Chapter 7

Results of Slab Test

7.1 Background

In this section, results and the observations from the slab tests are presented and discussed. The discussion includes the full-scale slab test, cores and sawn beams.

7.2 Full-scale Slab Test

The results and the observations of the full-scale test are discussed. The discussion includes load capacity, deflection characteristics and failure patterns.

7.2.1 Results

Cubes and beams were cast using concrete mix identical to that of the two slabs. The results are summarized in table 7-1. (Refer to Appendix F).

Table 7-1: Strength Properties of the Slabs Mix

Property	Cube Strength (MPa) 7 Days	28 Days	First Crack Strength (MPa)	MOR (MPa)	Equivalent Strength (MPa)	Equivalent Ratio
SFRC	18.1	33.6	4.6	5.3	1.86	34.9
Plain Concrete	17.4	33.6	4.2	5.3		

Table 7-2 shows the full-scale test results for first crack and maximum load and its corresponding deflections. The given deflections are measured at loading points.

Table 7-2: Full-Scale Test Results

Impact Points	@ First Crack Load (KN)	@ First Crack Deflection (mm)	@ Failure Load (KN)	@ Failure Deflection (mm)
Interior Loading (Test 1)				
Ms	383.80	1.48	656.70	4.50
Mp	398.40	1.36	731.0	3.94
Edge Loading (Test 2)				
Es	181	6.34	538.0	14.13
Ep	184	6.50	513.0	13.60
Corner Loading @150 mm diagonally from the corner angle bisector (Test 3)				
Cs1	193	7.35	413.0	13.26
Cp1	202	6.70	437.5	14.6
Corner Loading @ 300 mm diagonally from the corner angle bisector (Test 4)				
Cs2	485	10.73	568	12.12
Cp2	487	14.60	598	17.23

7.2.2 Comparison Between the Slabs

The two slabs were designed to have similar strength. The after cracking strength used (42% according to steel fibers manufacturers tables) was not achieved for the specified mix with the specified steel fiber content. The actual measured value was 34.9%; therefore a marginal difference of about 7.4% might be obtained.

The theoretical relation between the SFRC and plain concrete slab is shown in figure 7-1. Theoretically, a depth reduction of 16.6% can be achieved by adding 15kg/m^3 of hook-ended steel fiber to the parent concrete mix. With this steel fiber dose, the percentage of reduction is constant (i.e. Considering a plain concrete slab having 200 mm depth, the equivalent SFRC slab depth is 167.8, thus the reduction in depth is 16.6%).

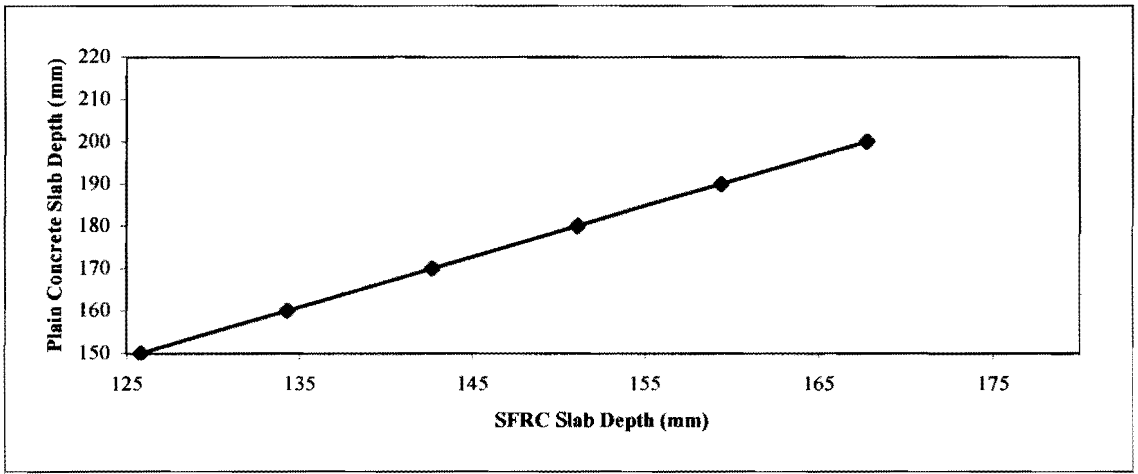

Figure 7-1: Relationship Between Depth of SFRC and Plain Concrete Slabs

Figure 7-2, figure 7-3, figure 7-4 and figure 7-5 show the load-deflection diagrams developed from the normalized measured datum for interior, edge, corner at 150 mm and corner at 300 mm respectively. It's appear from theses figures that the first crack point is difficult to estimate and a different researcher can deem different cracking point for same load–deflection relationship. Therefore, the determination of the toughness factor at a limiting deflection value can better justify the slabs. So it can be seen from the graphs that both slabs can absorb approximately equal energy, which indicate that their load capacities are equal. In these figures, the first portion of the load–deflection relation is curved and that is due to the initial seating for the loading plate (due to surface roughness) during the starting stage of the load application.

Figure 7-2 shows that, although the plain concrete slab has slightly higher load capacity, both slabs have the same pattern of load–deflection curve. Which imply that the two slabs have the same structural behaviour when considering the bearing capacity for the interior load case. Sudden failure anticipated for the plain concrete slab is not prominent. (See discussion on failure modes in section 7.2.2.3).

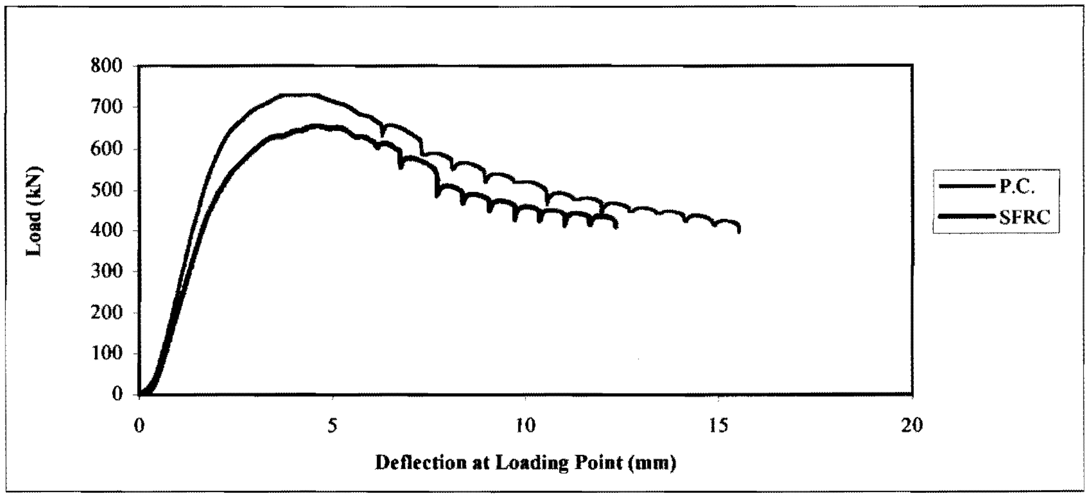

Figure 7-2: Full-scale: Test Load-Deflection Diagram (Interior Loading-Test 1)

Figure 7-3 shows the SFRC edge to have higher toughness and that the two edges have similar deflection at the ultimate failure. Apart from the localized reinforcement, the homogeneity of the SFRC tends to distribute the load in a larger area. The structural behaviour and failure characteristics of both slabs are again similar.

