3. EXPERIMENTAL WORK – SHEAR IN COBIAX SLABS

3.1. INTRODUCTION

In this chapter the task was to compare theoretical calculations for the shear strength of Cobiax slabs (discussed in Chapter 2) with force controlled shear tests performed on laboratory Cobiax slab specimens. This comparison had to be conducted to establish the shear strength reduction factor for a Cobiax slab, compared with a solid slab with the same thickness, tension reinforcement and concrete properties.

A Cobiax shear strength reduction factor of 0.55 times (Schellenbach-Held and Pfeffer, 1999) the shear strength of a concrete slab without shear reinforcement had been calculated at the Technical University of Darmstadt (TUD) in Germany. The Cobiax steel cages were omitted in the TUD tests. The objective of this chapter was to demonstrate that the presence of the steel cages holding the Cobiax spheres in position during construction, will act as shear reinforcement inside the slab, resulting in a less conservative shear strength reduction factor.

This method of multiplying the shear capacity of a solid slab with a shear strength reduction factor to obtain the shear strength of that slab with internal spherical voids, is a simplified method best supported by empirical test results. This method seems to be the easiest way to support the design engineer with answers for shear in Cobiax slabs, and also a faster way to predict shear strength when conducting a cost comparison between different slab types, as done in *Chapter 4* of this report.

Predicting the shear behaviour in concrete slabs with internal spherical voids is actually far more complex and could probably best be approached with powerful finite element software using three dimensional brick elements and non-homogenous material (concrete and steel reinforcement). One could with multiple analyses of different scenarios (slab content and dimensions) develop formulae that are typical for concrete slabs with internal spherical voids. This approach or a similar complex approach will not be conducted for the purposes of this report.

The experimental work comprised of the testing of twelve beam specimens of equal length and width, but having varying thicknesses and quantities of tension reinforcement, some with Cobiax spheres, and some solid. All beams, simulating strips of 600mm wide flat-slabs, were designed to fail in shear before failing in flexure, to allow for conclusions to be drawn regarding their shear capacities.

The samples were manufactured in Bloemfontein and transported to Pretoria on the day prior to testing. Three 150 x 150 cubes and three 150 x 150 x 700 beams were also manufactured and then tested on the same day as the sample beams so that the representative 13 day compression and flexural strengths could be established.

Due to casting and laboratory constrains the tests had to be carried out 13 days after casting. However, the age of testing has little significance seeing that all the tests were carried out on the same day. All predicted capacities are also based on the 13 day concrete strengths.

3.2. PREPARATION AND EXPERIMENTAL SETUP

Experimental Design

- A total of twelve sample beams were manufactured, each beam having a length of 1500 mm and a width of 600 mm.
- Three solid beams (without Cobiax spheres) were cast as well, having depths of 280 mm. In these beams the tension steel content was varied, each one having 3, 4 and 5 Y16 bars, respectively.
- For the 180 mm diameter Cobiax spheres used in the other 9 samples, the concrete webs or spheres were spaced at 200 mm centres in every sample. The beams were therefore dimensioned to contain two whole spheres in the centre, and two half spheres at the sides of every Cobiax sample. Every beam cross-section therefore contained 3 identical webs, central to the beam. (See Figure 3.2.1).

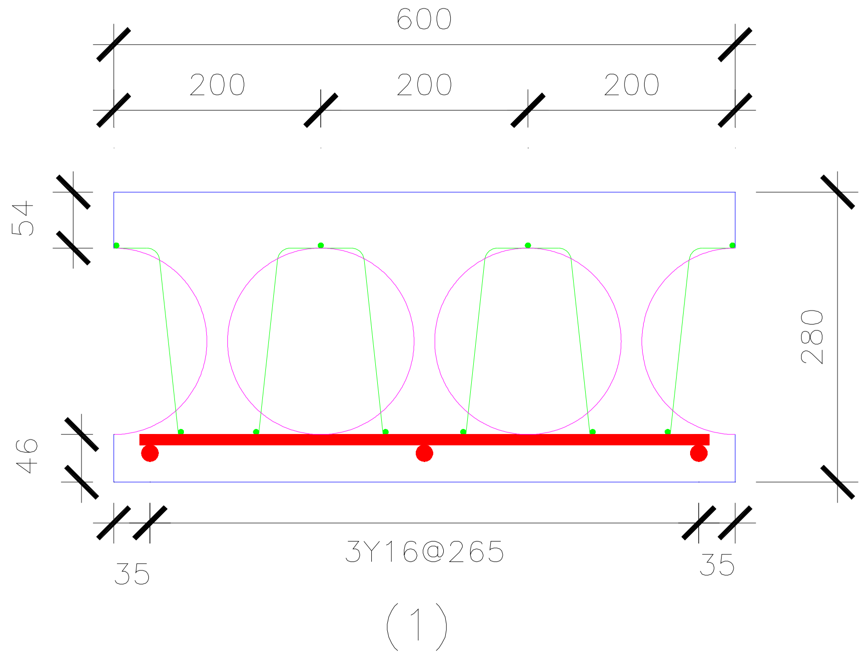

Figure 3.2.1 Cross section of a 280 mm thick Cobiax sample

- Three depths of 280, 295 and 310 mm were prepared for the Cobiax samples, with varying reinforcement quantities of 3, 4 and 5 Y16 bars for each depth. Details of the beams are presented in *Paragraph 3.5*.
- Cobiax cages (displayed in green in *Figure 3.2.1*) consist of 1 top and 2 bottom longitudinal bars, kept in place by transverse bars. Both the longitudinal and vertical bars of the cage will clearly contribute to the shear resistance. From a theoretical point of view these bars should be removed to obtain a true comparison between a solid slab and a voided slab containing spheres. However, this would result in some practical problems keeping the spheres in position during construction. On the other hand, the cages will always be present in a Cobiax slab, and it was therefore decided to use the Cobiax system exactly as it will be used in practice. It should be noted that vertical cage bars are not fully anchored around the main reinforcing bars when considering SABS 0144:1995 curtailment specifications. For this reason they will only partially contribute to the aggregate interlock capacity, and their contribution will reduce drastically after the welds between the vertical bars and bottom horizontal bars of the cages fail under large loads.

