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SHEAR CAPACITY OF COMPOSITE SLAB REINFORCED WITH STEEL FIBRE CONCRETE TOPPING

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Abstract: This paper present the results of combined bending and shear test on composite slabs reinforced with steel fibres in-situ concrete topping. The mechanical properties of steel fibre reinforced concretes (SFRCs) were first determined by varying the fibre dosage from 0% to 1.25%. The result shows that there was not much improvement on the SFRC properties for volume fraction of steel fibre (V_f) of between 1.0% and 1.25%. Following this, SFRC with V_f = 1.0% was chosen and cast onto the precast slab. In addition to this, the top of the precast slab was prepared with four (4) different surface roughness i.e. smooth as-cast, roughened in the longitudinal and transverse direction, and exposed aggregate. The SFRC replaced the conventional method of using cast in-situ reinforced concrete as structural topping. The experimental results on its ultimate shear capacity were further compared with the parametric equation proposed in this paper using SFRC as concrete topping. The result shows that the ultimate shear capacity was 4% and 6% lower than the calculated value for the exposed aggregate and longitudinal roughened surface, respectively. For the smooth as-cast and transverse roughened surface, they were even 28% and 24% kN lower than the calculated value. Further comparison with previous researcher also found that the ultimate shear capacity for specimens with SFRC topping (except for the exposed aggregate surface) was 10% to 37% lower than the conventional (plain) ones. The finding also suggests that surface roughened in the longitudinal direction was better than the other surface textures using SFRC as topping in terms of interface bonding.

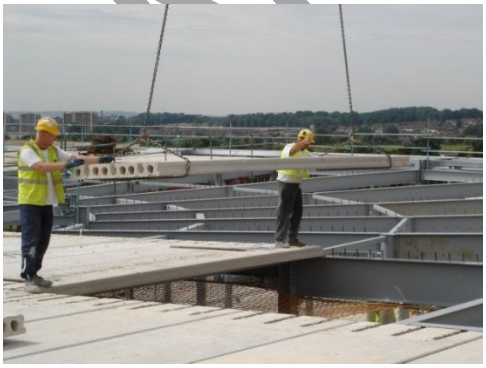
Keywords: *Steel fibre reinforced concrete; Concrete topping; Composite action; Ultimate shear capacity*

1.0 Introduction

Slab is one of the building elements that will indirectly affect the overall structural stability and performance of a building. Precast concrete slab such as hollow core unit is cast and cured in the factory under high quality control before transported to construction site and lifted into position with the aid of lifting equipment as shown in Fig. 1(a). Precast slab nowadays become more popular in the construction industry as they are provided in a wide range of slab types. Apart from that, they contribute many advantages including less working time, less labour and at the same time keep the site clean with minimal formwork.

In-situ reinforced concrete topping is usually cast onto the precast slab in order to enhance the structural performance of the slab system. The concrete topping is usually 50 – 75 mm thick reinforced with steel mesh as shown in Fig. 1(b) for diaphragm action and shrinkage control. When the two components are cast together, they will act compositely and under flexural load, they will bend monolithically prior to generate the sliding movement between each other at a particular load. Horizontal shear strength is developed at the interface in order to resist the sliding movement as illustrated in Fig. 2.

Nowadays, some modifications can be made by varying the materials used in the casting of concrete topping. This include the application of SFRC as structural topping which is possible in resulting better horizontal shear strength without the needs for additional shear links projecting from the top of the precast slab (Girhammar, 2008 and FIP, 1982). Also, SFRC is better in controlling shrinkage apart from the increase in concrete toughness, durability and energy absorption. Furthermore, the bridging effect in the concrete may possibility increase the slab bearing capacity using SFRC as concrete topping.



(a) Precast hollow core units are lifted and positioned on-site

(b) Steel mesh are placed on to the precast slab before the casting of concrete topping

Figure 1: The construction of precast building structure

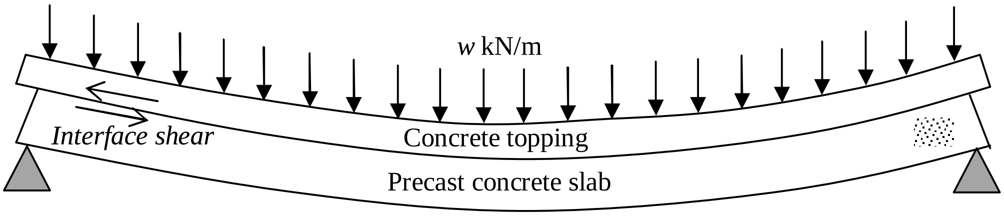


Figure 2: Interface shear acting between precast unit and concrete topping under flexural load

Study on SFRC by Roesler et. al. (2004) includes fracture behaviour of plain and fibre reinforced concrete slab. It was concluded that discrete fibres improved the load-deformation characteristics in comparison with plain concrete slab. The type of fibre (material, aspect ratio and geometry) and content were the main factors which increase the ultimate load-carrying capacity of the slabs. A year later, Khaloo & Afshari (2005) carry out experimental works on small SFRC slabs to study the flexural behaviour with varied fibre length (25 mm and 35 mm length), volume fraction (ratio of the volume of fibres to the volume of matrix between 0% and 1.5%) and concrete strength (cylindrical concrete strength of 30 MPa and 45 MPa at 28 days). They found that the rate of improvement in energy absorption reduced with the increase in fibre content ranging from 1.0% to 1.5%. Also, longer steel fibre with higher aspect ratio provides higher energy absorption for SFRC. They recommended that the addition of steel fibres in concrete must be within the volume fraction of between 0.75% and 1.75%. Later in 2007, Altun et. al. (2007) studied the effects of adding steel fibre on the mechanical properties of reinforced concrete (RC) beams. RC beams of grade C20 and C30 concretes were prepared with three (3) different steel fibre dosages of 0, 30 and 60 kg/m^3. They came out with some concluding remarks where steel fibre dosage of 30 kg/m^3 is better than 60 kg/m^3 in terms of concrete toughness, flexural strength, crack formation, crack size and crack propagation.

