Chapter 5

Experimental Procedure

5.1 Introduction

The literature review revealed conflicting and / or gray areas in some aspect of SFRC. The influence of the steel fiber on compressive strength was not well established. Previously conducted flexural strength tests were not quite clear in describing the flexural strength of the SFRC, more over some studies once again did not agree on the influence of the steel fiber on the strength at first crack and at maximum load. Although there is a wide agreement about using the Japanese method to interpret and calculate the toughness of the SFRC [18], results published are limited.

Results for only a few full-scale tests were found. Some results of these tests were suspect either due to the testing equipments limitations or due to problems with the testing. The failure patterns were not clarified. Tests were conducted using steel fiber contents of 20kg/m^3 and more, while the effect of low dosages was not tested. Above all, a full-scale study considering the three load conditions were not found. The only study found to consider these three load conditions was conducted on relatively small slabs.

In the light of the above-mentioned issues, a comparative study containing two elements has been conducted. The first is an experimental approach and the second an analytical approach. The behaviour of SFRC is compared to that of normal concrete containing no fibers.

5.2 Mix Composition

The concrete mixtures used in this investigation are based on the mixture shown in table 5-1. Six mixtures having steel fibers dosages of 0, 10, 15, 20, 25, 30 kg/m^3 were manufactured respectively. Steel fibers (CHD 80/60 NB) used in this investigation were hook-ended wires with an aspect ratio of 80, length of 60 mm and a tensile strength of 1100 MPa. Fly ash was used as cement replacement to improve the workability of the mixture and to increase the mixture paste content.

Table 5-1: Constituents of the Used Mix

Material / Content	Cement	Water	Pozzfil	Stones		Crusher	Filler
				19 mm	13 mm	Sand	Sand
Mass (kg/m^3)	282	194	78	833	222	662	72

5.3 Effect of Steel Fiber Content on Properties of Concrete

The effect of steel fiber dosage on workability, compressive strength, modulus of rupture (MOR), modulus of elasticity, first crack strength and equivalent strength ratio ($R_{e,1.5}$) was studied. The following tests were conducted on the mixtures described in section 5.2:

- Standard slump test on all the mixes used in this investigation
- Standard compressive strength test after 7 and 28 days.
- MOR standard test on specimens tested after 28 days
- Third-point loading test with special setup on Material Testing System (MTS) on standard cast beams.
- Static modulus of elasticity test.

5.3.1 Standard Slump Test

Six mixtures containing steel fiber contents between 0 and 30 kg/m^3 were tested to measure their workability. The standard apparatus was used and the standard procedure was followed as prescribed by (Standard Method: SABS Method 862:1994) [78].

5.3.2 Standard Compressive Strength Test

Compressive strength test was carried out after 7 and 28 days the standard cubes (150x150x150 mm). Three specimens (for every mixture) were tested after 7 days and three other specimens tested after 28 days. The procedure is prescribed by (Standard Method: SABS Method 863:1994) [79].

5.3.3 Standard Flexural Strength Test

Standard flexural strength tests were carried on three specimens according to Standard Method: SABS Method 864:1994 [80] (for every mixture). The beams were tested after 28 days on their side in relation to the as-cast position. The failure load is determined and modulus of rupture (MOR) is calculated on the basis of ordinary elastic theory viz:

$$MOR = \frac{PL}{bd^2} \Longrightarrow \text{Eq.5-1}$$

Where :

MOR = Modulus of rupture.

P = Maximum load.

L = Span.

b = Width of the beam.

d = Depth of the beam.

5.3.4 Standard Modulus of Elasticity Test

Static compression standard test was conducted on two cylindrical specimens for each mix (mixtures in section 5-1). The procedure given by the ASTM C 469-94a was followed [81].

5.3.5 Third-Point Loading Test on Standard Beams

Three standard beam specimens for each mix (150x150x500 mm) were tested after 28 days to determine their first crack flexural strength and toughness characteristics.

A Closed-Loop Material Testing System (MTS) was used in displacement control to apply the load, measure the applied load and record deflection. Displacement was applied in a rate of 0.02mm/sec and 10 readings per second were recorded. The test setup is shown in figure 5-1. Mid-span deflection was measured by using two Linear Voltage Displacement Transducer (LVDTs) (mounted on a rig) reading against a clamp fixed to the specimen in a way that minimizes the error. The rig was fixed to the beam's neutral axis (assumed zero stresses surface) by means of 4 screws adjusted on the centerline of the two supports. Swiveling rollers were used at one of the supports and one loading point to accommodate any probable specimen deformations prior testing.