The following factors were considered in the parameter selection to investigate the design of the experimental setup:

- As stated in *Paragraph 2.2,* beams without shear reinforcement is likely to fail in shear before failing in flexure if the a_v / d ratio is less than approximately 6.

where:

a_v = distance of a single point load to the face of the support

d = effective depth of the tension reinforcement

It is therefore normal practice in beam design to provide shear reinforcement to increase the shear capacity so that flexural failure will happen before shear failure. The largest quantity of shear reinforcement will be required for an a_v / d ratio of approximately 2.5 to 4 (see discussion in Section 2). The a_v / d ratios for the beams were therefore kept within these limits to be able to produce conservative results. The actual ratios for the experimental beams are given in *Table 3.2.1*, with *H* the slab thickness.

Table 3.2.1 $\frac{a_v}{d}$ **ratios**

H (mm)	a_v (mm)	d (mm)	a_v/d
280	687.5	252	2.73
295	687.5	267	2.57
310	687.5	282	2.44

- For smaller $\frac{a_v}{d}$ ratios, arch action will increase the shear capacity of the beam, which is not desirable for the purpose of this research.
- The test apparatus was limited to a 600 mm wide slab.
- The beams had to be manufactured in Bloemfontein and then transported over a distance of 460km to Pretoria, having the effect of preparation of as small as possible samples to enable handling and transportation. The weight of every sample varied between 600 and 750 kg.
- The larger and heavier the samples were, the more difficult it would have been to position the beams correctly during the experimental setup.
- Budget constrains were also applicable.

The beams were simply supported with a span of 1350 mm (see *Photo 3.2.1* and *Figure 3.2.2*). Each sample's longitudinal centreline was aligned with the longitudinal centreline of the supports. The distance from the beam end to the centre of the support was 75mm. The knife edge load (P_u) was applied at the sample's midspan. The samples were tested in force control at a rate of 40 kN/min. Experience show that this rate is acceptable and will result in negligible dynamic effects. The failure criterion is easily observed with a sudden drop in the applied force with a deflection that remains constant. Throughout the test the applied loads at midspan, as well as the displacements, were measured at 25 readings per second (25Hz).

Photo 3.2.1: Experimental setup

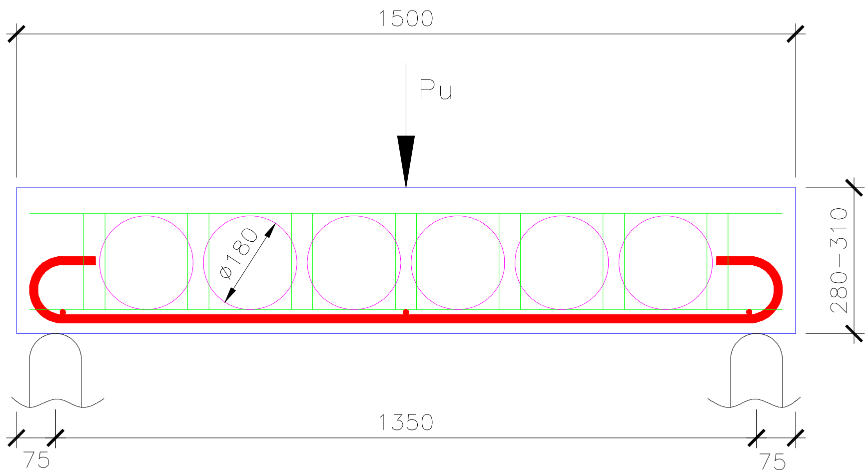

Figure 3.2.2 Experimental setup

The flexural capacity for each sample was calculated to ensure that shear failure would precede flexural failure. The results are presented in *Table 3.2.2*. *Figure 3.2.3* shows the results in graph format. These results are only an indication of the properties that will be required in the samples. The correct material properties are displayed later in this chapter. *Equations 2.4.1 to 2.4.3a* were used with all partial material safety factors set to unity.

- The slab thickness was varied by increasing the depth of the top flange, but keeping the thickness of the bottom flange constant for all beams. This was done to simulate construction practice.
- Reinforcement variation was decided on to assess the influence of tension reinforcement on the shear capacity.
- The reason for material factors being set to unity is to calculate the actual strength rather than the design strength.