Usually, precast composite slab structure is designed without providing any shear links projecting from the top of the precast slab. This is because it is believed that interface shear strength can be carried up by the bonding force between the precast slab and in-situ concrete topping. This can be achieved by treating the surface to a certain texture and roughness. The top surface of the precast slab will be brushed either in the longitudinal or transverse direction, and exposed aggregate to create the required surface texture. However, most designers prefer the surface to be remained smooth as-cast and according to FIP (1982), they believe that smooth surface has better interface bond compared with the rough one. The reason is that in order to get the required rough surface, the top of the precast slab is usually raked using a stiff brush. This leaves the surface with concrete laitance after the raking process and without proper cleaning method the interface bond weakens under long term sustained load. If this occurs, the composite action is lost and can lead to catastrophic failure. Research on interface shear strength was carried out by Gohnert (2003) to clarify the lack of congruency on the horizontal shear strength between various Code of Practice; ACI 318, BS 8110, Eurocode 2 and SABS 0100. In conclusion, it was not recommended to specify the interface shear strength as a function of the concrete compressive strength due to the scatter of data. In fact, actual measurement of roughness should be specified as stated in Eurocode 2 rather than merely state the instrument used to create the undulations of the surface roughness. In 2008, a more extensive work on shear capacity was carried out by Girhammar and Pajari (2008) on composite slabs of hollow core units and SFRC concrete topping. The top surface of the hollow core units was untreated in anyways assuming smooth as-cast surface. They found that bonding strength at the interface shows the tensile/bond strength of the fibre reinforced concrete topping increased slightly compared with the plain ones. At the end of their study, they claimed that there was no improvement to the shear capacity of the composite section for fibre reinforced concrete topping since web shear failure was the governing failure in all cases.

It was also observed that optimum V_f must be in the range of between 0.75% and 2.0% (Altun et. al., 2007). Obviously, for V_f higher than 2.0%, it become ineffectively because of the physical difficulties in providing a homogenous distribution of steel fibres in the structural members as well as decrease in the compressive strength as compared with plain concrete (Altun et. al., 2007). As for V_f less than 1.0%, it also becomes ineffective due to the decrease in tensile and flexural strengths.

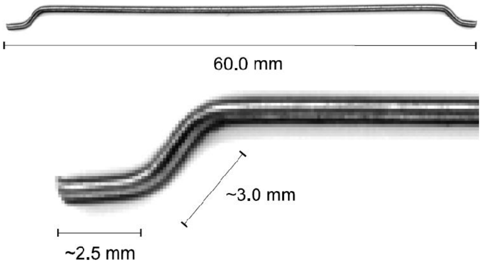
The current work is aimed at studying on (i) the mechanical properties of SFRC for lower fibre content of between 0% and 1.25%, and (ii) the ultimate shear capacity of composite slab with different surface roughness. Additionally, the experimental results were compared with the conventional cast in-situ concrete topping reinforced with steel mesh (Teck Yee, 2009). A parametric equation is also proposed in this paper to predict the shear capacity using SFRC as structural topping, taking into account the modular ratio of the two materials. This study was carried out to determine the suitability of SFRC as structural topping and a replacement to the conventional reinforced cast in-situ concrete. Furthermore, appropriate bonding without the needs for additional shear links can reduce maximum deflection and at the same time increase the capacity of composite slab using SFRC as concrete topping. The success of this research can reduce labour cost and time in the construction of precast building structure.

2.0 Research Methodology

First stage of the laboratory work is determining the mechanical properties of SFRC. This includes the compressive, tensile splitting and tensile flexural strengths and also the

Modulus of Elasticity in compression for normal weight concrete of grade C25. This includes; (1) tests on cube sample of 100 mm × 100 mm × 100 mm for compressive strength, (2) tests on cylindrical samples of 150 mm diameter × 300 mm long for splitting tensile strength and static Modulus of Elasticity in compression, and (3) tests on prism sample of 100 mm × 100 mm × 500 mm long for flexural strength. Altogether five concrete batches were prepared with five different volume fraction of steel fibre i.e. 0% (plain concrete as control specimen), 0.50%, 0.75%, 1.00% and 1.25% by absolute concrete weight. The type of steel fibre used in this study is a hooked-end type as shown in Fig. 3, where each fibre is 0.75 mm in diameter and 60 mm long. This gives an aspect ratio, *L/d* = 80.

(a) Steel fibres glued together in packing



(b) Detail dimension of the steel fibre

Figure 3: Hooked-end type steel fibres

The method for concrete mix design (plain concrete without any steel fibre) was in accordance with the design for normal concrete mixes (Teychenné et. al., 1988). All mix proportions of the raw materials were remained exactly the same for all five batches including the one with the added 0.50%, 0.75%, 1.00% and 1.25% steel fibre. Table 1 gives the proportions for plain concrete and SFRC mixes. During the mixing process, steel fibres were added last to the fresh concrete at a rate of 20 kg/min while the drum mixer rotates at high speed for 5 minutes (RILEM TC162-TDF, 2000).

Nine cubes, four cylinders and three beams were cast from each batch and compacted using a poker vibrator. All samples were water cured at a control temperature between 19°C and 21°C until the test day i.e. 7, 14 and 28 days. The method of testing in compression, splitting tensile and flexural strengths were in accordance to BS EN 12390: Part 3 (BSI, 2009), BS EN 12390: Part 6 (BSI, 2009), BS EN 12390: Part 5 (BSI, 2009), respectively. Apart from that, Modulus of Elasticity in compression was also determined where the specimen preparation and testing method follows BS 1881: Part 121 (BSI, 1983).

For the second stage of the laboratory work, the composite slabs specimens were prepared and cast in two stages; (i) casting of the precast slab, and (ii) casting of the

SFRC topping. The mix proportions for both elements are given in Table 2. In the first stage, precast slab of 500 mm wide × 1100 mm long × 100 mm deep was prepared and installed with four 12 mm diameter high tensile steel bars (4T12) as reinforcement. The concrete cover was fixed at 25 mm from the bottom surface. Concrete was then poured and compacted using poker vibrator. Cubes and cylinders were also prepared to determine the compressive strength, tensile splitting strength and the Modulus of Elasticity.

To study the effectiveness of the interface bond, the top of the precast slab was prepared with different surface roughness. This includes smooth as-cast, roughened in the longitudinal and transverse direction, and exposed aggregate. To create the roughened surface (both longitudinal and transverse direction), the top of the precast slab was raked using a stiff brush. Meanwhile, the exposed aggregate surface was treated after 28 days curing using a hammer hand tool. One specimen was untreated in any way (smooth by using trowel), as it is classify as smooth as-cast surface.

In the second stage, concrete topping were reinforced with 1.0% volume fraction of steel fibre (by absolute concrete weight). The SFRC topping was cast onto the precast slab after the precast slab achieved the design compressive strength of 40 N/mm^2 at 28 days. The SFRC topping was 500 mm wide × 1100 mm long × 75 mm deep. Before casting the SFRC topping, the top surface of the precast slab was wetted with the slab being light dark grey in colour with no standing surface water (FIP, 1982). All specimens were cured using wet burlap until the test day.

