.6 |
| 287961 | ZF75 | 0.76 | 329 | 379 | 38.2 |
| 287962 | Z275 | 0.76 | 318 | 405 | 28.1 |
| 287965 | ZF75 | 0.91 | 279 | 362 | 35.8 |
| 287968 | Z275 | 1.21 | 285 | 372 | 33.8 |
| 287969 | ZF75 | 1.21 | 322 | 383 | 38.6 |
| 289173 | 8000 | 0.76 | 309 | 371 | 28.0 |
| 289174 | 8000 | 0.91 | 290 | 375 | 30.3 |

## TEST RESULTS

### *Flexural Slab Tests*

In all cases the load increased gradually up to a point where shear bond failure (longitudinal slip) occurred suddenly on one side of the specimen. Upon slippage, the load dropped rapidly to a lower level. At this point two general types of behaviour were observed. The P-3615 and the 0.76 mm P-3606 tests remained at a constant lower load level until completion of the test. In contrast, the 1.21 mm P-3606 specimens regained capacity after the initial loss of chemical bond between the concrete and steel, to a level in excess of the initial slip load, due to the mechanical interlock between the deck embossments and the concrete. This increase in capacity can be attributed to the thicker sheet steel which improved the stiffness of the embossments, and hence the shear bond between the concrete and the steel. Shear cracks in the slab occurred at failure for all specimens and diagonal fracture of the concrete, as well as debonding of the concrete-deck interface was observed on one end of each specimen only. The two specimens that contained PVC conduits exhibited similar failure modes in comparison to other slab specimens, although additional cracks developed at the transverse pipe locations.

Shear-bond coefficient values, $k_5$ and $k_6$, were determined following the CSSBI "Criteria for the Testing of Composite Slabs" *(1988a)* requirements. Experimental values of $V_t$ / bd are plotted with respect to the inverse of the shear span (1/L') for the 0.76 mm P-3615 deck in the normal position (Fig. 5). The additional test specimens that contained PVC conduits where also included in this comparison graph. The test results indicate that the presence of conduits did not have a measurable effect on the shear-bond capacity when the shear span was long (600 mm), however, for specimen P3615-22-N-400b (400 mm shear span) the resulting capacity was significantly below the 15% error band, *i.e.* an approximate decrease of 30% was observed. Hence, the presence of conduits in the high shear force zone of a composite slab is expected to have an impact on the overall flexural capacity.

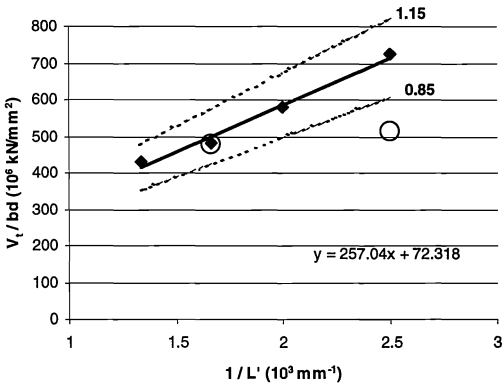



Fig. 5: Shear-Bond Coefficients for 0.76 mm Normal Position P-3615 Composite Flexural Deck Specimens (slabs with pipes also shown: o).

A comparison of the shear-bond capacity of the various slab test specimens when placed in the normal or inverted position was also carried out (Fig. 6). Each line shown in the figure was determined through the linear regression of four tests per slab type, which included specimens with varying shear spans and concrete thickness. The results shown for the 0.76 mm P-3606 slab specimens are significantly lower than those measured for the P-3615 tests. This is mainly attributed to the different coatings that were used (see Table 1), where a more adhesive wipecoat finish (ZF75) was specified for the P-3615 deck panels.