Table 3.2.2 Comparison between moment failure loads and shear failure loads based purely on design values

SANS 10100					
fcu =	30	**MPa**	**Cover**	20	**mm**
fy =	450	**MPa**	**AY16 =**	201	**mm²**
b =	600	**mm**	**γm**	1.0	Material factor - Moment
L =	1350	**mm**	**γmc**	1.0	Material factor - Shear
Solid	**Height (mm)**	**d (mm)**	**Pm (kN)**	**Ps (kN)**	**Failure Mode**
280Y3	280	252	194	199	Moment
280Y4	280	252	254	219	Shear
280Y5	280	252	313	236	Shear
295Y3	295	267	206	204	Shear
295Y4	295	267	270	225	Shear
295Y5	295	267	333	242	Shear
310Y3	310	282	218	209	Shear
310Y4	310	282	286	230	Shear
310Y5	310	282	353	247	Shear
Pm =	Failure load for flexure (midspan point load)				
Ps =	Failure load for shear (midspan point load)				
Failure	**"Moment" =**	Beam will fail in flexure			
mode =	**"Shear" =**	Beam will fail in shear			

In *Table 3.2.2* the definitions of the symbols not explained in the table itself are:

fcu = characteristic concrete cube compression strength

fy = steel reinforcement yield strength

b = width of the specimen

L = span of the specimen

AY16 = area of a 16 mm diameter steel reinforcement bar

d = centroid depth of the tension reinforcement, measured from the top of the beam

The legends, for example *280Y3*, can be explained as follows:

280 = total thickness of the beam

Y3 = amount of steel reinforcement bars in the beam, spreaded over the 600 mm width

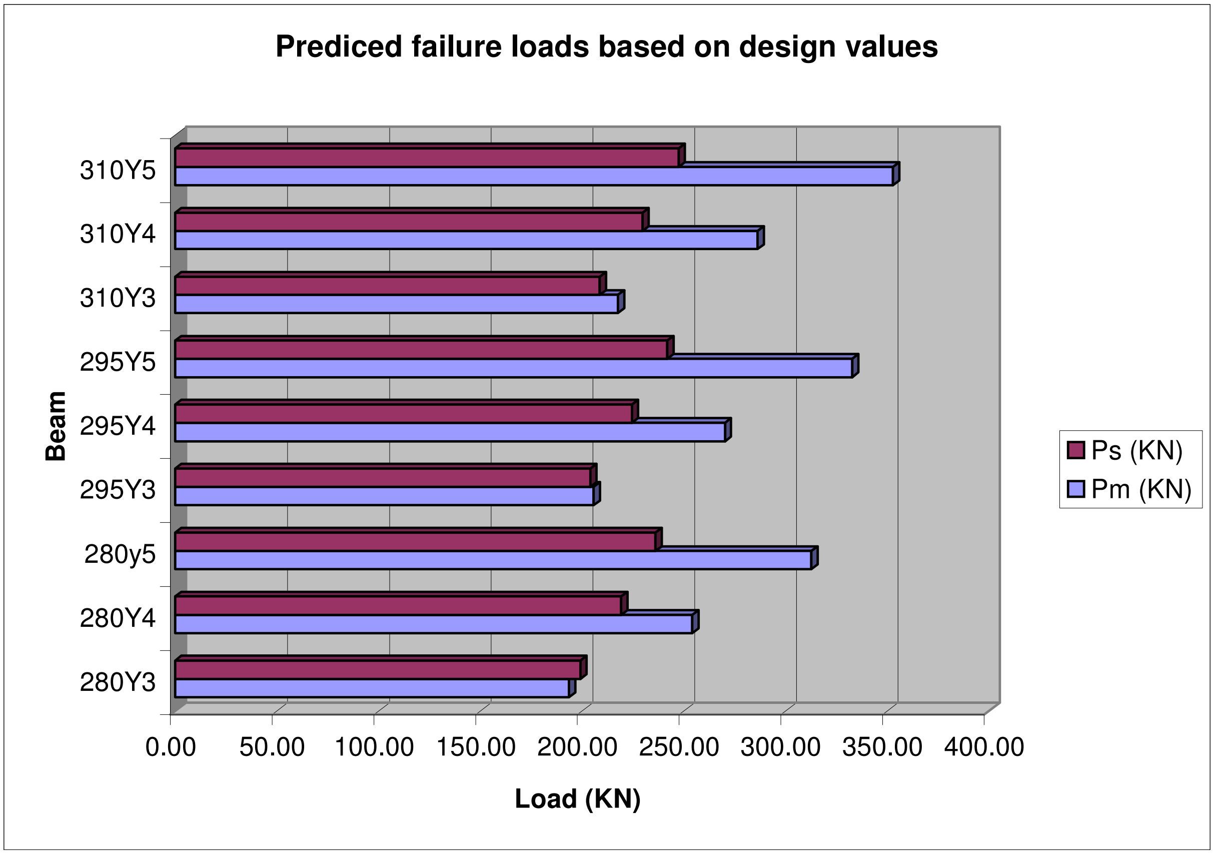

Figure 3.2.3 Predicted moment failure and shear failure loads based on design values

The Cobiax beams were expected to have a lower shear capacity than the solid beams. All calculations for the solid beams showed that shear failure would precede or happen simultaneously to flexural failure, and it was therefore concluded that the Cobiax beams would display a similar behaviour.

The depth of the stress block in flexure for the Cobiax beams never exceeded the minimum depth of the top flange during this research. For the 280 mm deep beam, the minimum depth of the top flange is 50 mm. The method used to design Cobiax slabs are for this reason the same as for solid slabs, where the presence of the voids only reduces the own-weight and slightly reduces the slab stiffness, as well as shear capacity.

The calculations indicated that the 280 mm solid slab with 3 Y16's (S280Y3) could fail in flexure before failing in shear. However, normally the flexural reinforcement will enter the work-hardening zone, and the flexural capacity will increase beyond that in shear. This configuration was accepted for this reason.