The specimens were tested under the combined bending and shear. The test setup is shown in Fig. 4. The specimens were simply supported and subjected to a pair of point load. A strain gauge was installed at the top surface of the precast slab (interface) and two pairs of Demec pips were fixed at the top and bottom of each specimen. They were installed at mid-span to measure the development of concrete strains. Mid-span deflection and interface slip was also measured using LVDTs as shown in Fig. 4. The testing method was divided into three stages; (i) prior to cracking, (ii) first cracking and, (iii) ultimate failure. In the first stage, load was applied incrementally at every 5 kN. After the first cracking occurs, loading was then controlled by deflection at a constant range until failure. Failure was defined when either the specimen failed in shear or interface failure i.e. SFRC topping separated from the precast slab.

Table 1: Mix proportions for plain concrete and SFRC mixes

Concrete Batch	**Design Concrete Compressive Cube Strength at 28-days (N/mm^2)**	**Ordinary Portland Cement (kg)**	**Fine Aggregate (kg)**	**Coarse Aggregate (max. size 10 mm) (kg)**	**Water (kg)**	**Steel fibres (%)**	**Water-cement Ratio**	**Super-plasticizer (ml)**
Batch 1	25	22.0	68.2	49.4	15.0	0.00	0.68	45.0
Batch 2						0.50		
Batch 3						0.75		
Batch 4						1.00		
Batch 5						1.25		

Table 2: Mix proportions design for composite slab specimens

Specimen Type	**Design Concrete Compressive Cube Strength at 28-days (N/mm^2)**	**Total Specimen Volume (m^3)**	**Ordinary Portland Cement (kg)**	**Fine Aggregate (kg)**	**Coarse Aggregate (kg)**	**Water (kg)**	**Steel fibre (kg)**	**Water-cement Ratio**	**Super-plasticizer (ml)**
Precast slab	40	0.306	132.8	285.0	238.6	70.4	-	0.53	-
Concrete topping (1.0 % of SFs dosage)	25	0.237	80.2	248.4	179.9	54.5	5.688	0.68	160.4

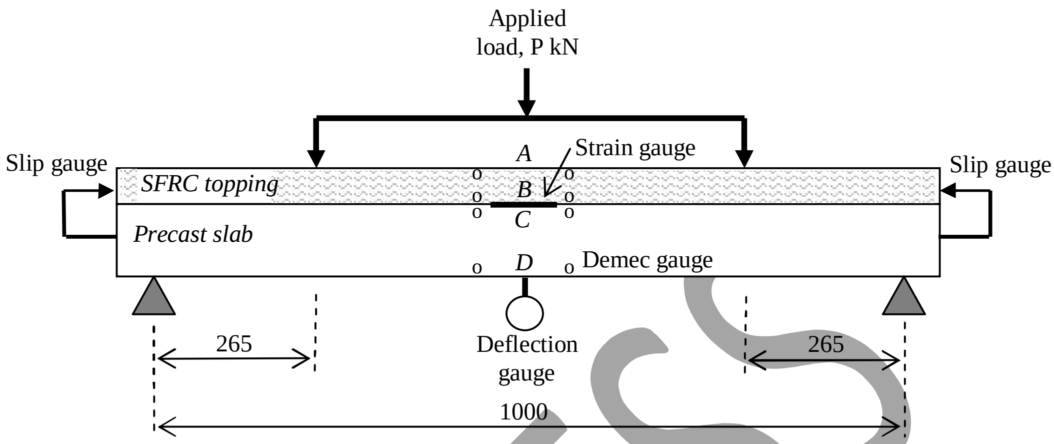


Figure 4: Combined bending and shear test setup (*all dimension are in mm*)

3.0 Results and Analysis

3.1 Stage 1 – Mechanical Properties of SFRC

The investigation on the mechanical properties of SFRC comprises of concrete compressive, tensile splitting and flexural strengths. Addition to this, the Modulus of Elasticity in compression was also determined. The concrete compressive and flexural strength were tested at 7, 14 and 28 days to determine the strength development of SFRC, where as the tensile strength and Modulus of Elasticity in compression were tested at 28 days.

The concrete properties with different volume fraction of steel fibre are summarised in Table 3. Concrete cube compressive strength at 28-days for specimens with V_f of between 0.50% and 1.25% were lower than the plain ones. The lowest compressive strength of 31.9 N/mm^2 at 28 days was recorded for the specimen with V_f = 0.50%. As V_f increases between 0.75% and 1.25%, there was a slight increase in the compressive strength of between 1.0 to 2.0 N/mm^2. However, the overall compressive strength was still acceptable since they were higher than the designed strength of 25 N/mm^2. The study also proof that for higher aspect ratio; *L/d* = 80, SFRC may induce lower compressive strength but increase in toughness and peak strain, which can lead to better crack control and energy absorption (Wang et al., 2010).

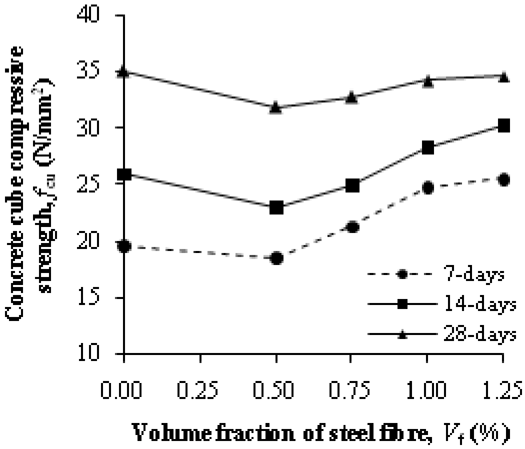
Table 3: Concrete properties with different steel fibre volumetric percentage

	Concrete Cube Compressive Strength (N/mm^2) †			Cylinder Splitting Tensile Strength (N/mm^2) †	Flexural Tensile Strength (N/mm^2) †			Modulus of Elasticity (kN/mm^2)
Concrete Batch	**7-days**	**14-days**	**28-days**	**28-days**	**7-days**	**14-days**	**28-days**	**28-days**
Batch 1 (Plain concrete)	19.7	25.9	35.1	2.63	5.3	5.5	6.4	16.20*
Batch 2 (V_f = 0.50%)	18.6	23.0	31.9	2.67	5.3	6.3	7.1	28.90
Batch 3 (V_f = 0.75%)	21.3	25.0	32.8	2.66	4.4	6.6	7.3	28.75
Batch 4 (V_f = 1.00%)	24.7	28.3	34.2	2.74	4.9	6.5	7.8	32.30
Batch 5 (V_f = 1.25%)	25.5	30.3	34.6	2.79	5.4	6.9	7.9	33.65