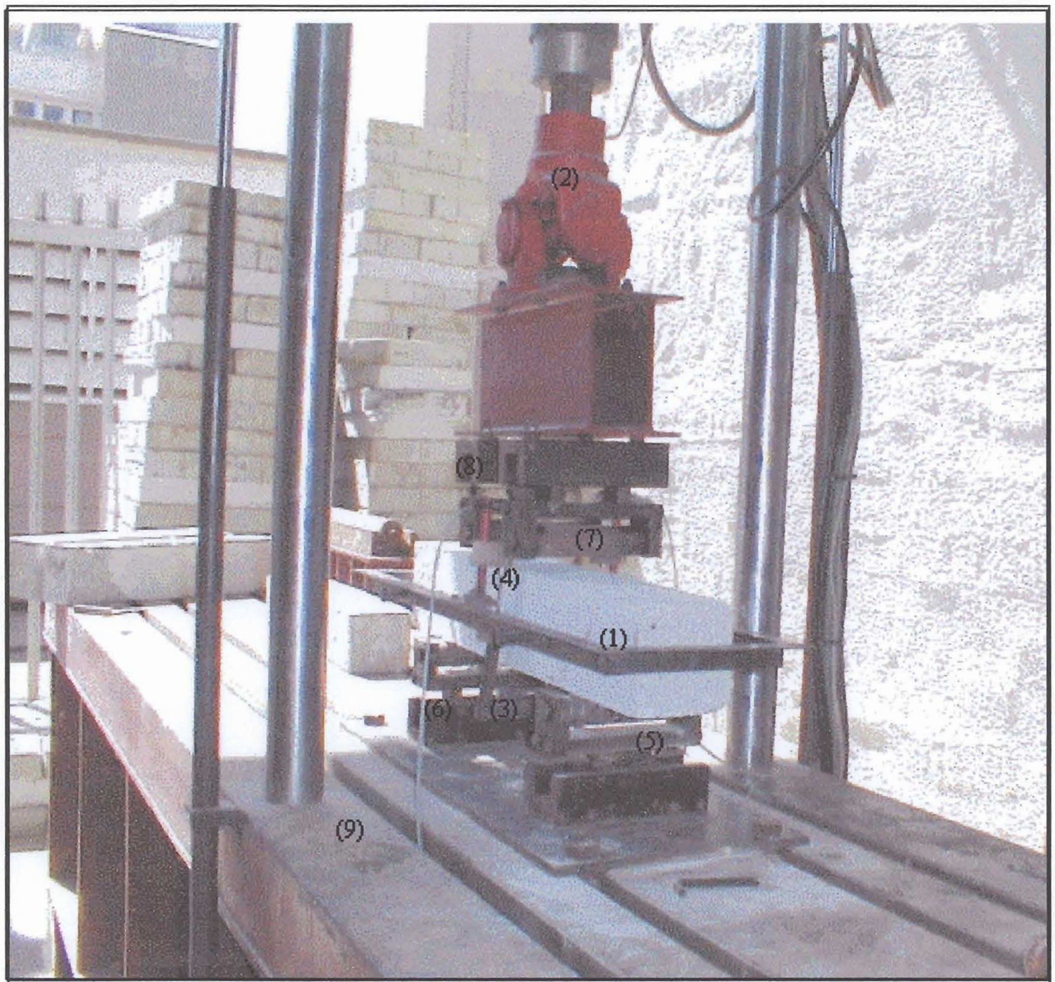
The load was applied by using two bearing rollers (one of them a swiveling roller) 150 mm apart with their center lines 75 mm from center of the beam. The beam supports were bolted to the (MTS) body and set 450 mm apart.

The number of steel fibers at the cracked section was counted to ensure that the steel fibers were well distributed in the specimen under consideration.

First crack load was estimated as the load point at which the load-deflection curve deviate from linearity. The technique used is described in Appendix C. The first crack strength was calculated by substituting the first crack load in equation 5-1.

Japanese standard method (JSCE-SF4) was used to calculate equivalent strength and equivalent strength ratio. The following steps were followed:

- Area under the load-deflection curve up to deflections of 1.5 mm (tested span is 450 mm) was calculated. Numerical approach (trapezoidal method) was used, by dividing the area under the curve into trapezoids. The area up to a certain deflection is the sum of the trapezoids area up to that deflection.
- The Equivalent load is then calculated by dividing the area by the deflection up to which the area is calculated.
- The equivalent strength and equivalent flexural ratio were calculated using equation 2-.2 and equation 2-3 presented in section 2.1.2.