Sample Preparation

The samples were manufactured at Peri Wiehahn's premises in Bloemfontein. Following construction of the formwork, the tension reinforcement was positioned in the boxes, and the cages containing the Cobiax spheres were fixed to the tension reinforcement. The semi spheres were fixed to sides of the formwork boxes. Prior to casting, inspections were performed to ensure that all elements were correctly positioned in accordance with the design drawings.

The concrete was poured during the following day. A first concrete layer of approximately 70 mm was poured to ensure the tension reinforcement and bottom bars of the Cobiax cages were embedded by at least 20 mm. This prevented the spheres from floating to the top during casting, since they could not escape the cages that were then anchored in the bottom 70 mm of hardened concrete. This first concrete layer added sufficient dead weight to hold all components down during the second pour to the top of the slab. Lifting of cages would result in a smaller d value, that would extinguish the hope of any trustworthy results. The second and final pour was done approximately 4 hours later.

The second pour's concrete were utilised to construct the test cubes and beams, to ensure that a representative sample of the concrete forming the compression block (top concrete) was collected.

3.3. OBSERVATIONS

As can be seen from *Photos 3.3.1*, the shear cracks started from bending cracks in the case of the solid slabs. This is common for $2.5 < \frac{a_v}{d} < 6.0$.

In the case of the Cobiax slabs though, the crack sometimes started at the web, and then further developed down and back to the support along the tension reinforcement and also upwards to the top of the beam towards the line of load application. These observations are well justified by the predictions of Park & Paulay (1975). (See *Paragraph 2.2*)

Solid slab crack ***Cobiax slab crack***

Photo 3.3.1: Observed crack patterns at failure

3.4. RESULTS

The following table is a summary of the failure loads obtained for each sample.

Table 3.4.1 Beams tested and results obtained

S = Solid slab
C = Cobiax Slab
Y3 = 3 x Y16 bars
Load = Load applied by hydraulic press for failure to occur

Beam	Load (kN)
S280Y3	242
S280Y4	326
S280Y5	354
C280Y3	268
C280Y4	279
C280Y5	330
C295Y3	259
C295Y4	301
C295Y5	343
C310Y3	276
C310Y4	271
C310Y5	353

Figure 3.4.1 shows the failure loads of all samples compared to SANS 10100 characteristic shear capacity (with $\lambda_{mv} = 1$) calculated for a solid section. The solid and Cobiax samples all exceeded the predicted capacity. From these results it would appear as if no reduction in capacity is required for the Cobiax slabs. However, further investigations were required in terms of material properties before any such conclusions could be made.

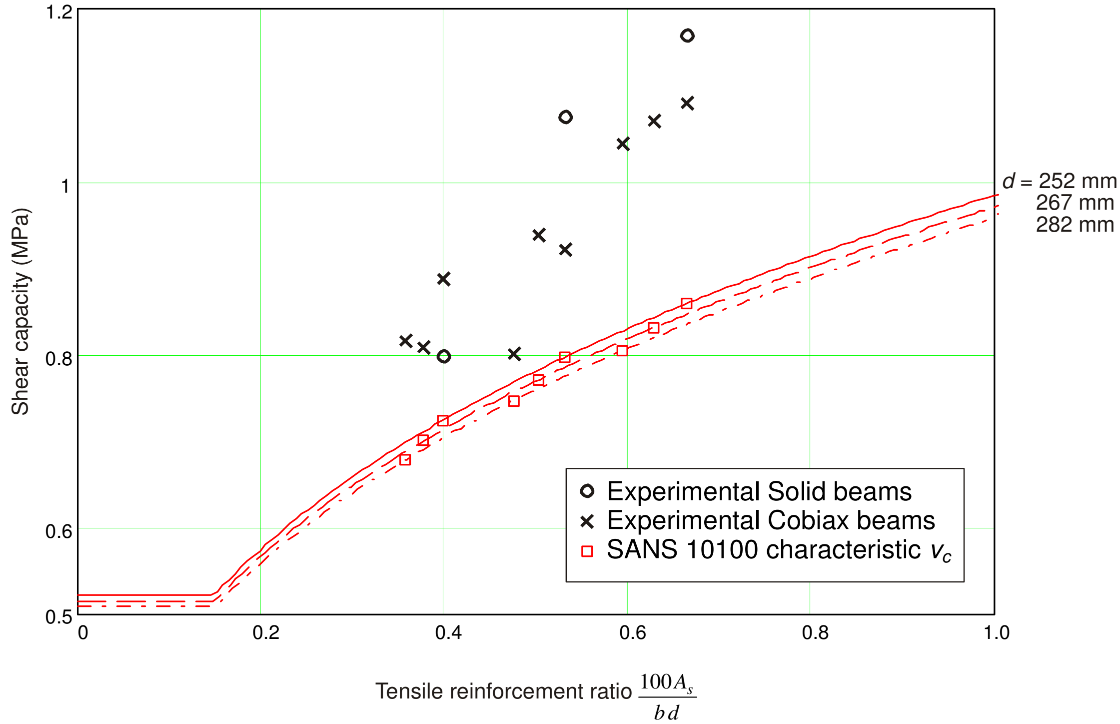

Figure 3.4.1: Failure stress of all beams compared to SANS 10100 characteristic shear capacity.

More detailed results are presented in the following sections, supported by a discussion on the observed behaviour.

Solid slabs

The load-deflection response of the solid slabs is illustrated in *Figure 3.4.2*. The behaviour is mostly brittle with an almost linear behaviour up to the peak load. After obtaining the peak load, there is a rapid reduction in resistance, characteristic of a shear failure. The exception is S280Y4 which exhibits a softening behaviour before reaching the peak load and a more gradual reduction in strength after reaching the maximum load.