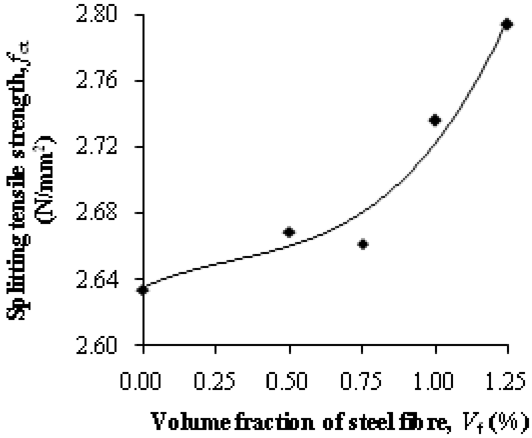
† *Average of three (3) samples*

* *Strains gauges were not in good working condition resulted in lower Modulus of Elasticity*

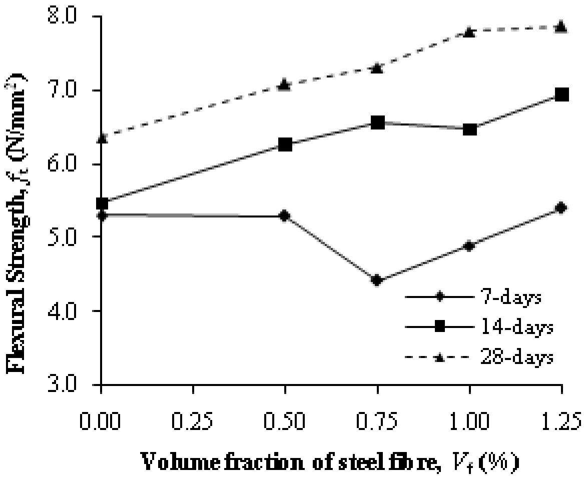
As for the cylinder splitting and flexural beam tests, the experimental result shows an increase in strength. The splitting tensile strength increased by 1.52%, 1.14%, 4.18% and 6.08% compared with plain concrete for V_f of 0.5%, 0.75%, 1.00% and 1.25%, respectively. As for the flexural strength, there was an increase of between 10.94% and 23.44% (at 28-days) compared with the control specimen. The experimental investigation found that SFRC is better than plain concrete especially under tensile and flexural load, where steel fibre takes part in absorbing the applied load. Fig. 5 shows the relationship between volume fraction of steel fibre and the compressive, tensile and flexural strengths. The Modulus of Elasticity given in Table 3 also shows an increase in strength as the amount of steel fibre increases. This increase shows that by adding steel fibre in concrete, SFRC can absorb higher stress, thus, support the finding by the previous researcher in improving the energy absorption (Khaloo & Afshari, 2005).



(a) Relationship between concrete cube compressive strength and V_f



(b) Relationship between concrete splitting tensile strength and V_f at 28 days



(c) Relationship between concrete flexural strength and V_f

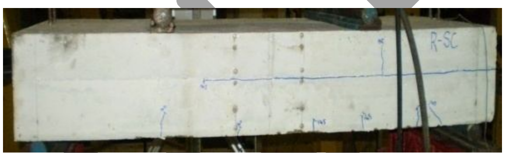
Figure 5: The mechanical properties of SFRC with different volume fraction of steel fibre

3.2 Stage 2 – Composite Slab

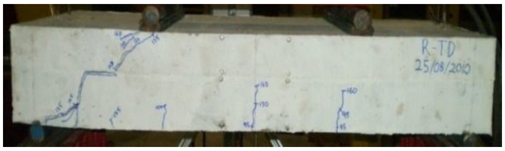
From the Stage 1 results, the study suggest that for V_f = 0.50% and V_f = 0.75%, they are merely ineffective because there is not much improvement on the strength development and even the Modulus of Elasticity compared with plain concrete. Meanwhile, for SFRC with V_f = 1.00% and V_f = 1.25%, they resulted in the increase in the splitting tensile and flexural strengths. However, the increase is relatively small up to only 0.1 N/mm^2. Based on this finding, V_f = 1.0% was added to the concrete topping in the second stage of the experimental work. Adding these steel fibres is a replacement to the conventional cast in-situ concrete topping using steel mesh as reinforcements. The concrete properties of the precast slab and SFRC topping are given in Table 4. The concrete properties achieved the design strength of 40 N/mm^2 and 25 N/mm^2 at 28-days for the precast slab and SFRC topping, respectively.

Fig. 6 shows the cracking pattern at failure for all specimens. Except for smooth as-cast surface, other specimens exhibit shear cracking failure at ultimate. For all specimens, first cracking was observed as flexural crack initiated at the mid-span region. For the smooth as-cast surface, there was a sudden failure at the interface with the SFRC topping and precast slab separated at ultimate. The interface failure indicates that the composite slab lost its monolithic behaviour when the load transfer mechanism provided by the interface bond failed.

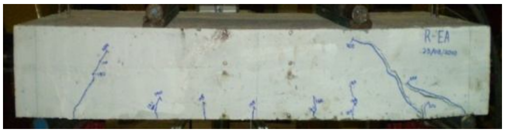
As for the other specimens, when the load was further increased, more flexural cracks were developed at the mid-span region. This was then followed by shear tension failure either at the left or right end section. The shear tension failure was initiated at the support and grew upwards making an angle of between 30° and 45°. The crack was then moved further upwards into the SFRC topping towards the loading point. The test was stopped when concrete crushing occurred at the support.