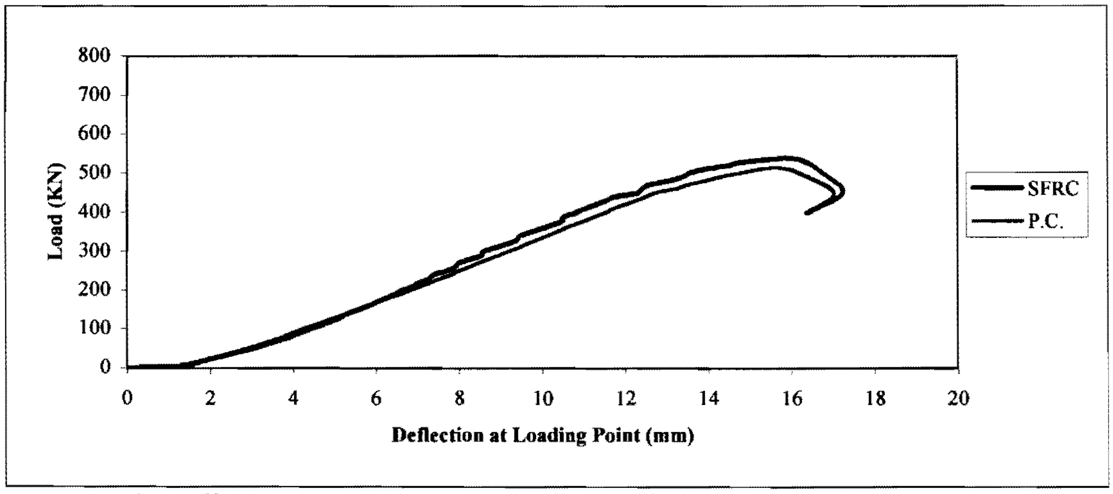

Figure 7-3: Full-scale Test: Load-Deflection Diagram (Edges Loading-Test 2)

Figure 7-4 shows that the behaviour of the two corners are approximately equal. They have different patterns of load-deflection curve, therefore, their structural behaviour is deemed to be slightly different. The ultimate failure for the SFRC corner was found at higher deflection but for low load capacity, while the plain concrete, fails at less deflection but at higher load. This can be taken as an advantage, because higher strains can be withstood with the SFRC than that of the plain concrete. Thus the brittle behaviour of the plain concrete corner is altered to a more elastic behaviour.

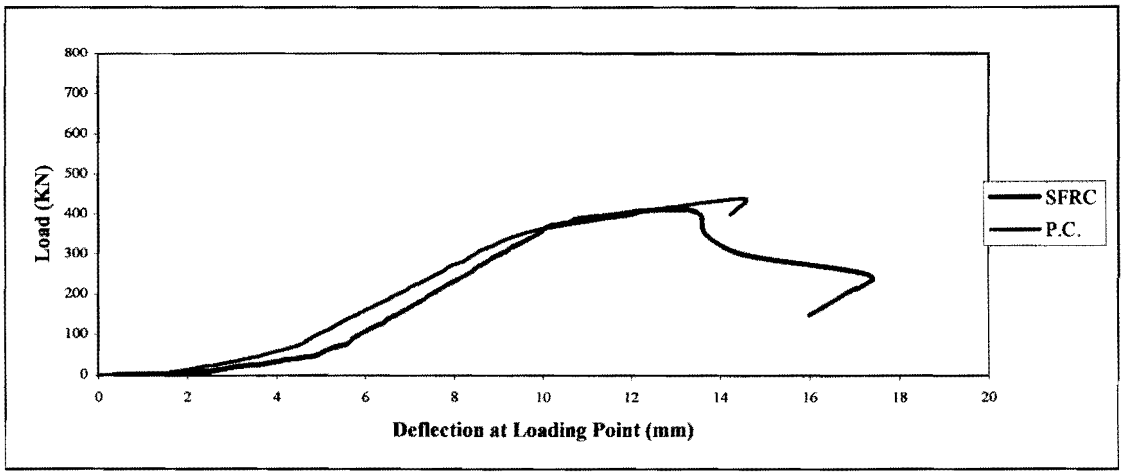

Figure 7-4: Full-scale Test: Load-Deflection Diagram (Load @150 mm-Test 3)

Figure 7-5 shows that the corner behaviour is completely dependent on the location of the load from its angle bisector. When corners are loaded at 300 mm from its angle bisector, the resulting load-deflection diagram is different to that for corner loaded at 150 mm (adjacent to the edges).

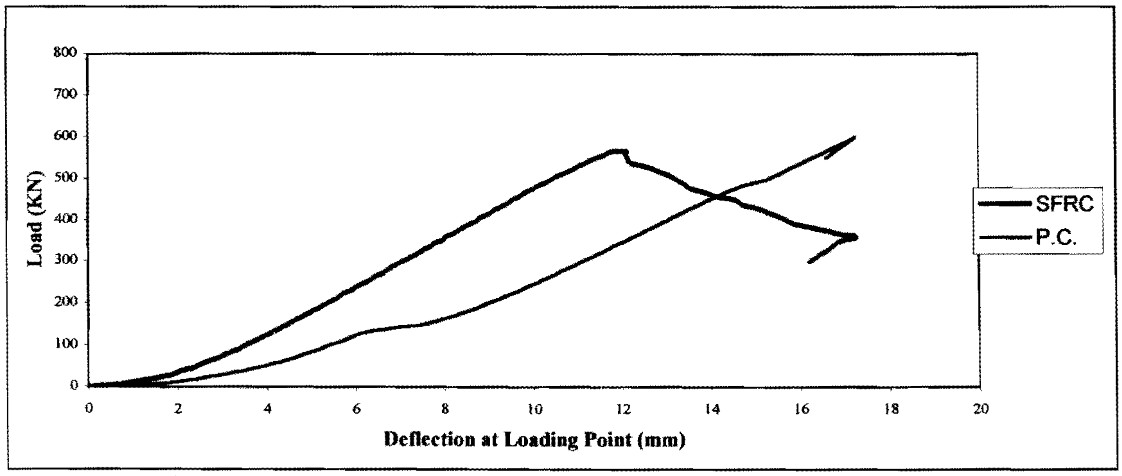

Figure 7-5: Full-scale Test: Load-Deflection Diagram (Load @300mm-Test 4)

7.2.2.1 Load Capacity

Figure 7-6 shows that, for interior, corners loaded at 150mm, corners loaded at 300 mm, and edge loading, the plain concrete was found to have 3.8%, 1.7%, 4.7%, and 0.4% greater first crack strength than the SFRC respectively. In comparison to the interior load, the corners loaded at 300 mm seems to yield very high first crack load. These relatively high values could be explained by the fact that these tests were conducted 90 days after casting the slab while the interior tests were conducted after 28 days. Other reason could be that something went wrong with the corners at 300 mm.