(1) Steel rig.
(2) M.T.S loading arm.
(3) Griping clamp.
(4) LVDT 's clamp (held on the rig).
(5) Swiveling support.
(6) Fixed support.
(7) Fixed loading cylinder.
(8) Swiveling loading cylinder.
(9) M.T.S. body.

Figure 5-1: Test Set-up: Third-Point Loading Test

5.4 Slab Test

The test contains four elements:

- Major test on two full-scale slabs subjected to interior, edge, corner loaded at 150 mm from its angle bisector and corner loaded at 300 mm from its angle bisector.
- Plate-bearing test.

- Compressive strength test on cores drilled from the two slabs.
- Third-point loading test on beam specimens sawn from the two slabs.

5.4.1 Full-scale Test

Endeavoring to get similar behaviour for the two slabs, the depth of the SFRC slab was reduced by16.6% in comparison with the plain concrete. According to steel fiber manufacturer design tables (refer to tables in Appendix B) the flexural strength of the SFRC slab is improved by 42% relative to plain concrete strength. Hence the SFRC depth should be reduced in order to get similar behaviour for both slabs. The following criteria were followed:

$$f = \frac{M}{Z} \quad \Longrightarrow \text{Eq.5-2}$$

$$Z = \frac{bd^2}{6}$$

Where :

f = Flexural strength.

M = Moment of resistance.

Z = Modulus of section.

b = Width.

d = Depth.

Equation 5-2 was used to calculate for both slabs the following:

For SFRC slab :

$$f + 0.42f = \frac{6M_s}{(d_s)^2} \quad \Longrightarrow \text{Eq.5-3}$$

For plain concrete slab :

$$f = \frac{6M_p}{(d_p)^2} \quad \Longrightarrow \text{Eq.5-4}$$

Where :

M_s M_p = Moment of resistance of SFRC and plain concrete slab respectively.

d_s d_p = Depth of SFRC and plain concrete slabs respectively.

f = Flexural stength of plain concrete.

By equating equation 5-3 and equation 5-4, the two slabs yield equal moment of resistance. Therefore similar behaviour:

$$(d_p)^2 = 1.42(d_s)^2 \Longrightarrow \text{Eq.5-5}$$

It is obvious from equation 5-5 that, 125.8 mm depth for SFRC slab is equivalent to 150 mm plain concrete slab.

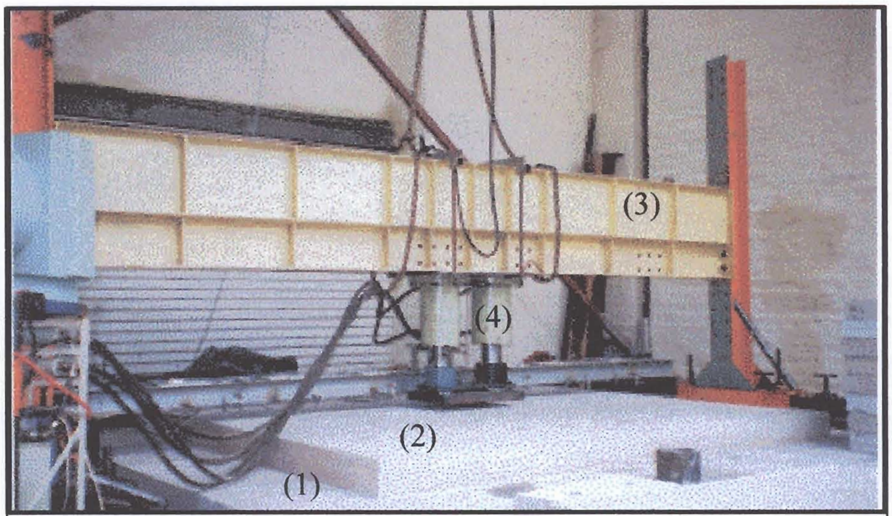
Full-scale tests were conducted on two ground slabs subjected to interior, edge and corner loading. The first slab was SFRC (3.0x3.0x0.125m) and the second plain concrete (3.0x3.0x0.15m). The interior points were tested after 28 days and the edges and corners tested after 90 days. A 150 mm thick foamed concrete subbase cast on top of 1000 mm deep solid concrete floor.