(a) R-SC (Smooth as cast)



(b) R-TD (Roughened in the transverse direction)



(c) R-LD (Roughened in the longitudinal direction) (d) R-EA (Roughened by exposed aggregate)

Figure 6: Cracking pattern at failure

Table 4: Composite slabs concrete properties

<table>
<tr><th rowspan="3">Specimen Name and Surface Type</th><th colspan="5">Precast Slab</th><th colspan="5">SFRC Topping</th></tr>
<tr><th colspan="3">Concrete Cube Compressive Strength, f_{cu} (N/mm^2) *</th><th>Concrete Cylinder Splitting Tensile Strength, f_{ct} (N/mm^2) *</th><th>Modulus of Elasticity (kN/mm^2)</th><th colspan="3">*Concrete Cube Compressive Strength, f_{cu} (N/mm^2)</th><th>Splitting Tensile Strength, f_{ct}</th><th>Modulus of Elasticity (kN/mm^2)</th></tr>
<tr><th>7 days</th><th>28 days</th><th>Test day (44 days)</th><th>28 days</th><th>28 days</th><th>7 days</th><th>28 days</th><th>Test day (36 days)</th><th>28 days</th><th>28 days</th></tr>
<tr><td>R-SC Smooth as-cast</td><td rowspan="4">27.22</td><td rowspan="4">42.16</td><td rowspan="4">43.83</td><td rowspan="4">3.15</td><td rowspan="4">25.45</td><td rowspan="4">17.87</td><td rowspan="4">27.15</td><td rowspan="4">30.51</td><td rowspan="4">2.42</td><td rowspan="4">32.30</td></tr>
<tr><td>R-TD Roughened in the transverse direction</td></tr>
<tr><td>R-LD Roughened in the longitudinal direction</td></tr>
<tr><td>R-EA Roughened by exposed aggregate</td></tr>
</table>

**Average of three (3) samples*

The relationship between the applied load and mid-span deflection for all specimens is shown in Fig. 7. The figure shows that specimen with exposed aggregate surface (R-EA) failed at the highest load of 197.23 kN; 15.34% higher than P_{cal}. Apart from that, the lowest mid-span deflection of 4.64 mm was recorded for specimen with smooth as-cast surface (R-SC). Specimen with longitudinal roughened surface (R-LD) also performed well as the ultimate load was recorded at 191.73 kN; 12.12% higher than P_{cal}. This was, however, 2.79% lower than R-EA specimen. On the other hand, specimen with transverse roughened (R-TD) and smooth as-cast surface (R-SC) failed at 155.73 kN and 146.93 kN, respectively, which was 8.93% and 14.08% lower than P_{cal}.

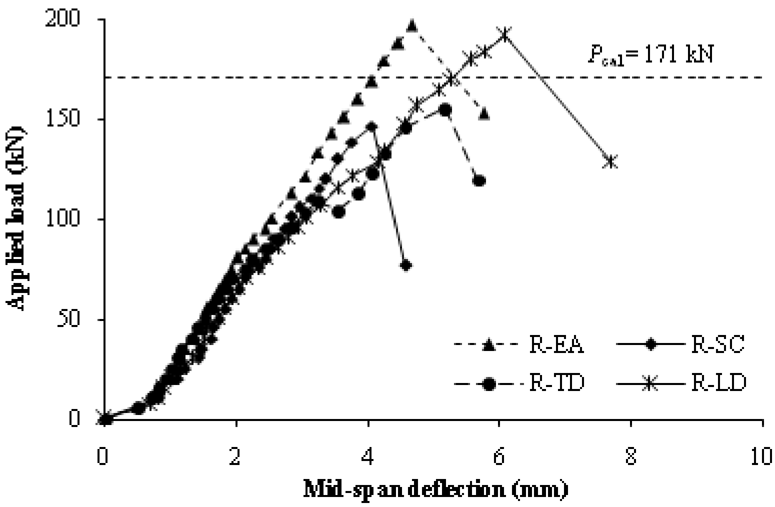
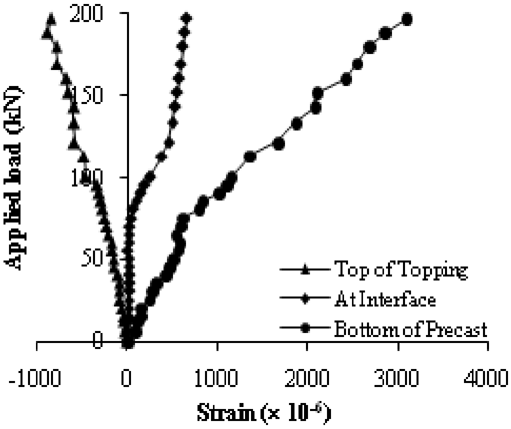
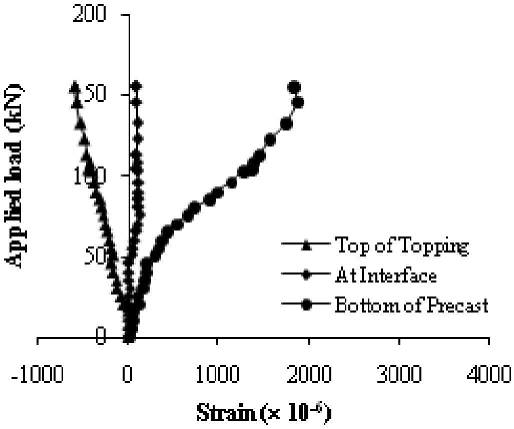


Figure 7: Relationship between applied load and mid-span deflection

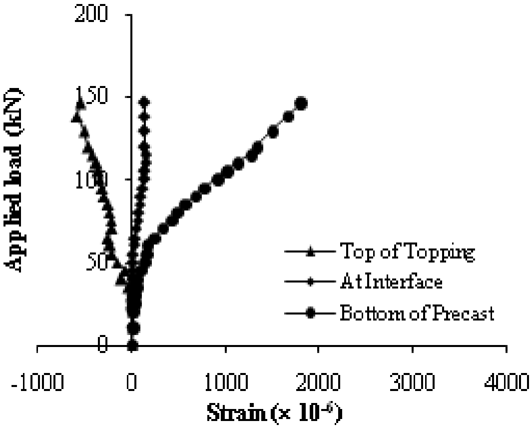
Fig. 8 shows the applied load and strain relationship at the top of the SFRC topping, interface and bottom of precast slab. All specimens show similar pattern where negative strain was recorded at the top of the SFRC topping meanwhile positive strain was recorded at the bottom of the precast slab. This shows the top of the SFRC topping was in compression whereas the bottom of the precast was in tension. As for the interface, the strain was apparently zero (also known as free strain) during the first few loading increment up to about 50 kN before recording positive strains indicating that the interface was in tension when loading was further increased. Also, positive strain at the interface confirms that the calculated neutral axis to be in the SFRC topping i.e. x = 62.45 mm from the top. Prior to cracking, the bottom strain were acting elastically and after the first cracking occurred, large strains were recorded under small loading increment. This shows the non-linearity behaviour of the specimens. When the specimens were near to their ultimate, large bottom strain were recorded between 2000 $\times 10^{-6}$ and 4000 $\times 10^{-6}$.