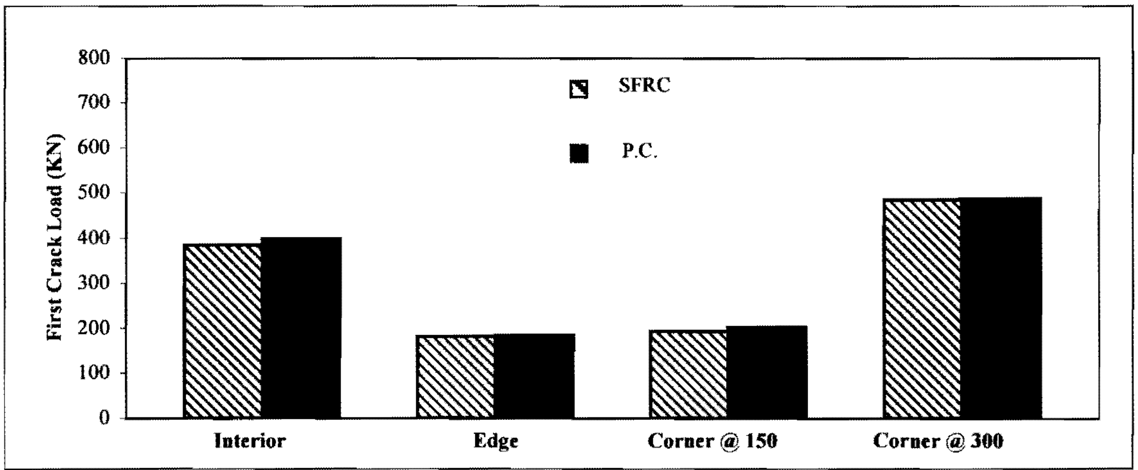

Figure 7-6: Load Capacity of the Slabs at First Crack

Figure 7-7 shows that, at maximum load the interior and corners of the plain concrete slab were found to have 11.3% and 5.5% greater values while the edge of the SFRC slab was found to be 5% stronger than the plain concrete slab. The edge of the SFRC slab shows higher load capacity than the edge of the plain concrete slab which is not the case at the first crack load as seen in figure 7-6. However the increase is negligible, but it can be an indication of the effect of the localized fiber orientation at that edge which cause the edge to sustain higher maximum load before its load capacity drops down.

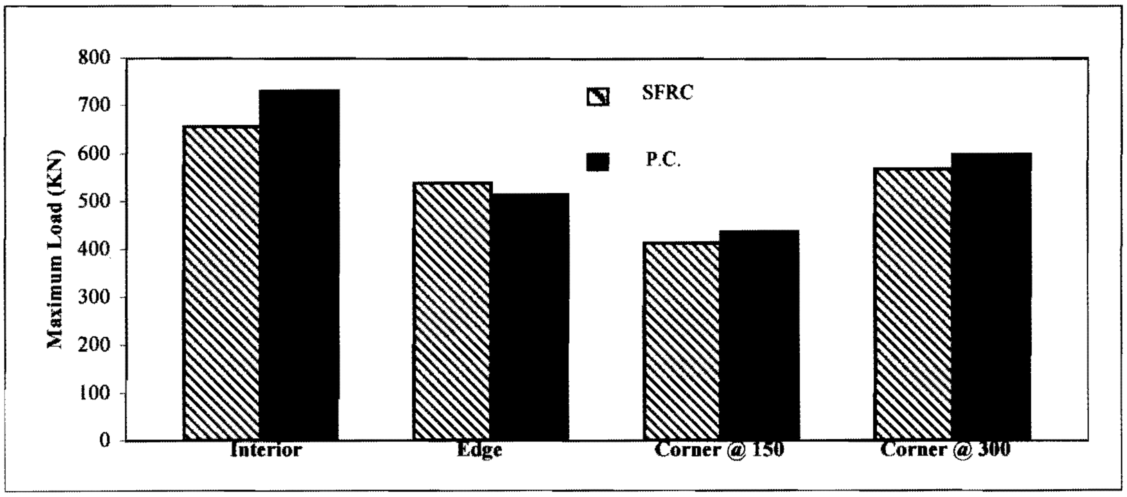

Figure 7-7: Load Capacity of the Slabs at Maximum. Load

The full-scale loading test showed that the two slabs are approximately equal in terms of first crack and maximum (termed failure) load capacities (Bearing in mind that the SFRC slab has 16.6 % less depth than the plain concrete slab).

7.2.2.2 Deflection Characteristics

Figure 7-8 shows the first crack deflection under load point for interior edge and corners of the SFRC and plain concrete slabs. The SFRC interior and corners at 150 mm was found to have 8.8% and 9.7% greater deflection respectively while edges and corners at 300 mm were found to have 2.5% and 36.1% less deflection respectively. Difference in deflection for the corners at 300 mm is huge.

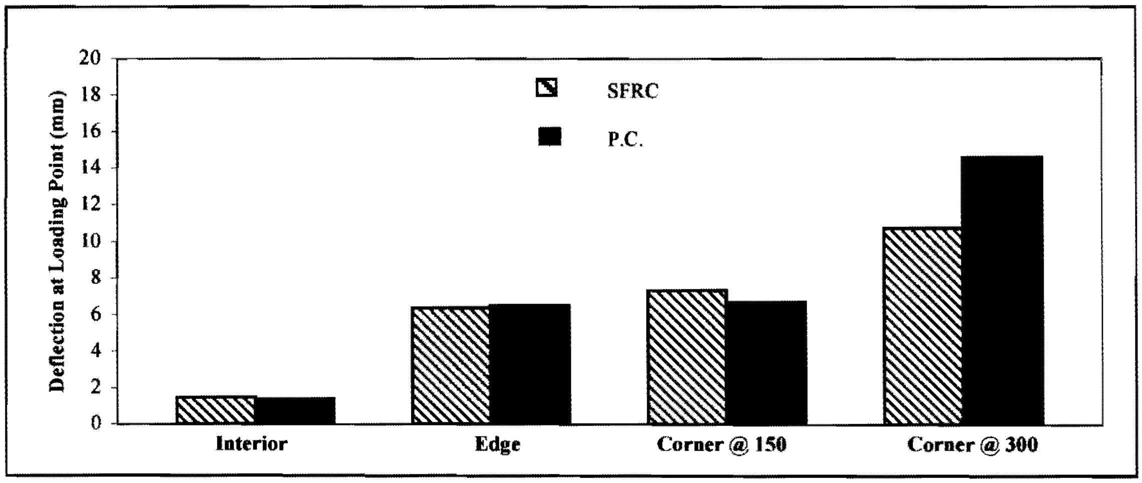

Figure 7-8: Deflection at Loading Point (at first crack load)

Figure 7-9 shows the deflection under load point at maximum load for interior, edge and corners of SFRC and plain concrete slabs. The SFRC interior and edge was found to have 14.3% and 3 % less deflection respectively. On the other hand, the SFRC corners at 150mm and at 300mm were found to have 10% and 42% greater deflection than plain concrete respectively. Difference in deflection for the corner at 300 mm is huge.