The SFRC slab has identical mixture to that of the plain concrete slab refer (table 5-1). 15 kg/m^3 of steel fibers was added to the plain concrete mixture to cast the SFRC slab.

A closed-loop material testing system was used in displacement control (1.5 mm/ min.) to apply the load through a hydraulic twin jack and record the data (100 readings/sec) from LVDTs sited at certain spots. A single steel loading plate (100x100x50mm) was placed centrally beneath the jack's load transferring plate. The devices and instruments used in the test can be seen in figure 5-2 and figure 5-3. Eight different test setups were used in this investigation and details of the different setups can be seen in figure 5-4. The setup for four load cases is considered:

5.4.1.1 Interior load (Test 1)

Seven LVDT's were used to measure the deflection at loading point and six other locations. One LVDT was adjusted and located to measure the vertical displacement beneath the loading point while the other six LVDTs were located at 300 mm intervals as shown in figure 5-4/Test 1. The LVDTs were clamped on a steel beam simply supported next to the foamed concrete sub base. Figure 5-2 shows the general setup for the interior load case.



(1) Foamed concrete sub base. (3) Sliding frame (to hold the jack).
(2) Loaded slab. (4) Hydraulic twin jack.

Figure 5-2: Photo Shows the General Test Setting: Full-scale Test (Interior load)

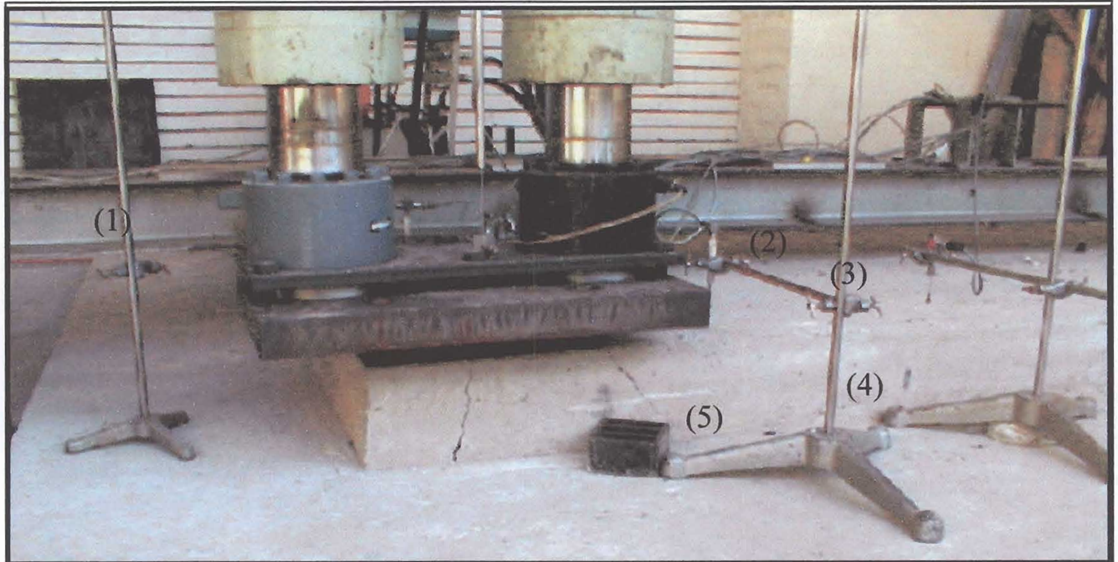
5.4.1.2 Edge load (Test 2)

Six LVDTs were used as shown in figure 5-4/ Test 2. The Procedure is described in section 5.4.1.1. The practicality of moving the jack around the slabs limited the possible test positions to one set of edges.

5.4.1.3 Corner load (Test 3 and 4)

Two sets of corners were tested. The loading points were chosen at 150 mm and 300 mm from the corner angle bisector for each set of corners respectively. See figure 5-4/Test 3 for corner at 150 mm and figure 5-4/Test 4 for corner loaded at 300 mm. It was aimed to evaluate the influence of the loading position relative to the corner.

Figure 5-3 shows the general test setup for corner load. Three LVDTs were clamped on a steel beam, having the same setting as in section 5.4.1.1 above. The other three LVDTs were mounted on a clamp fixed to a steel footing. One LVDT was adjusted and located to measure the vertical displacement beneath the loading point while the rest were located at points where failure is likely to be detected.