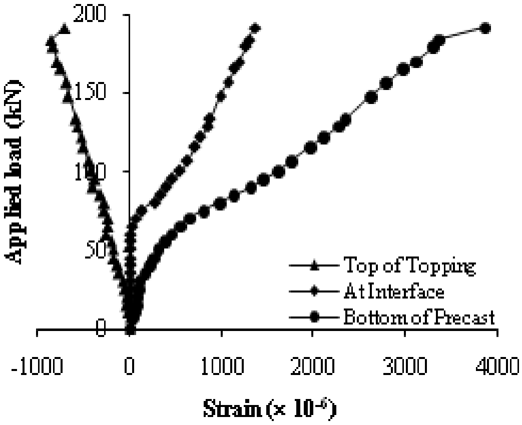
(a) R-SC (Smooth as cast)



(b) R-TD (Roughened in the transverse direction)



(c) R-LD (Roughened in the longitudinal direction



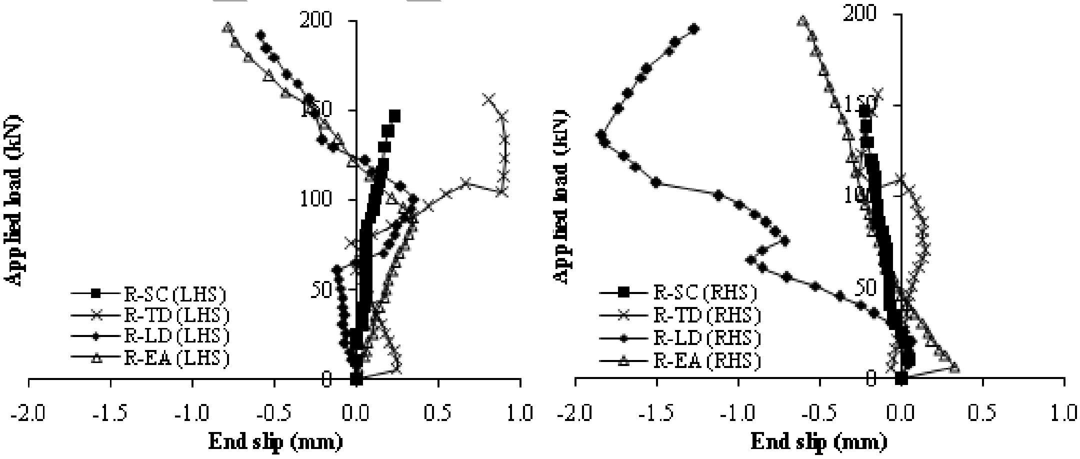
(d) R-EA (Roughened by exposed aggregate)

Figure 8: Relationship between the applied load and strain

The relationship between the applied load and interface slip is shown in Fig. 9. As shown previously in Fig. 4, the interface slip was measured only at the left and right end section. Since the interface slip at the mid-span section was unable to be measured experimentally, they were calculated theoretically from the strain diagram proposed by Ibrahim et. al. (2008).

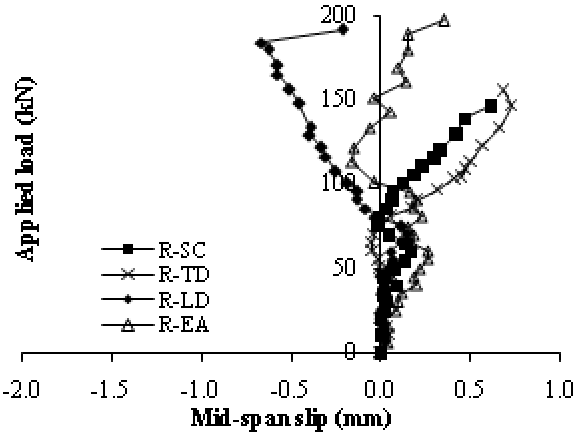
Illustrated in Fig. 10(a), strain gauge installed at the interface gave reading at point C while Demec gauge recorded values at point A and D (see Fig. 4). Fig. 10(a) represents the strain diagram when the specimen responds as full composite action under flexural load. When the interface was disturbed e.g. interface slip, the strain changes between point B and C (see Fig. 10 (b)). Hence, the strain gradient between point A' and C' was inapplicable as it does not present the actual strain behaviour in the SFRC topping as shown in Fig. 10(c). Therefore, interface slip (e.g. different in term of strain between point C' and B') can only be determined when the strain gradient between point A' and B' (apparent strain gradient) equal to the true strain gradient between point C' and D'. Typical strain distribution diagram for the R-SC specimen is shown in Fig. 11. The diagram confirms the disturbance at the interface due to the unequal gradients above and below the interface. This is also known as partial composite action where further analysis produces relationship between the applied load and mid-span slip (see Fig. 9(c)).

From Fig. 9, the end and mid-span slip were not similar to each other both in pattern or magnitude. Positive slippage shows that SFRC topping was bending in the same way as the precast slab until at a particular load where the steel fibre takes part in absorbing the energy and preventing the SFRC topping from further bending. Hence, it causes negative slippage as well as threatens the failure at the interface. As for the mid-span slip shown in Fig. 9(c), it was also experiencing positive and negative slippage. Therefore, this confirms the end slip behaviour showing similar pattern with the mid-span slip.



(a) Left hand section (b) Right hand section

Figure 9: Applied load and interface slip relationship



(c) Mid-span section

Figure 9: Applied load and interface slip relationship (*continued*)

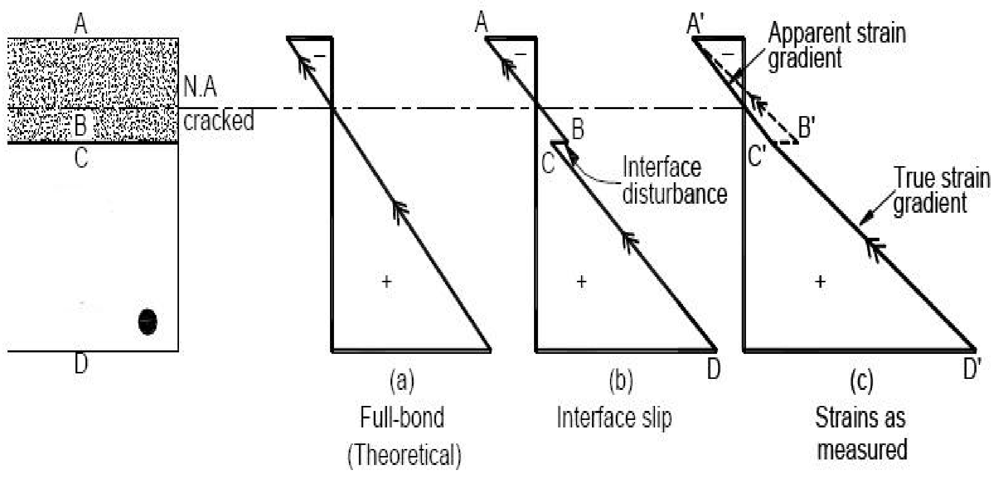


Figure 10: Strain distribution diagram (Ibrahim I.S. et al. 2008)