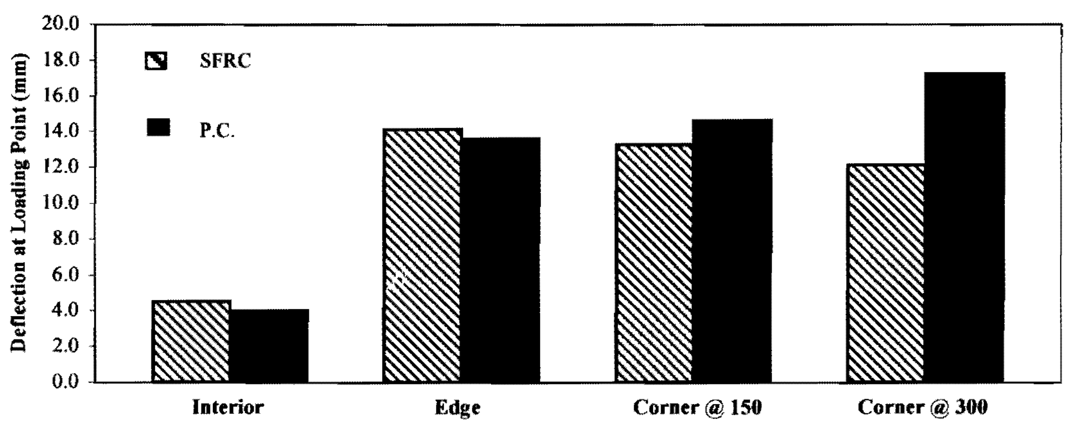

Figure 7-9: Deflection at Loading Point (at maximum load)

Figures from figure 7-10 to figure 7-17 shows the deflection profile for each case of load for both slabs. The deflection profile for each pair seems to have same pattern. It can be seen that for the most of the load conditions, the settlement is not that great relative to plain concrete which agrees with the study of Kaushik et al 1989, which stated that the settlement values of SFRC slab are less than or equal to that of plain concrete slab having same depth. Excessive deflection values associated with the free corners can cause densification or/and shearing of the underlying layers. Although free corners are rare in the reality of the pavement (usually doweled to the next corner), one should check that the resulting deflection doesn't exceed the underlying materials' vertical strain capacity for all critical load conditions.

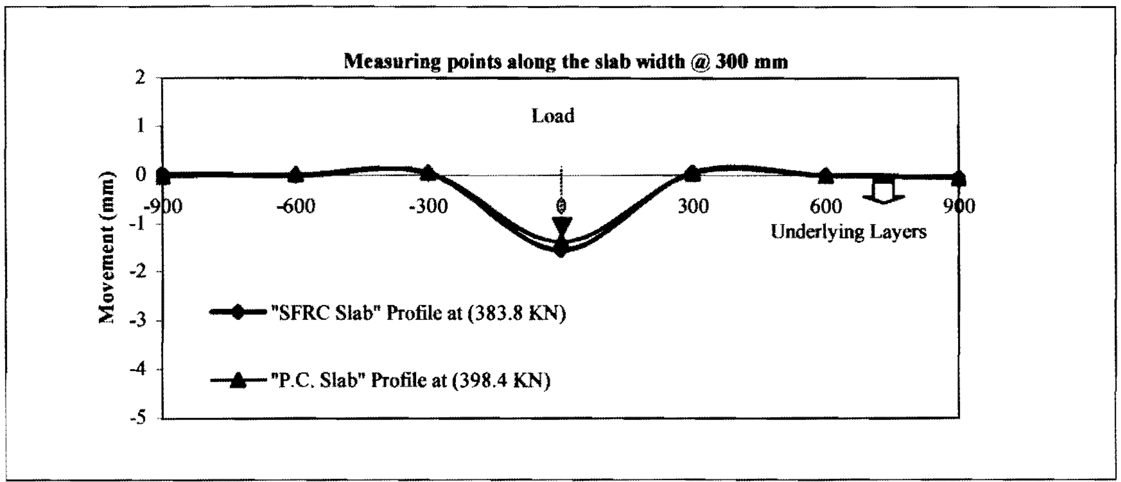

Figure 7-10: Deflection Profile at First Crack (Interior Loading-Test 1)

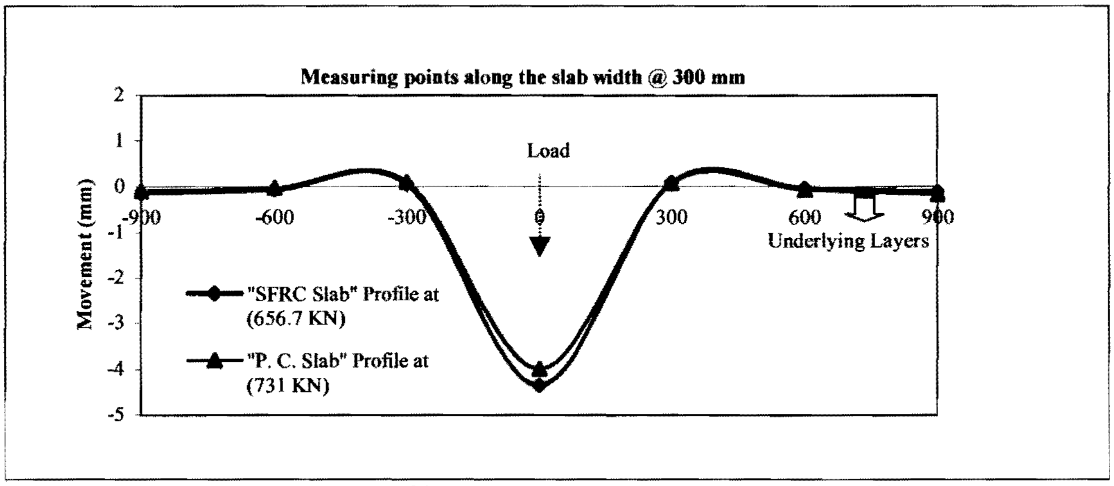

Figure 7-11: Deflection Profile at Maximum Load (Interior Loading-Test 1)

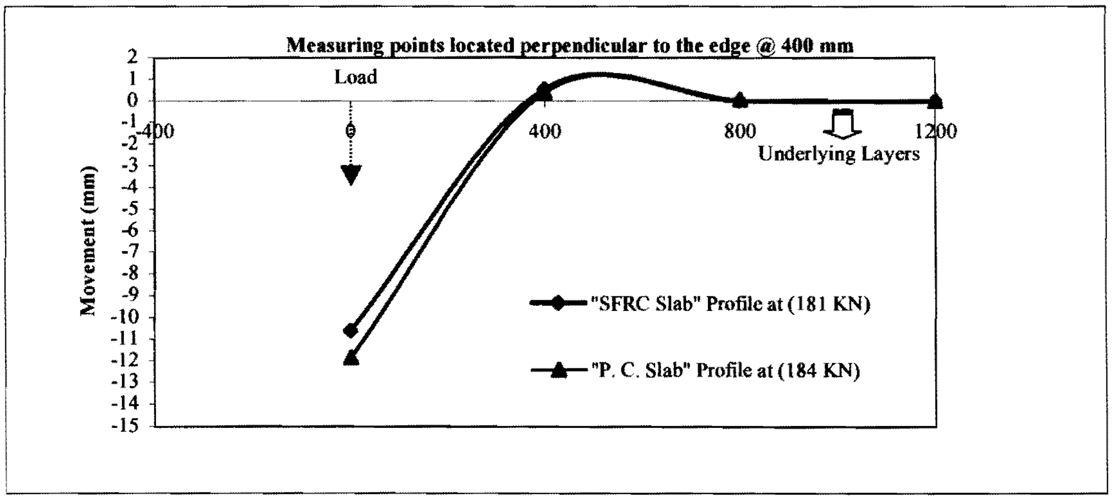

Figure 7-12: Deflection Profile at First Crack (Edge Loading-Test 2)