(1) Steel beam (used to hold LVDTs). (2) LVDT. (3) Clamp.
(4) Steel footing. (5) Loading plate.

Figure 5-3: Photo Shows the General Test Setting: Full-scale Test (Corner Load)

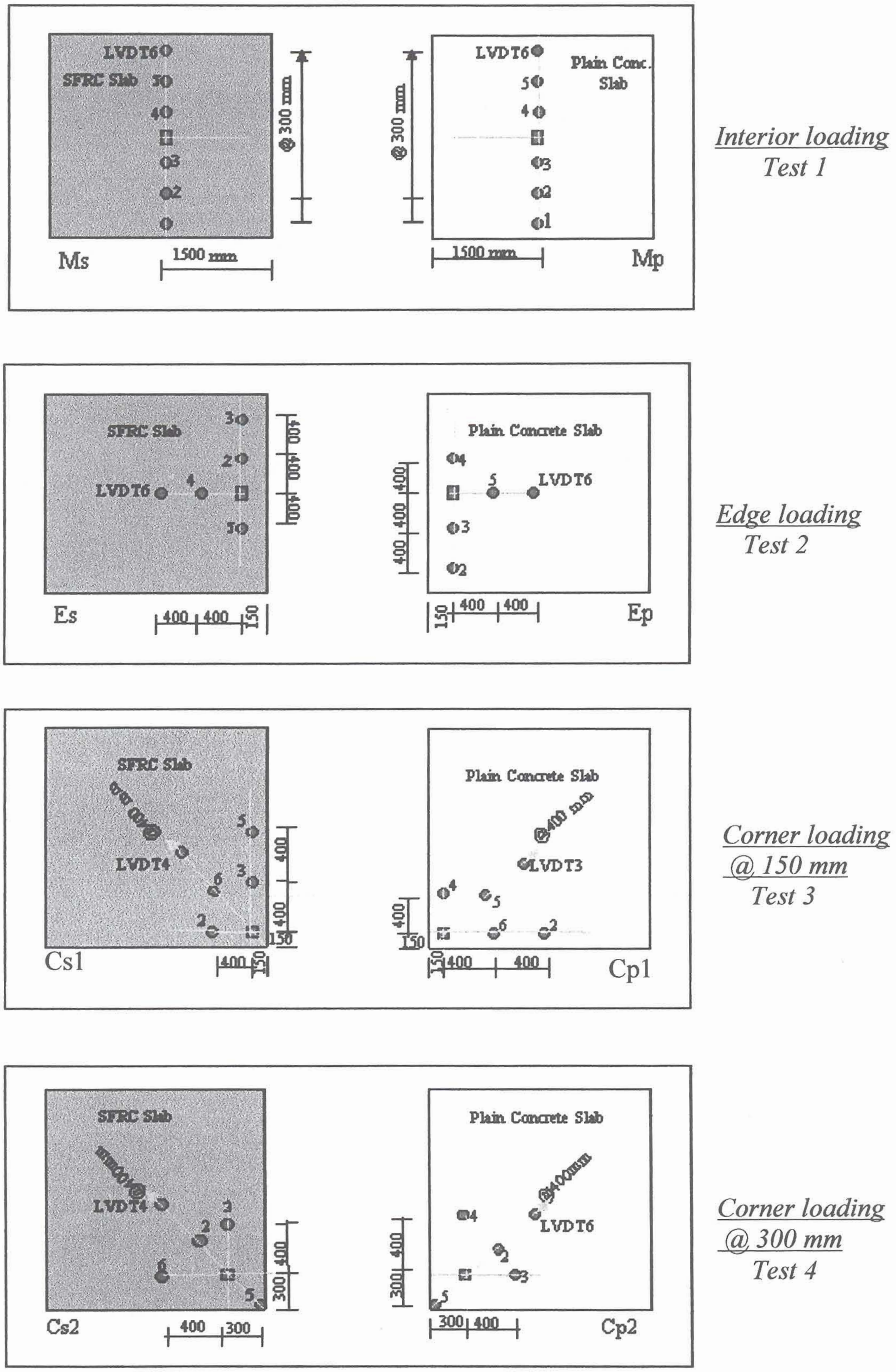


Figure 5-4: Loading and Measuring Points: Full-scale Test

The load-deflection relation was established for the two slabs for each case of loading. Spot readings every 10 KN were used to eliminate the effect of noise recorded. The first crack was estimated as the point at which the load-deflection curve first deviates from linearity. Differentiation techniques were used to estimate the first crack (sample of its assessment is found in Appendix C). The highest recorded load is assumed to be the maximum or "failure load". The readings of the LVDTs at first crack and "failure" were used to establish a deflection profile for each set of test. Mode of failure for each test was visually established during testing