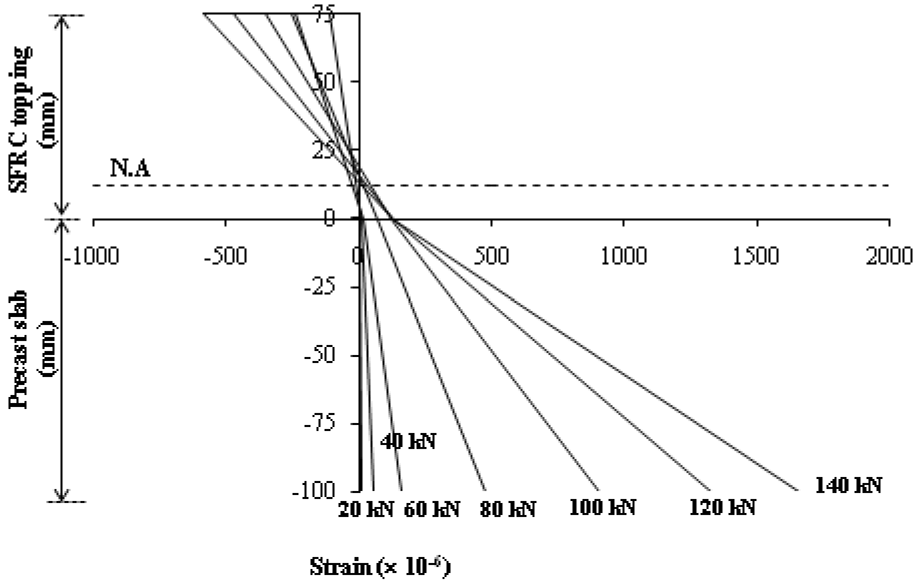


Figure 11: Strain distribution diagram for R-SC specimen

4.0 Discussion and Parametrical Equation

The shear force and mid-span deflection relationship is shown in Fig. 12. They were also compared with the conventional method of preparing structural topping using plain concrete and mild steel mesh as reinforcements. This work was done previously by Teck Yee (2009) with the same dimensional properties and using the same test setup as shown in Fig. 4. Except for R-EA, specimens with SFRC topping had lower ultimate shear capacity compared with the conventional topping. This may be due to higher compressive concrete strength of 35.61 N/mm^2 at 28 days for the conventional topping compared to only 27.15 N/mm^2 for the SFRC. The results are summarized in Table 5. Conventional topping with smooth surface (Plain R-SC) fails at 117.69 kN, which is the highest compared with the other specimens. Meanwhile, for SFRC topping, only R-LD and R-EA specimen produces higher ultimate shear capacity of 97.02 kN and 99.77 kN, respectively compared with the other types of surface finishes. This is, however, in contrast with the conventional topping for exposed aggregate (Plain R-EA) where the ultimate shear capacity is 15.28 kN lower than the SFRC topping with the same surface finishes (R-EA). This may be due to better dispersion of fibres in the topping even though the surface finish was the worst in terms of bonding capabilities. The lowest ultimate shear capacity was 74.62 kN for smooth as-cast surface meanwhile for transverse roughened failed 2.70% higher than the smooth ones.

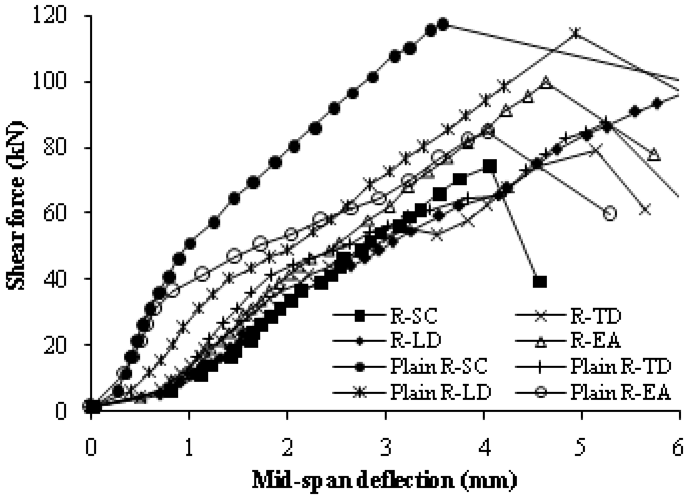


Figure 12: Shear force and mid-span deflection relationship up to 6 mm

To further compare the experimental results, a parametrical work in determining the ultimate shear capacity for SFRC topping is proposed in this paper and will be further discussed. The general equation known as shear stress equation in a beam or slab having a prismatic cross section and made from homogeneous materials (Hibbeler, 2005) can be expressed as:

$$\tau = \frac{V \int y \, dA}{I_{comp} b_v} \qquad (1)$$

When two surfaces are in contact and subjected to a shear force and compressive force, the two surfaces will tend to displace relative to each other in the direction of the applied load, produces $\tau = \mu \cdot \sigma_n$. Crushing occurs when these constituents reach their compressive capacity, which is directly related to the concrete cohesion, k_c and concrete splitting tensile strength, f_{ct}. By taking into account these factors, the shear stress formula becomes:

$$\tau = k_c \cdot f_{ct} + \mu \cdot \sigma_n \qquad (2)$$

Rearranging Eq. (1) and (2):

$$k_c \cdot f_{ct} + \mu \cdot \sigma_n = \frac{V \int y \, dA}{I_{comp} b_v} \qquad (3)$$

As the specimens were subjected to two point loads, the term of force per unit area, σ_n caused by the minimum external force across the interface was taken as zero. Therefore, Eq. (3) becomes:

$$k_c \cdot f_{ct} = \frac{V \int y \, dA}{I_{comp} b_v} \qquad (4)$$

where I_{comp} is the second moment of area computed about neutral axis from the transformed section and b_v is the interface width or concrete topping width.