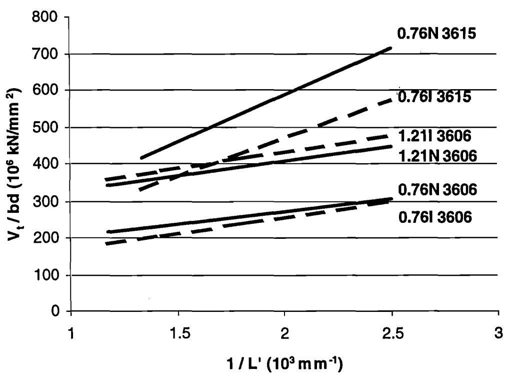


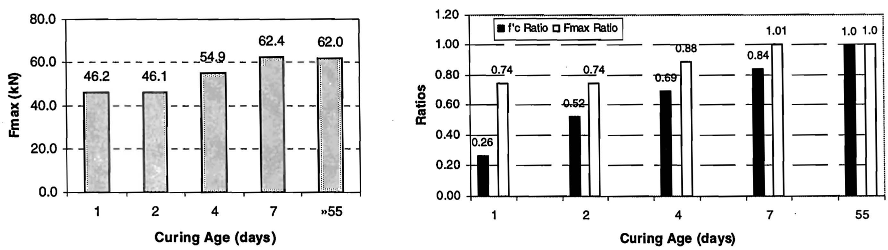

Fig. 6: Comparison of Shear Bond Coefficients for Normal and Inverted Position P-3606 and P-3615 Composite Flexural Deck Specimens

The 0.76N P-3615 (normal position) and 0.76I P-3615 (inverted position) deck bending test results indicate that the shear capacity is reduced when the deck is placed in the inverted position. However, the extent of this reduction for these particular tests can be explained by the lower concrete strength for the inverted deck specimens, *i.e.* 21.6 *vs.* 34.6 MPa. A reduction in shear capacity was also measured for the 0.76 mm P-3606 specimens, although only a minor decrease occurred in this case. In contrast the thicker 1.21 mm P-3606 specimens exhibited a higher shear bond capacity in the inverted position. The concrete strengths for the normal and inverted P-3606 specimens were similar.

The influence of the deck thickness on bond can be examined with the P-3606 specimens. The thicker 1.21 mm deck sections were able to carry higher shear forces due to the higher stiffness of both the embossments and the web elements, which resulted in relatively higher shear bond due to mechanical interlock. In addition, increased longitudinal friction forces developed between the 1.21 mm deck and the concrete near the supports as a result of the significantly higher end reactions in comparison with the 0.76 mm deck. The same behaviour was observed for both the normal and inverted positions, which indicates that the orientation of the profile had a limited affect on the overall performance.

### *Pullout Tests*

Pullout tests were completed for normal position 0.91 mm P-3615 deck specimens with curing ages of 1, 2, 4, 7, and ≈ 55 days, with $F_{max}$ results presented in Fig. 7(a). Note: $F_{max}$ is the maximum total shear load measured for the pullout tests. The shear bond capacity after only 1 day was approximately 74% of that measured after the concrete had reached its full strength. Tests run at 4 days showed that nearly 88% of the ultimate shear bond capacity could be realised and attainment of the full shear bond capacity took only 7 days. A comparison of the increase in concrete strength and shear bond capacity with curing time is shown in Fig. 5(b), where both resistance values are normalised with respect to the 55-day values. In general, the shear bond resistance was not overly influenced by the concrete strength.




(a) (b)

Fig. 7: Influence of Curing Age of the Concrete on $F_{max}$ for 0.91 mm Normal Embossed Deck

A comparison of the ultimate shear bond capacity of ZF75 and Z275 0.76-1.21 mm P-3615 pullout specimens with the deck thickness was carried out for normal and inverted decks. A direct correlation between an increase in steel thickness and a resulting increase in the shear bond of the Z275 normal and inverted embossed specimens was observed (Figs. 9 & 11). Embossed