5.4.1.4 Comments on the Slab Setup

The test setup used for the full-scale test has the following shortcomings and limitations:

- The foam concrete sub base was very hard and might not simulate the field conditions. The measured K-value is very high compared to those used for the normal pavements. The deformation behaviour might also differ. The negative pressure associated with the normal compacted layer is not possible or not likely to take place with the foamed concrete, because of the high compressibility due to high void ratio found in the foamed concretes.
- Slabs were constructed adjacent to each other and the sub base was cast as one unit. Therefore the effect of the loading on the first slabs might extend to create a residual stresses and strains on the sub base of the second slab, which can affect the load capacity of that second slab.
- Three LVDTs were mounted on steel footings and these steel footings were put on the foamed concrete sub base. The readings of these LVDTs might be affected while the specific slab was under loading.
- The LVDT needles were rested freely on the top surface on the slabs to measure the vertical deflection. These needles should have glued to the slabs to eliminate the effect to surface roughness. This mistake was duplicated in all tests; therefore its effect might not have a significant influence in comparing the slabs.
- The practicality of moving the jack around the slabs limited the possible test positions to one set of edges and two sets of corners.
- The two slabs were cast on an enclosed environment having walls on three

sides while the fourth side was open. The SFRC slab was cast closer to the open side. The two slabs therefore did not have identical curing conditions. The SFRC slab had a longer sun exposure time and more exposed to the rain than the plain concrete slab. That might slightly affects the results.

- The available space for testing was limited; therefore, larger slabs were not possible.
- Only semi static load (low rate loading application) was applied and no provision was made for the cyclic loading.

5.4.2 Plate-Bearing Test

A plat-bearing test was conducted after 28 days to estimate the modulus of reaction for the foamed concrete sub base. The closed-loop system used in section 5.4.1 was used to apply the load and measure the deflection at the loading point. A loading plate having 250 mm diameter was used as a bearing plate. The following steps were followed to calculate the K-value as required by Westergaard [11]:

- A stress-deflection curve was established and the stress at a deflection of 1.25 mm was obtained from the curve.
- K_{250} is calculated by dividing the obtained stress by1.25.
- The recommended bearing-plate diameter is 750 mm. Therefore; a correction factor was applied to convert K_{250} into K_{750}. The experimental diagram of Stratton in figure 5-5 was used to read off the correction factor corresponding to the plate diameter of 250 mm (Factor = 2.55).

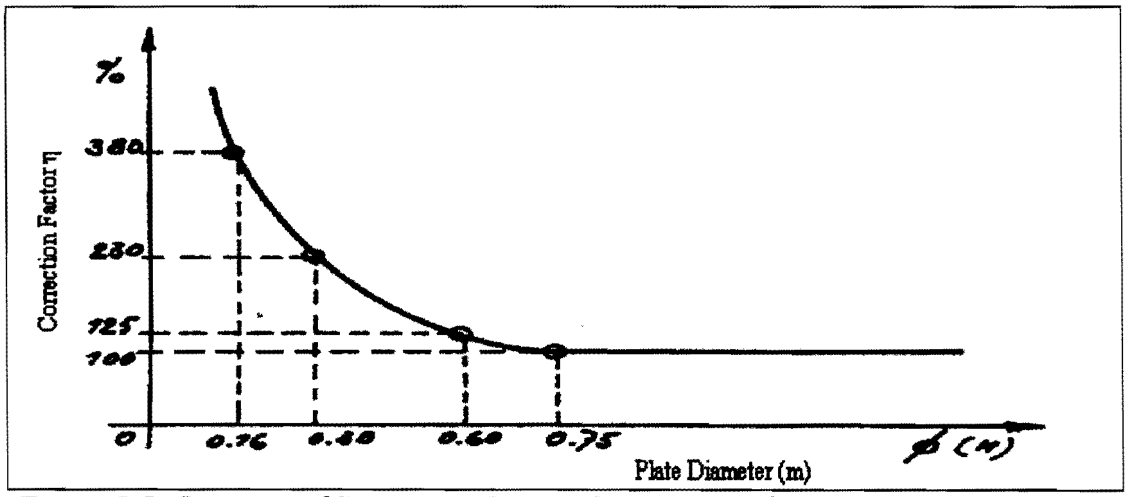


Figure 5-5: Diagram of Stratton to Correct for Bearing-Plate Size

- The following formulas are used to calculate K-value

$$K_{250} = \frac{\sigma_{1.25}}{1.25} \Longrightarrow \text{Eq.5-6}$$

$$K_{750} = \frac{K_{250}}{\eta} \Longrightarrow \text{Eq.5-7}$$

Where :