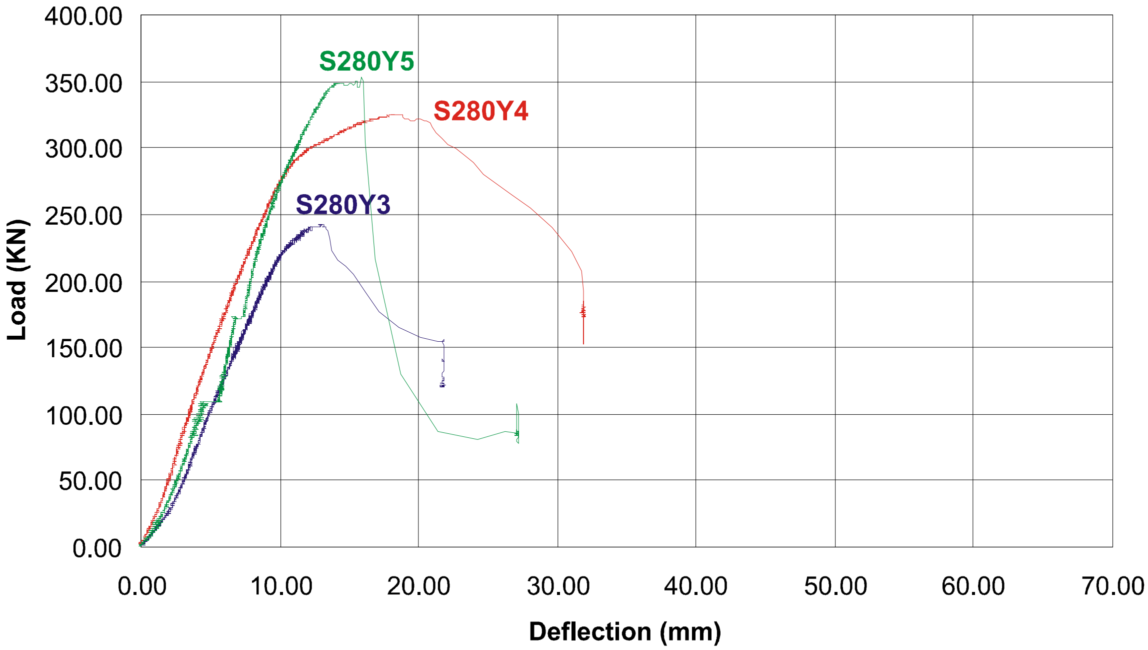

Figure 3.4.2: Load-deflection response of solid slabs

Figure 3.4.3 compares the experimental shear strength to characteristic values predicted by three design codes of practice discussed in *Chapter 2*, with material properties presented in *Table 3.2.2*. This figure clearly illustrates that the shear strength of beam S280Y3 is lower than expected and does not follow the anticipated trend. The reason for the difference could be a result of the typical scatter expected from experimental shear tests as discussed in *Paragraph 2.2*. Although the shear capacity for this beam is above that predicted by BS 8110 and SANS 10100, EC2 over predicts its strength. It is concluded that this beam had a lower than average shear strength.

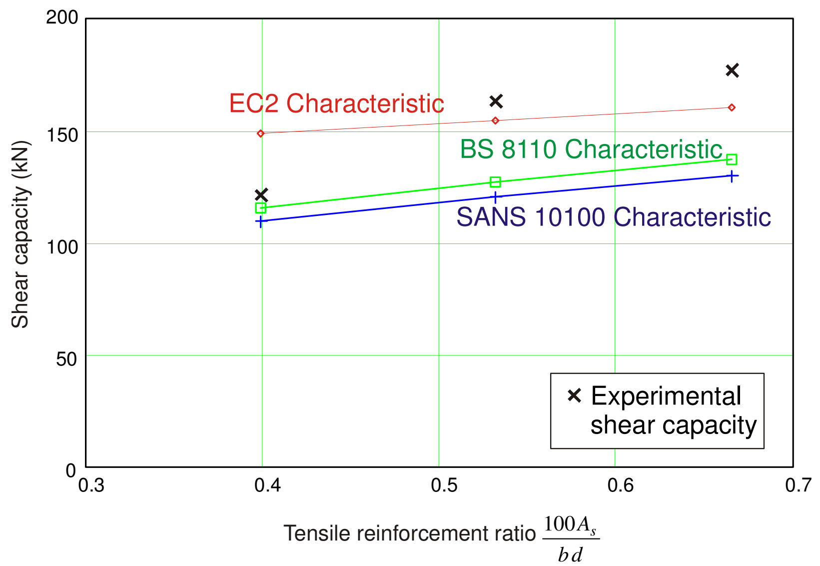

Figure 3.4.3: Shear capacity of 280 mm solid slabs compared to characteristic predicted values

Comparison of solid and Cobiax slabs

Figures 3.4.4 to 3.4.6 compare the load-deflection responses of 280 mm solid samples to that of the Cobiax samples. The peak loads achieved by the solids samples were higher than that of the Cobiax samples with the exception of one specimen, S280Y3.