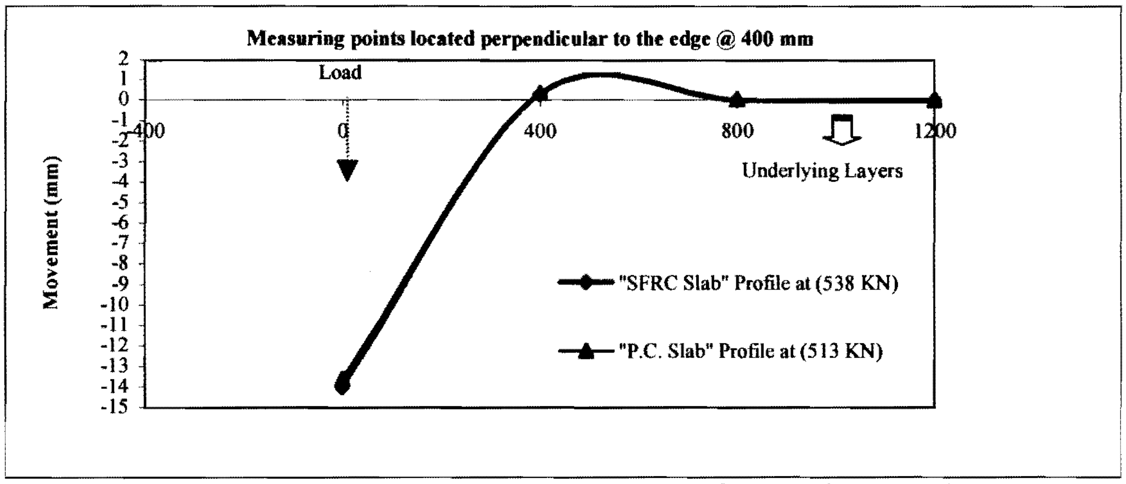

Figure 7-13: Deflection Profile at Maximum Load (Edge Loading Test 2)

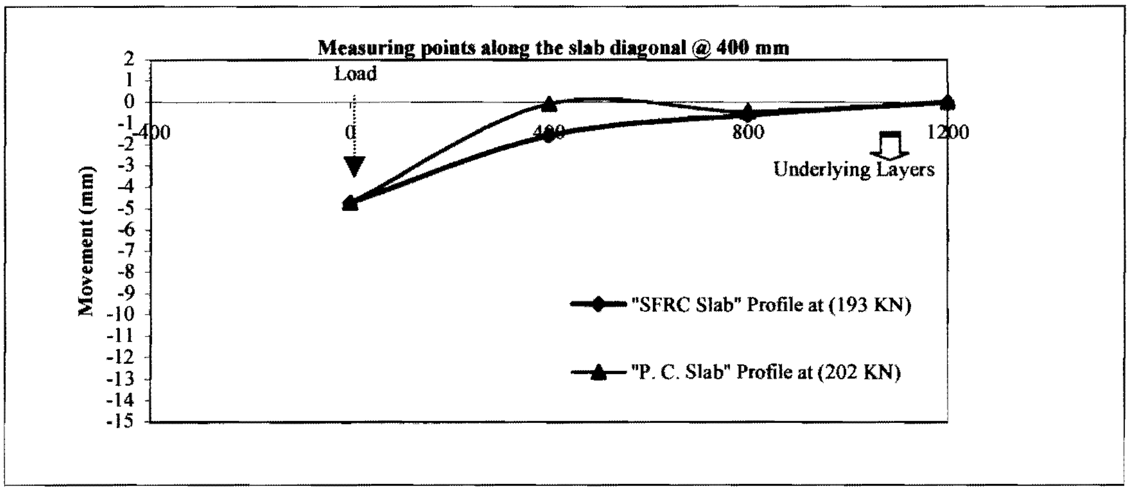

Figure 7-14: Deflection Profile First Crack (Corner at 150 mm-Test 3)

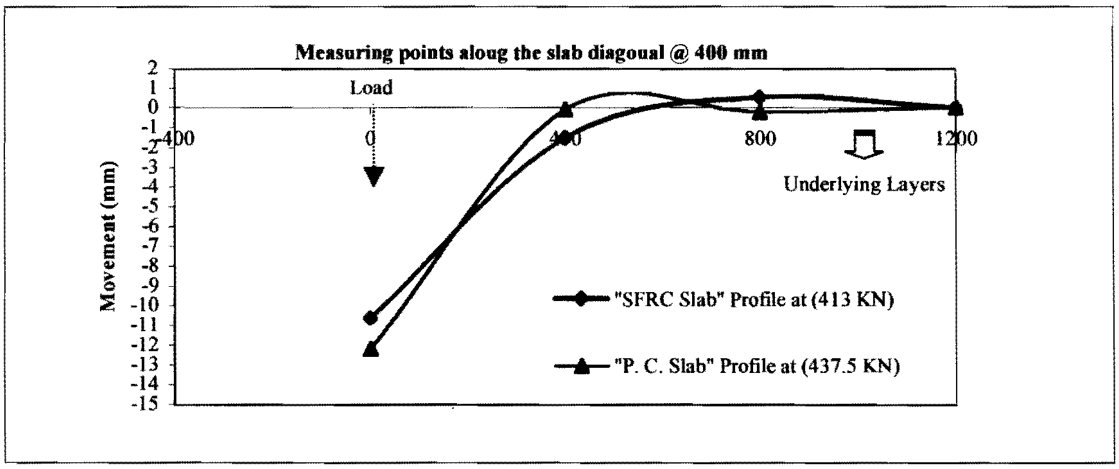

Figure 7-15: Deflection Profile at Maximum Load (Corner at 150mm-Test 3)

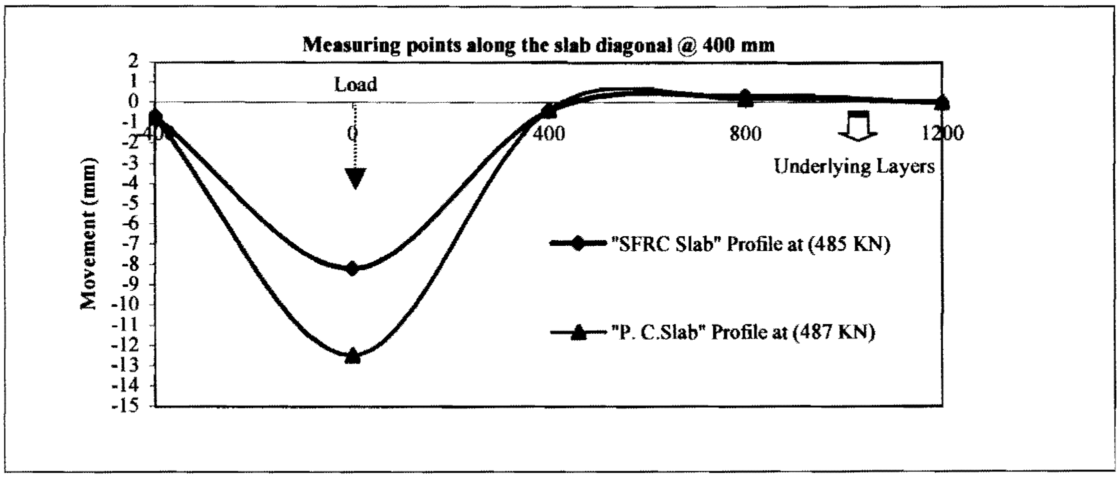

Figure 7-16: Deflection Profile at First Crack (Corner at 300 mm-Test 4)