However, the shear formula both given in Eq. (1) and Eq. (4) does not give accurate results when applied to members having cross sections that are short or flat, or at points where the cross section suddenly changes. Furthermore, Eq. (1) was derived indirectly from flexural formula and it is necessary that the material behave in linear-elastic manner and also necessary to have a Modulus of Elasticity that is same either in tension or compression zone (Hibbeler, 2005). Hence, Eq. (1) can be modified by considering the modular ratio of the two materials, η and transformation factor (as a muiltiplier to converts the dimensions of the cross section of the composite member into a member that made from a single material) in determining the first moment of area and the moment of inertia of the composite section computed about the neutral axis. Therefore, Eq. (4) can be represented as;

$$k_c \cdot f_{ct} = \frac{V \Sigma_i \int_{A_i} n_i y dA}{I_{comp} b_v} = \frac{V S_{comp}}{I_{comp} b_v} \qquad (5)$$

where the sum shall be extended over all cross sectional areas, A_i below the considered horizontal surface (in this study the neutral axis lies in the concrete topping).

In the present study, the precast slab was transformed into one made entirely of SFRC topping by reducing the width to an equivalent width for concrete topping and transform the total area of steel into an equivalent area of concrete, S_{comp}. Therefore we have;

$$S_{comp} = \left[\frac{E_{slab}}{E_{topping}}(b_v)\right](y_p h_p) - (y_s A_s) + \left[\frac{E_{steel}}{E_{topping}}(A_s)\right](y_s) \qquad (6)$$

where E_{slab} is the Elastic Modulus of the precast slab, $E_{topping}$ is the Elastic Modulus of the concrete topping, E_{steel} is the Elastic Modulus of steel reinforcement, y_p is the distance from the neutral axis of composite section to the centroid of precast slab, y_s is the distance from the neutral axis of composite section to the steel centroid in the precast slab, h_p is the precast slab depth and A_s is the area of the tension reinforcement.

Replacing $\eta_1 = \frac{E_{slab}}{E_{topping}}$ and $\eta_2 = \frac{E_{steel}}{E_{topping}}$, Eq. (6) becomes;

$$S_{comp} = [\eta_1 b_v](y_p h_p) + [\eta_2 A_s - A_s](y_s) \quad (7)$$

Replacing $\eta_1 b_v = b_{v,trans}$ and $\eta_2 A_s = A_{s,trans}$ and substitute Eq. (7) into Eq. (5), yield;

$$k_c \cdot f_{ct} = \frac{V}{I_{comp} b_v}\left[(y_p h_p b_{v,trans}) + (A_{s,trans} - A_s)(y_s)\right] \quad (8)$$

or

$$V_{u,calc} = \frac{I_{comp} b_v}{[(y_p h_p b_{v,trans}) + (A_{s,trans} - A_s)(y_s)]} \cdot k_c f_{ct} \quad (9)$$

where $V_{u,\ calc}$ is the calculated ultimate shear capacity, $b_{v,\ trans}$ is the equivalent precast width after being transformed, and $A_{s,\ trans}$ is the equivalent area of tension reinforcement after being transformed, k_c is the factors depending on the roughness of the interface and f_{ct} is the concrete splitting tensile strength of the precast slab. The value of k_c is taken as 1.0 since they were not able to be measured during the test. The value of k_c is also given in Eurocode 2 (European Committee, 2004) for different types of surface texture. However, the value provided in the code is not valid in the current study since k_c is used to determine the interface shear strength, where the shear capacity is usually less than ultimate, or before the first interface slip occurs.

Table 5: Comparison between the experimental results and calculated values

Specimen	**Shear force before the first interface slip occur, V_{slip} (kN)**	**Experimental Ultimate Shear Capacity, $V_{u,\ exp}$ (kN)**	**Calculated Ultimate Shear Capacity Eq. (9), $V_{u,\ calc}$ (kN)**	$\frac{V_{u,exp}}{V_{u,calc}}$
R-SC	14.07	74.62	103.5	0.72
R-TD	11.52	79.02		0.76
R-LD	16.62	97.02		0.94
R-EA	11.52	99.77		0.96
Plain R-SC*	21.84	117.69	104.5	1.12
Plain R-TD*	6.54	87.64		0.84
Plain R-LD*	20.59	114.69		1.10
Plain R-EA*	1.39	84.49		0.81

* *The test was carried out by previous researcher (Teck Yee, 2009)*

The comparison in Table 5 shows that for SFRC topping, $V_{u, exp}$ is 4% to 28% lower than $V_{u, calc}$. As for the conventional topping, Plain R-SC and Plain R-LD is 12% and 10% higher than $V_{u, calc}$. This may be due to the influence of roughness and concrete cohesion to the ultimate shear capacity of the specimens. Therefore, the value of k_c proposed in this paper should be between 0.8 and 0.9. Furthermore, the volume fraction of steel fibre used in this research remained at 1.0% which may also contribute to the lower shear capacity compared with the plain ones.

5.0 Conclusion

From the test carried out on the mechanical properties of SFRC and the ultimate shear capacity of the composite slab, several concluding remarks can be drawn as follows:

i. SFRC with V_f = 0.50% and V_f = 0.75% were merely ineffective in terms of compression, splitting tensile and flexural strengths and even for the Elastic Modulus as compared to plain concrete because there was not much improvement on the mechanical properties of SFRC.

ii. There was not much improvement for SFRC with V_f = 1.0% and V_f = 1.25% for the splitting tensile and flexural strengths. Therefore, SFRC with V_f = 1.0% is better than V_f = 1.25% by considering the economical factor and ease of work.

iii. SFRC specimens had lower shear capacity at failure as compared with the conventional (plain) ones.

iv. For SFRC topping, specimens with surface roughened by exposed aggregate and longitudinal roughened produces the highest ultimate shear capacity of 97.02 kN and 99.77 kN, respectively.

v. A parametrical equation is proposed in this paper considering the Modulus of Elasticity of the SFRC topping, precast slab and tension steel, and also the transformation factor in calculating S_{comp}.

vi. Comparison between the experimental results and the proposed Eq. (9) for specimens with SFRC topping found that $V_{u, exp}$ is 4% to 28% lower than $V_{u, calc}$. However, for the conventional topping, only plain R-SC and plain R-LD specimens were 12% and 10% higher than $V_{u, calc}$.

vii. From the test results, the study suggested that longitudinal roughened surface was better than other surfaces using SFRC as concrete topping in terms of interface bonding.

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