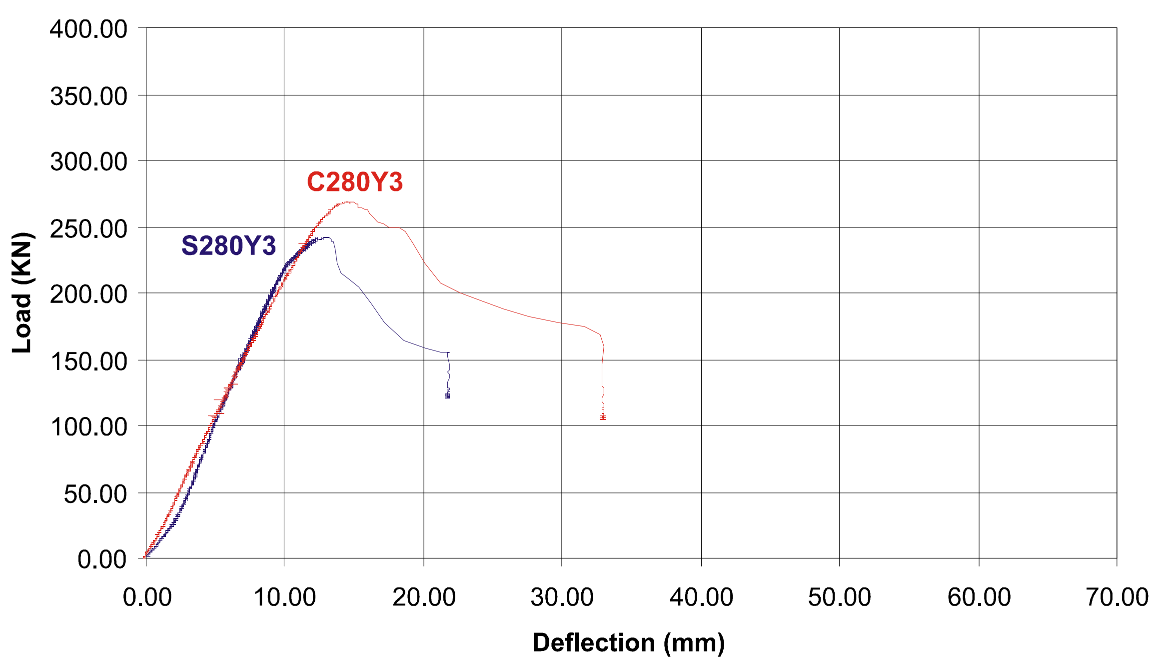

Figure 3.4.4: Load-deflection response of 280 mm slabs with 3 Y16's

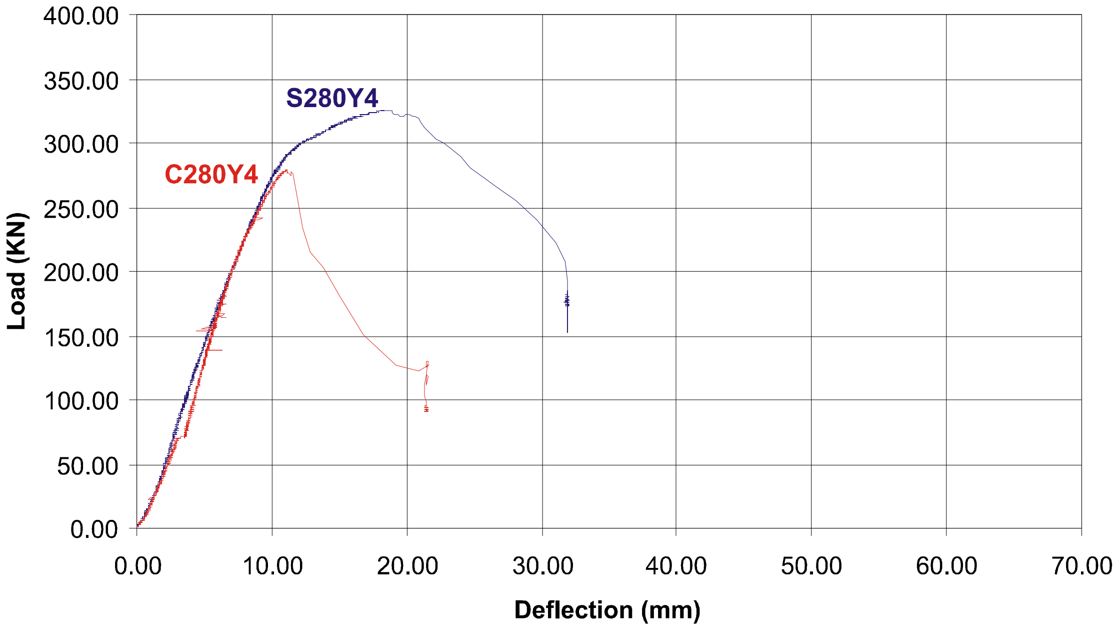

Figure 3.4.5: Load-deflection response of 280 mm slabs with 4 Y16's

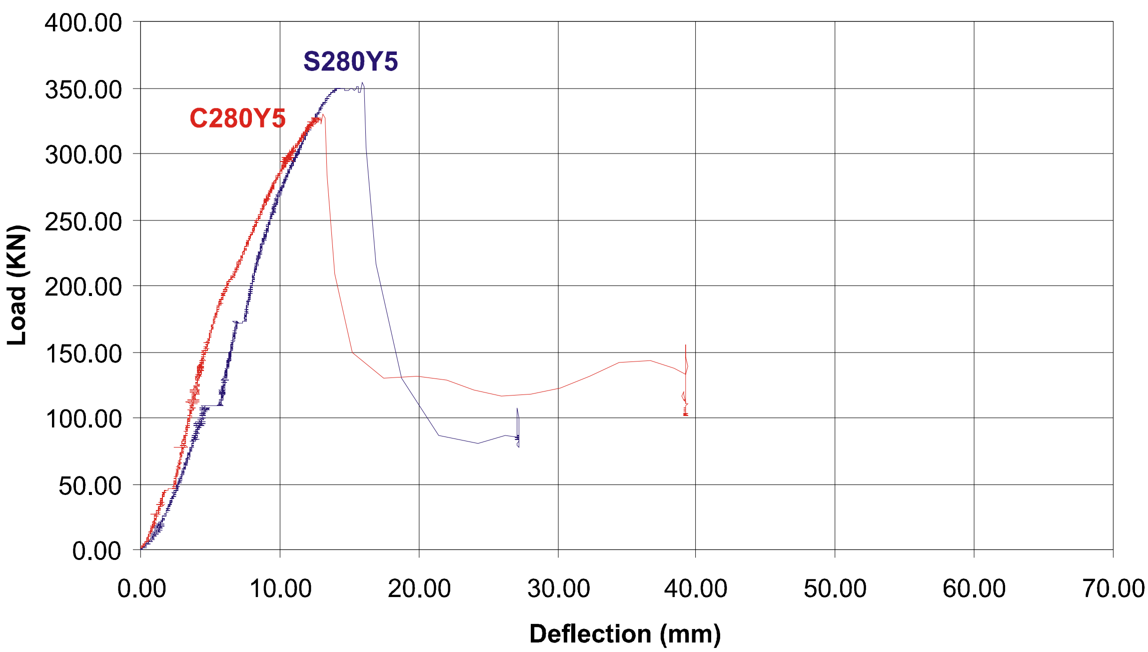

Figure 3.4.6: Load-deflection response of 280 mm slabs with 5 Y16's

The minimum Cobiax to solid slab capacity ratio obtained was 0.857 MPa.

Interesting to note is that the Cobiax slabs (see *Figures 3.4.4 to 3.4.8*) also resist the applied loads up to certain peak values, yet then tend to display more ductile behaviour than solid slabs without shear reinforcement, for two out of three cases, as the load decreases. This behaviour could also be seen during a sample test, where the solid samples began to show shear cracks and then suddenly collapsed, compared to the Cobiax slabs that started to show shear cracks that opened much wider, allowing more deflection to occur. More Cobiax and solid samples are to be compared with regards to this ductile behaviour before any final conclusions can be made. It should be borne in mind that this higher ductility in the Cobiax slab specimens is of no real benefit, since the ductile behaviour occurs at a reduced load.

The observed ductility is not characteristic of a shear failure in beams without shear reinforcement and can only be attributed to the presence of the vertical legs of the Cobiax cages acting partially as shear reinforcement. Where the 45º angle crack crosses the path of these vertical bars, the vertical bars tend to hold the concrete on both sides of the crack together for much longer, until these bars are torn out of the concrete or sheared off.

<u>*Remainder of Cobiax slabs*</u>

The load deflection response of the remaining Cobiax slabs with thicknesses of 295mm and 310mm are illustrated in *Figures 3.4.7 and 3.4.8* respectively. The failure mode is similar to that observed for the 280 mm Cobiax slabs. Following the reduction in the peak load, a lower load value is reached, which remains constant for a significant deflection, indicating a greater ductility than observed for the 280 mm solid slabs.