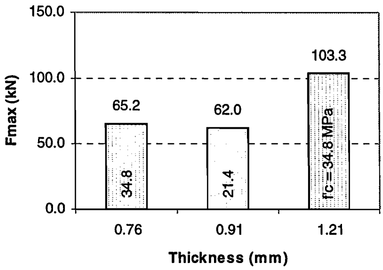



Fig. 8: $F_{max}$ *vs.* t ZF75 Normal P-3615 Deck

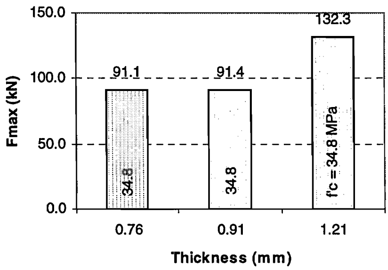



Fig. 10: $F_{max}$ *vs.* t ZF75 Inverse P-3615 Deck

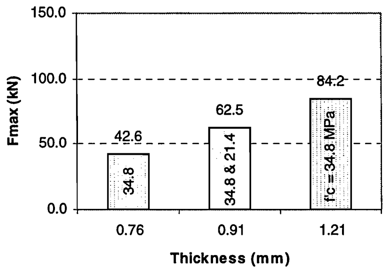



Fig. 9: $F_{max}$ *vs.* t Z275 Normal P-3615 Deck

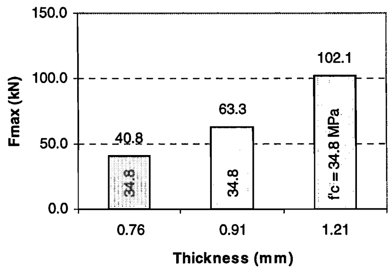



Fig.11: $F_{max}$ *vs.* t Z275 Inverse P-3615 Deck

decks must deform to allow separation between the concrete and the steel, and hence thicker steel provides a greater resistance to such deformation and thereby to shear bond failure.

This was also the case for the ZF75 deck specimens, except for the 0.91 mm deck sections where the $F_{max}$ values either decreased slightly or remained relatively even with those measured for the 0.76 mm decks (Figs. 8 & 10). It is possible that the relationship between thickness and $F_{max}$ was affected by the use of test specimens from different manufacturers, thus the surface coating although meeting the same specifications, may have been slightly different, hence lowering the bond between steel and concrete. Sorevco manufactured all of the Z275 specimens (except test 3.27B), whereas the ZF75 deck sections were fabricated by Dofasco, except for the 0.91 mm normal embossed specimens that were produced by Sorevco. This change in manufacturer may have resulted in the 62.0 kN ultimate strength value recorded for the 0.91 mm ZF75 specimens (Fig. 8).

The pullout deck specimens that were received from Canam Manac had various coatings used to protect the base metal of the profile, *i.e.* ZF75, Z275, Painted and Series 8000. A comparison of the influence of surface coating on the shear bond of composite deck sections was completed for the 0.76, 0.91 and 1.21 mm normal position P-3615 deck pullout specimens (Figs. 12-14). In most instances the highest shear bond was realised by the ZF75 coated steels followed by the Z275 decks. Use of the painted and series 8000 coatings reduced the bond between the composite deck and the concrete slab.

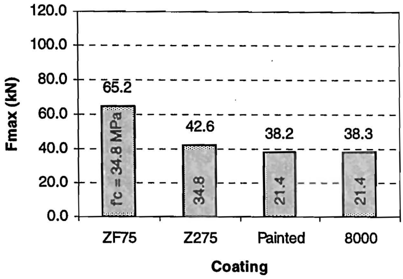



Fig. 12: $F_{max}$ *vs.* Coating 0.76 mm Normal Embossed Deck

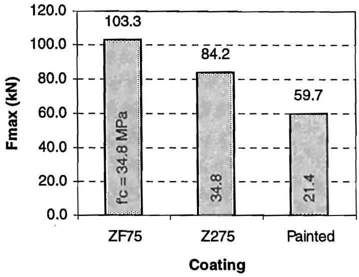



Fig. 14: $F_{max}$ *vs.* Coating for 1.21 mm Normal Embossed Deck

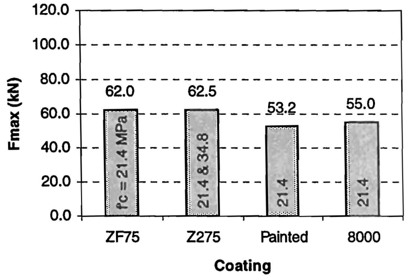



Fig. 13: $F_{max}$ *vs.* Coating for 0.91 mm Normal Embossed Deck

Pullout tests were run with 0.76 to 1.21 mm thick ZF75 and Z275 P-3615 decks to assess the influence of placing the steel profile in the normal or inverted position (Figs. 15 & 16). The ZF75 specimens showed that shear bond increased for all thicknesses when the deck was placed in the inverted position. The overall average increase in shear bond strength between the normal and inverse positions was 38%. However, ultimate values for the Z275 decks did not increase noticeably for the inverted decks except for the 1.21 mm thick specimens.