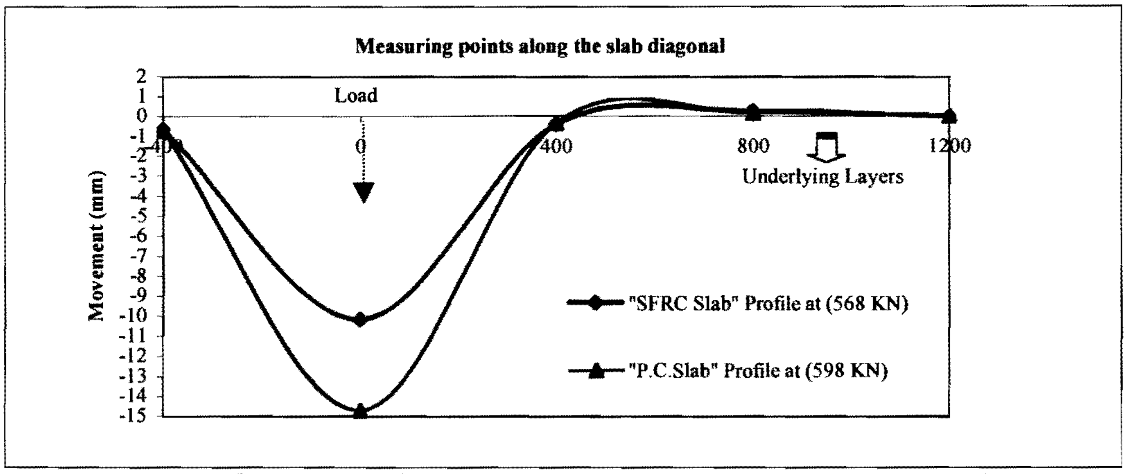

Figure 7-17: Deflection Profile at Maximum Load (Corner at300mm-Test4)

Once again (excluding the results for corners tested at 300) the slabs are approximately equal in terms of deflection.

7.2.2.3 Failure Characteristics

Based on experimental work and test observation, the mode of failure for edges and corners was not altered and the SFRC shows integrity and consistency compared to the plain concrete. Although corners loaded at 150 mm from angle bisector and edges for both slabs were sheared, the steel fiber ones held together with the slab while the plain concrete ones punched down about 10 mm deep into the sub-base. Corners loaded at 300 mm failed in bending. Cracking of the SFRC corner occurred at a distance from the angle bisector, which is greater than that of the plain concrete. This confirms that the SFRC slab distributes the load to a bigger area than that of the plain concrete slab. While spalling occurred along the crack at the edge and around the loading plate print at the corner of the plain concrete slab, the SFRC slab has shown no spalling.

Circumferential cracking with the "full fan" type associated with the interior load case as described in previous similar tests [83] was not observed on top of the two slabs. The settlement profile in figures 7-10 figure 7-11 shows the possibility that tiny circumferential cracking might have taken place at a distance of 400 mm from the center of the load. The reason behind tiny circumferential or no visible crack could possibly be the high K-value associated with the foamed concrete sub-base. It was also observed that the loading plate punched deeper by about 30 mm into the plain concrete slab while it punched about less than 10 mm into the SFRC. This might be attributed to the localized steel fiber reinforcement under and around the loading plate.

7.3 Compressive Strength Test on Cores

The compressive strength test on core specimens is discusses for the following:

- Variation on strength among the tested specimens.
- Mode of failure.
- Strength gain (results of the 28 days compared to those of 90 days).
- Conversion formula (formula to convert core strength into cube strength and formula to convert cube strength into flexural strength).

7.3.1 Results

Table 7-3: Compressive Strength Test on the Cores Taken from the Slabs.

Cores	L/D	Density (Kg/m^3)	Failure Load (KN)	Core Strength (MPa)	Actual Strength (MPa)	Potential Strength (MPa)
Steel Fiber Reinforced Concrete Cores						
S1	104/99	2299.6	318.6	41.41	38.6	50.3
S2	106/99	2305.2	293.1	38.1	36	47
S3	101/99.5	2267.7	368.6	47.4	43.9	57.2
S4	98/99	2281.2	339.5	44.1	40.4	52.7
S5	102.5/97.5	2353.3	317	42.5	39.9	52
S6	99/99	2258.2	349	45.3	41.7	54.4
Average				42.8	40.1	52.3
Plain Concrete Cores						
P1	105/99	2302.41	259.3	33.7	31.7	41.4
P2	106/99.5	2282.1	271.4	34.9	32.9	43
P3	101.5/100	2007.1	314.6	40	36.9	48.2
P4	98/98.1	2295.1	335	44.3	40.8	53.2
P5	102/99	2267	344	44.7	41.6	54.3
P6	99/98.5	2412.5	280.5	36.8	33.9	44.2
Average				39.1	36.3	47.4

7.3.2 Strength Variation

The variation of the compressive strength of the SFRC core specimens is expected to be higher than plain concrete ones, due to the randomness of the quantity and orientation of steel fibers in SFRC specimen. In spite of that, lower standard deviation of 3.5 was found for the SFRC specimens while a standard deviation of 5.4 was calculated for plain concrete specimens. The reason for that can be attributed the normal variation in concrete due to mixing, coring, casting, curing...etc.

7.3.3 Mode of Failure

Fracture mechanism for SFRC and plain concrete core specimens (under compression) was observed to fairly agree with that described by Neville and Brooks [21]. Under uniaxial compression, four stages were observed. The first stage was the formation of tension cracks parallel to the direction of load, second inclined cracks start to propagate and then noticeable disintegration to the specimens was seen and in the last stage failure took place. The only difference between SFRC specimens and plain concrete specimens was the last stage; at which the plain concrete specimens burst suddenly while the SFRC ones had a relatively gradual failure.

7.3.4 Strength Gain

After 90 days, 55.7% and 41.1 % are the gain in compressive strength for the SFRC and plain concrete cores relative to the 28 days strength. The extra strength for SFRC cores was anticipated and can be attributed to the after crack behaviour associated with SFRC. The failure of concrete specimens under uniaxial compression is mainly due to the formation of tensile cracks parallel to the direction of loading. Inclusion of randomly oriented steel fibers in the specimens arrest the propagation of tension cracks, there by apparently increases the compressive strength of concrete. This mechanism not only contributes to generate higher compressive strength for the SFRC specimens, but also contributes to the relaxed mode of failure. Figure 7-18 shows that the SFRC has a higher rate of strength growth than plain concrete.