K_{250} = Modulus of subgrade reaction using bearing - plate of 250 mm diameter.

$\sigma_{1.25}$ = Stress at deflection of 1.25 mm.

K_{750} = Corrected modulus of subgrade reaction for bearing - plate of 750 mm diameter.

η = Stratton correction factor.

The bearing-plate test was only performed after 28 days and it would have been better if the test was repeated after 90 days to calculate K-value (at 90 days) for the theoretical analysis of the edge and corner load cases.

5.4.3 Core Test

Twelve cores of 100mm diameter were taken at 60 days from the two slabs (six cores each slab). The cores were drilled and capped according to SABS Method 865 [82]. The cores were caped to approximately 100mm length and stored in a constant room temperature (22C°) and subjected to a standard compressive strength test after 90 days (the time at which the slabs were tested). This core strength was converted to actual and potential standard cube strength using the conversion formula given in the British Concrete Society Technical Report No. 11 [21]. The following are the formulas:

$$f_{act.} = \frac{2.3 f_\lambda}{1.5 + \frac{1}{\lambda}} \Longrightarrow \text{Eq.5-8}$$

$$f_{Pot.} = \frac{3.0 f_\lambda}{1.5 + \frac{1}{\lambda}} \Longrightarrow \text{Eq.5-9}$$

Where :

f_{act} = Actual cube strength.

f_{pot} = Potential cube strength.

f_λ = Core strength.

λ = Ratio of length to diameter of the core

5.4.4 Third-Point Loading Test on Sawn Beams

Similar test setup that in section 5.3.3 was used to test four sawn specimens (two from each slab). Specimens were stored in a constant temperature room for three weeks; thereafter, third-point loading tests were conducted. Endeavoring to simulate the field curing conditions and loading direction, the four specimens were tested dry and loaded on the casting side.

The load-deflection relation was established for each individual tested specimen. The first crack load was estimated as the load at which the load-deflection curve deviates from linearity. (Refer to appendix C for sample of calculation). The highest recorded reading for the load is considered as a maximum load "failure load". The first crack strength was calculated by substituting the first crack load in equation 5-1. Equivalent flexural load, strength and equivalent flexural ratio were calculated using the Japanese method as described in section 5.3.2. In addition to that, modulus of elasticity was calculated using equation 2-5 in section 2.1.7 sample of calculation is given in Appendix D.

5.5 Theoretical Analysis

To compare the theory and practice, the following theoretical models were used:

- Westergaard for the first crack load and elastic deflection.
- Meyerhof for the ultimate load capacity.
- Falkner et al for the ultimate load capacity.
- Shentu et al for the ultimate load capacity.

The slab properties were measured, calculated or assumed. The casting depth was taken as the slab thickness. The first crack strength and equivalent strength ratio were measured from the third-point tests conducted after 28 days on beam specimens. Modulus of elasticity was calculated from a third-point test described in section 5.4.4. For the purpose of this research, Poisson's ratio and direct tensile strength were estimated to be equal for both SFRC and plain concrete. The K-value after 28 days was measured as described in section 5.4.2. After 90 days (edge and corner tests) greater K-value was estimated. The higher estimated K-value is because of the expected growth of its value because of the pozzolanic material contained in the foam concrete sub base.

The above-mentioned properties were used as inputs to Westergaard, Meyerhof, Falkner et al and Shentu et al to calculate the interior load capacity for the two slabs. Westergaard and Meyerhof were further used to calculated edge and corner load. Westergaard formulas for interior, edge and corner deflection were used to calculate the elastic deflection relevant to Westergaard load. Models are given in chapter 4.

Further calculations were performed considering various K-values. K-values ranging between 0.015 and 0.4 MPa/mm were assumed and used together with the slab's properties to calculate the load capacity and deflection as described thereof. Sample of calculations are presented in appendix E.