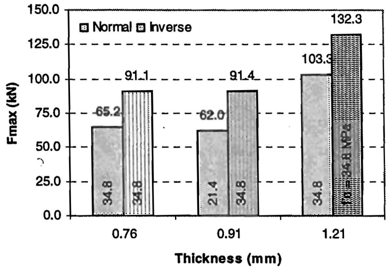



Fig. 15: $F_{max}$ *vs.* Deck Position for ZF75 Embossed Deck

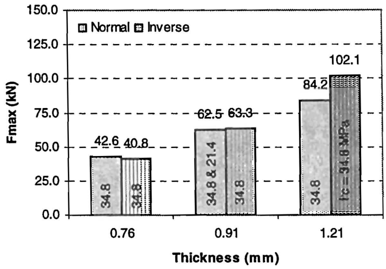



Fig. 16: $F_{max}$ *vs.* Deck Position for Z275 Embossed Deck

The influence of mechanical bond was studied by comparing the pullout test results of embossed and flat decks for the ZF75 and Z275 specimens in the normal position (Figs. 17 and 18). The flat ZF75 specimens showed 60 and 55% of the embossed resistance for the 0.76 and 0.91 mm decks, respectively. The reduction from embossed to flat section shear bond resistance was significantly more pronounced for the Z275 specimens: 28 and 19% for the 0.76 and 0.91 mm decks, respectively. The decrease in shear bond capacity from embossed to flat decks was more obvious for the Z275 specimens because the adhesive bond between the concrete and steel was not as effective as found for the ZF75 sections.

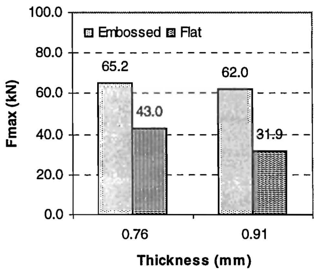



Fig. 17: $F_{max}$ *vs.* Mechanical Bonding for ZF75 Normal Deck

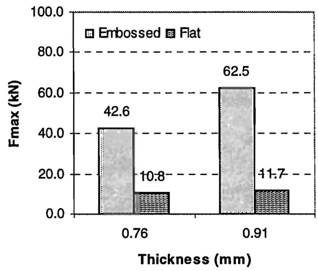



Fig. 18: $F_{max}$ *vs.* Mechanical Bonding for Z275 Normal Deck

## CONCLUSIONS

A total of 26 two-point bending slab tests and 50 pullout tests were carried out to evaluate the effect of steel thickness, surface coating, deck position, curing age of the concrete, and the presence of electrical conduits in the slab on the performance of composite floor deck systems. The results of the test program have shown that an increase in the thickness of the steel deck generally results in a higher overall capacity of the composite slab. The specimens with the ZF75 coating were found to exhibit the highest capacity, mainly because of a superior adhesive bond between the concrete and the deck, as confirmed by comparing pullout test results for embossed and flat deck profiles. It must be noted that adhesive bonding properties may vary with time and steel surface conditions, hence, care must be exercised when using such higher values in design. The use of the 8000 series and painted finishes generally resulted in comparable capacities, which were somewhat lower than those recorded for the Z275 specimens. Overall, when considering the slab and pullout tests, the bond capacity of the inverted deck specimens is usually equal to or greater than that measured for specimens fabricated with the deck in the normal position. Only for the 0.76 mm P-3615 slab tests did this observation not hold true, although this result may be attributed to the difference in concrete capacity between the normal and inverted deck specimens. With regards to slab capacity as a function of curing age; pullout test results reveal that a shear bond capacity equal to 74% of the 28 day capacity could be reached after one day of curing under room conditions and that the full shear bond capacity is present after only 7 days from the time of concrete pouring. Finally, the addition of PVC pipes in the shear span transverse to the deck flutes has a negative impact on the shear bond capacity for floor systems with short shear spans.

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