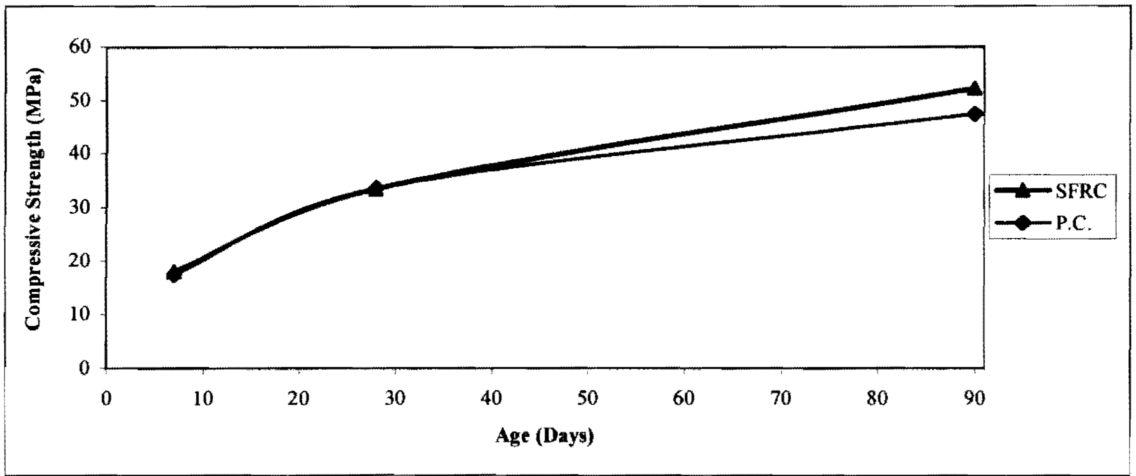

Figure 7-18: Strength Gain for SFRC and Plain Concrete

It was found that, potential strengths (derived for cores tested at 90 days) are greater by about 55.7% and 41.1% than the cube strength (assessed from crushing cubes at 28 days) for SFRC and plain concrete specimens respectively. It can be seen that an average of about 14.6% greater gain in strength is found for the SFRC specimens. The continued cement/ fly ash hydration causes both type of concrete to gain strength, but it has greater influence to the SFRC. Additional strength to the cement paste can also increase the interfacial bond between that paste and the steel fibers, which in turn contributes to more increase in the strength of the entire concrete. Another reason could be that the steel fibers acts to reduce the deterioration of the strength due to shrinkage cracks resulting from the dryness of the concrete. Previous tests show substantial increase in strength gain rate with high steel fibers dosages [24]. Although the difference in strength gain found by this test is marginal compared to the plain concrete, it can contribute to more economical pavement design especially when the pavement will not be loaded until many months after it has been placed.

7.3.5 Conversion Formula

Empirical formulas prescribed by BS1881: Part 120; 1983 and adapted from Neville and Brooks [21] were used to assess potential strength for both SFRC and plain concrete core specimens. The potential strength results in table 7-3 were compared to results from cubes having identical mix tested after 28 days. Comparison between compressive strength derived from cores and 28 days cube strength revealed that the conversion formulas are satisfactory and applicable to evaluate compressive strength for SFRC cores. Formulas that convert for cores with steel bars are not considered due to difficulties in finding the cross sectional area of steel fibers and to random orientation of these fibers.

7.4 Third-Point Loading Test on Sawn Beams

The results of the third-point loading test on the sawn beam specimens are discussed from the following angles:

- Capacity comparison between the SFRC and plain concrete,
- Toughness characteristics.
- Failure mode.
- Modulus of elasticity.

7.4.1 Results

Table 7-4 shows the results of the sawn beam specimens. Results of load, strength and deflection at first crack and at maximum load are presented. The equivalent load, strength and strength ratio are calculated according to the JSCE-SF4 method. Modulus of elasticity is calculated on the bases of elasticity theory using the load-deflection curve obtained from the third-point loading test.

Table 7-4: Sawn Beams Taken from the Slabs: The JSCE-SF4 Characteristics

Property \ Beams			IS	OS	IP	OP
Dimensions (hxbxL mm)			131x122x544	130x118x556	155x158x563	158x154x560
At First Crack		Load (KN)	19.8	18.3	26	24.5
		Strength (MPa)	4.3	4.2	3	2.9
		Deflection (mm)	0.04	0.04	0.03	0.03
At Maximum Load		Load (KN)	22.48	21.61	31	27.7
		Strength (MPa)	4.8	4.9	3.7	3.4
		Deflection (mm)	0.08	0.08	0.06	0.07
Japanese Standard JSCE-SF4 Properties	Equivalent Load (KN)	$P_{e,3}$	10.24	11.01	–	–
		$P_{e,1.5}$	13.11	16.01	–	–
	Equivalent Strength (MPa)	$f_{e,3}$	2.2	2.49	–	–
		$f_{e,1.5}$	2.82	3.61	–	–
	Equivalent Strength Ratio	$R_{e,3}$	51.2	60.6	1	1
		$R_{e,1.5}$	65.6	88	1	1
Modulus of Elasticity (MPa)			$28x10^3$	$27x10^3$	$23.4x10^3$	$24.6x10^3$

7.4.2 Capacity

Table 7-4 shows that the load capacities of the sawn plain concrete beams are greater than those for the SFRC beams bearing in mind that smaller section are loaded for the last. On the other hand the flexural strength capacity of the SFRC beam is greater by 40 and 35% for first crack and maximum load respectively, which agrees with many other results reported elsewhere. It can also be seen that the smaller depths for the SFRC beams cause them to yield higher deflection, which agrees with the results obtained from the full-scale slab testing.

The higher flexural strength capacity found for the sawn SFRC beams can be explained by the upward movement for the beam's neutral axis which implies that greater portion of the beam's section is involved in resisting the applied flexural stress.

7.4.3 Toughness Characteristics

First crack flexural strength calculated at 90 days from sawn beam specimens is found less than the one calculated from beam specimens tested at 28 days. The sawing action might be the reason for that. First crack strength estimated from load deflection test is approximately equal to the modulus of rupture for the same concrete, therefore, for design purposes the term f_{ct} in equation 2-1 can be substituted with the modulus of rupture or it can be assesses from the compressive strength as discussed later. The modulus of rupture is convenient and easy to assess compared to the first crack strength

The equivalent flexural strength ratios presented in table 7-4 show that the after crack strength is equal to about 50 to 60 % of the first crack strength when considering 3mm deflection and equal to about 65 to 88% of the first crack when 1.5 mm deflection is considered. This means higher flexural strength can be gained when considering less deflection. It can be argued that the flexural strength assessment still needs further investigation, because with limiting the unfavourable deflection to a lower values higher flexural strength could be obtained which in turn reduced the section of structural elements. The situation is aggravated when speaking about ground slabs. Further work is required to prove that the L/150 (L=length) free deflection will yield deflection values that are not very small compared with the tolerated limits for ground slabs. The L/300 deflection limit could be considered for the pavements and that the extra strength could be utilized.

Figure 7-19 and figure 7-20 show that the load-deflection curve for the SFRC beams has a straight portion immediately after the maximum load. That portion is also reported elsewhere and known as the region of instability. That straight portion is because the sequences of its occurrence are faster than the response of the measuring devices. In fact that is one of the reasons for using the JSCE–SF4 method instead of the ASTM C1018. Apart from difficulties in assessing the first crack strength, the toughness factors and their relative residual strength ratios calculated within this zone are erroneous.

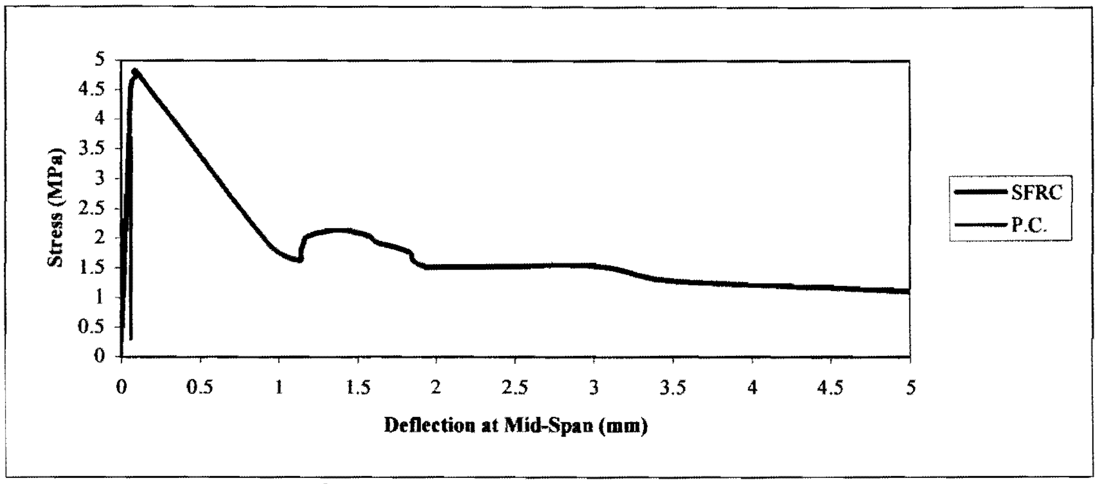

Figure 7-19: Stress-Deflection Diagram (IS &IP-inner beams)

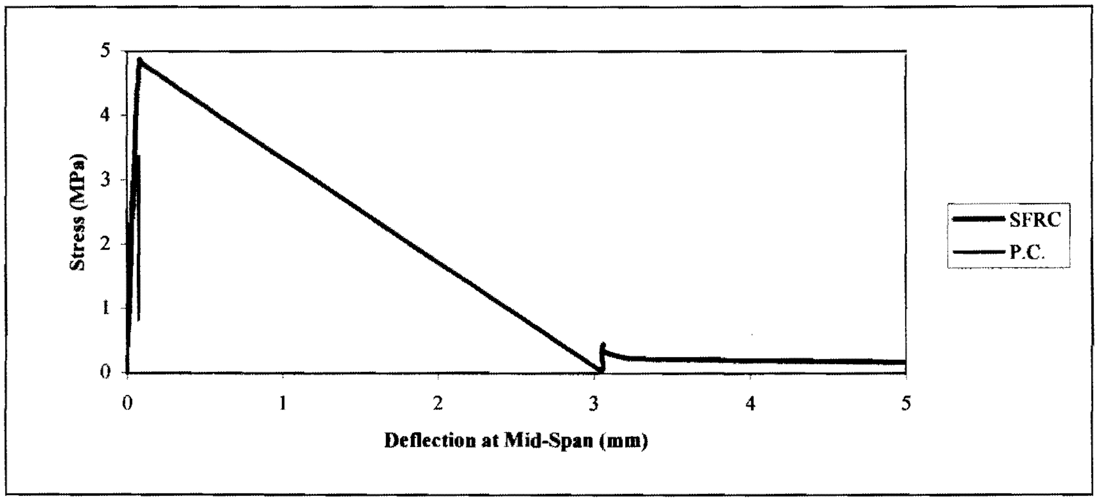

Figure 7-20: Stress-Deflection Diagram (OS &OP-outer beams)

Values for 28 days equivalent strength ratios ($R_{e,\ 3}$) derived from steel fibers manufacturer's tables are found to be high by about 7.1% compared to the measured values (at 28 days) (refer to table 7-1). The average measured value from the sawn beams tested after 90 days is 56. Thus, the 90 days strength derived from the sawn beams is 14% greater than that derived from the steel fiber manufacturer tables. Some of that difference might be allocated to the strength growth either due to the hydration of the pozzolanic material contained in the mixture or to the increase of bond between steel fibers and concrete paste. These tables are found to be satisfactory for design purposes.

7.4.4 Modulus of Elasticity

E-values calculated from the third-point loading test are shown in table 7-4. The calculated values from sawn beams tested after 90 days agree with those found from cylinders under compressive strength tested after 28 days. The first method is believed to be better than the second one, because with one test many other properties can be assesses.

7.4.5 Mode of Failure

Figures 7-19 and figure 7-20 shows that the plain concrete beams fail suddenly while the SFRC beams fail slowly. The after-crack toughness can be explained from the multiple peaks associated with the load deflection curve for the SFRC. The sequence of the multiple peaks involves, that the steel fibers at the very bottom beam fibers initially hinder the first crack from growing to upper fibers. With increasing the load these steel fibers break down by either stretching or pulling out and the crack will grow to an upper level through the beam depth, once again another steel fiber does same above and so on till the beam fails completely. The breaking down of the steel fibers was evident from the breaking sounds heard during the test.

7.4.6 Empirical Formula for First Crack Strength

A number of empirical formulae have been suggested to relate flexural strength (f_{ct}) and compressive strength (f_{cu}). The following formula at Eq.7-1 is given by many design catalogues [11][39]

$$f_{ct} = 0.393(f_{cu})^{2/3} \Longrightarrow \text{Eq.7-1}$$

Where:

f_{ct} = Flexural strength.

f_{cu} = Compressive strength

From compressive strength data after 28 days and after 90 days, it is found that the equation is over estimating the flexural strength by about 5.7% and 25.5% of the SFRC and plain concrete respectively. Obviously the equation can be used for design purposes to estimate the flexural strength of the SFRC, which is beneficial in the sense that the compressive strength test is easy to conduct and cheap compared to the third-point loading test.

7.5 Conclusions

7.5.1 Full-scale Slabs

- For plain concrete ground slabs with a depth of 150 mm a theoretical reduction of 16.6% in depth could be achieved by adding ($15kg/m^3$) steel fibers to its mix. Theoretically the reduction (16.6%) is constant. Further practical tests are required to investigate the consistency of that reduction with other depths.
- Keeping in mind that the SFRC is thinner by 16.6%, the measured load capacity and deflection and the observed failure of the SFRC and plain concrete slabs for the interior, edge, and corners at 150 mm are found to be approximately equal.
- The results obtained from corners loaded at 300 mm are suspicious. Further tests should be conducted to investigate its load capacity and deflection characteristics.
- The mode of failure characteristics of the SFRC slab is marginally affected by the addition of steel fibers. The brittle behaviour of the plain concrete corner is altered to a more elastic behaviour when adding the steel fiber to concrete. This could have influence on concrete ground slabs, because the breaking off and shattering of corners could then be reduced if not completely overcome by using SFRC. Further investigation is required.
- Results could be affected by the foamed concrete support stiffness and failure mode. Further investigation is required.

7.5.2 Cores

- The strength variation among SFRC cores was expected to be higher than that for the plain concrete. Tests revealed that variation is higher among the plain concrete specimens, therefore higher consistency was achieved for SFRC than that for plain concrete.
- Gradual mode of failure is found for the compressed SFRC specimens, while brittle failure is found for the plain concrete ones.
- 14.5% greater strength gain in compressive strength was found after 90 days for the SFRC cores compared to plain concrete cores. The gain is due to the increasing of the bond between steel fibers and the concrete paste at the interfacial surface.

- Conversion formulas (for cores without steel bars) developed for plain concrete to convert core strength into cube strength is found applicable to the SFRC. Further tests should however be conducted to refine the formula.

7.5.3 Sawn Beams

- After 90 days the first crack flexural strength and strength at maximum load of the SFRC was found 40 and 35% greater respectively compared to plain concrete.
- The equivalent strength ratio derived from sawn beams tested at 90 days and beam specimens tested at 28 days were 60 and 35 respectively. The manufacture values at same fiber type and content is 42%. Reason for late strength development might be attributed to the pozzolanic material used in the mix. The manufactures tables are considered satisfactory for design purposes.
- The Japanese standard for determination of the SFRC toughness is deemed to give satisfactory and reliable results.
- Test results indicate that the modulus of elasticity calculated from data developed by third-point loading test is satisfactory.
- Gradual type of failure is observed for SFRC (unlike the plain concrete).
- Conversion formula to convert cube strength into flexural strength for plain concrete is found applicable to the SFRC. Further tests should however be conducted to refine the formula.