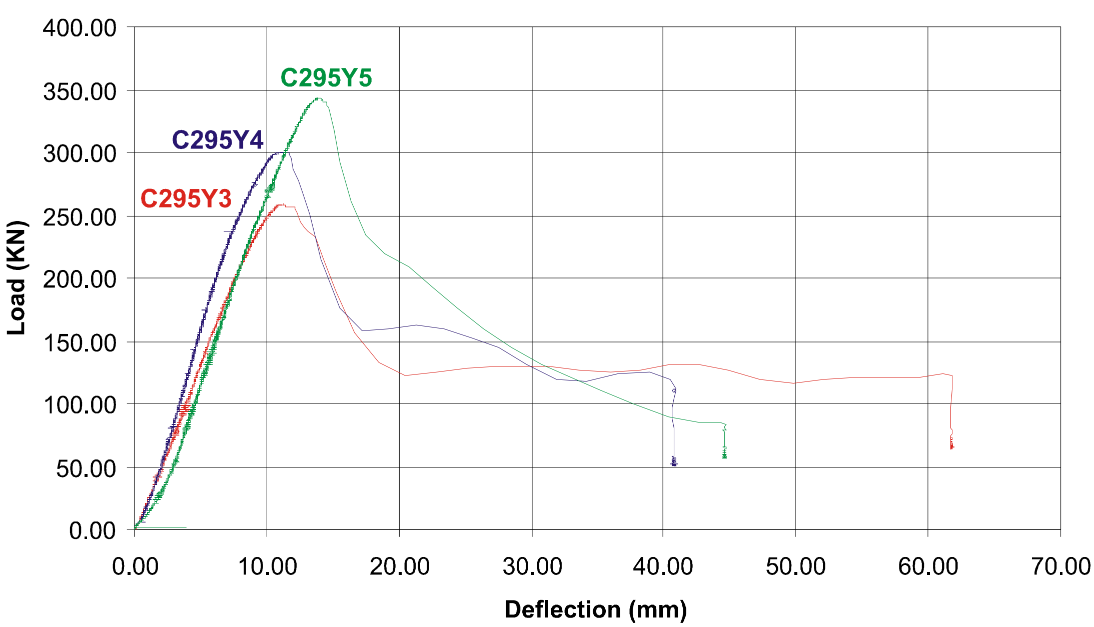

Figure 3.4.7: Load-deflection response of 295 mm Cobiax slabs

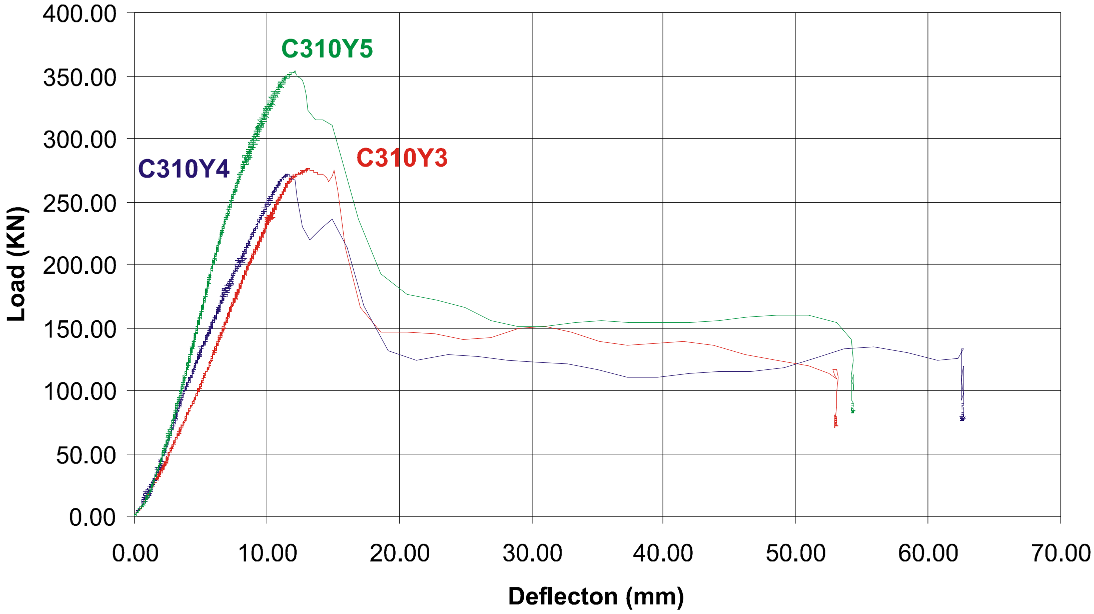

Figure 3.4.8: Load-deflection response of 310 mm Cobiax slabs

3.5. JUSTIFICATION OF RESULTS

The main observations from the results were:

1. The experimental results were significantly higher than predicted using characteristic material strengths.
2. The Cobiax results were higher than the values predicted using the actual material strengths and applying the 0.55 factor to the equivalent solid slab strength.
3. In one scenario the strength of the Cobiax beam even exceeded that of the equivalent solid slab.

Cases 1 and 2 will be discussed and the results justified:

1. The foremost reason for the significant difference between the values calculated before the experiment and the experimental results is that the concrete and reinforcement steel were much stronger than what was designed for. A ready-mix was used and the slump was adjusted due to a misunderstanding. The result was a much higher 13 day strength than was anticipated.

   The steel yield strength was also much higher than anticipated. The preliminary calculations have been done using $f_{cu} = 30MPa$ and $f_y = 450MPa$, but the actual values, as can be seen in *Table 3.5.1* and *Table 3.5.2*, were $f_{cu} = 45.1MPa$ and $f_y = 558.75MPa$. Beam specimens were also tested to establish the tension strength of the concrete.

Table 3.5.1 Concrete test cubes and beam results

Concrete			
Cube No	MPa	Beam No	MPa
A1	45.30	B1	2.23
A2	42.30	B2	3.70
A3	47.70	B3	3.35
Mean	**45.10**	Mean	**3.09**

Table 3.5.2 Steel test results

Steel						
	Size	Yield Stress (MPa)	Tensile Stress (MPa)	Elongation (%)	Area (mm^2)	Length (m)
C1	Y10	565	690	22	76.8	13
C2	Y10	530	645	20	76.2	13
C3	Y10	520	640	21	76.6	13
C4	Y10	620	720	21	77.6	13
Mean		**558.75**	**673.75**	**21**	**76.8**	**13**

The calculations had to be re-done using the actual material strengths and, as shown in *Table 3.5.3,* the failure loads were much closer to the experimental values (See *Figure 3.5.1*). K can be obtained from *Equation 2.11.1*.

Table 3.5.3 Comparison between predicted moment failure loads and shear failure loads based on actual values

SANS10100					
fcu =	45.1	MPa	Cover	20	mm
fy =	558.75	MPa	AY16	201	mm^2
b =	600	mm			
L =	1350	mm	γm	1.0	
K =	0.156		γmc	1.0	
Solid	**Height (mm)**	**d (mm)**	**Pm (kN)**	**Ps (kN)**	**Failure Mode**
280Y3	280	252	242	228	Shear
280Y4	280	252	319	251	Shear
280Y5	280	252	394	270	Shear
295Y3	295	267	257	234	Shear
295Y4	295	267	339	257	Shear
295Y5	295	267	419	277	Shear
310Y3	310	282	272	239	Shear
310Y4	310	282	359	263	Shear
310Y5	310	282	444	283	Shear
Pm =	Failure load for Flexure				
Ps =	Failure load for shear				
Failure mode	**"Moment"**	Beam will fail in flexure			
	"Shear"	Beam will fail in shear			

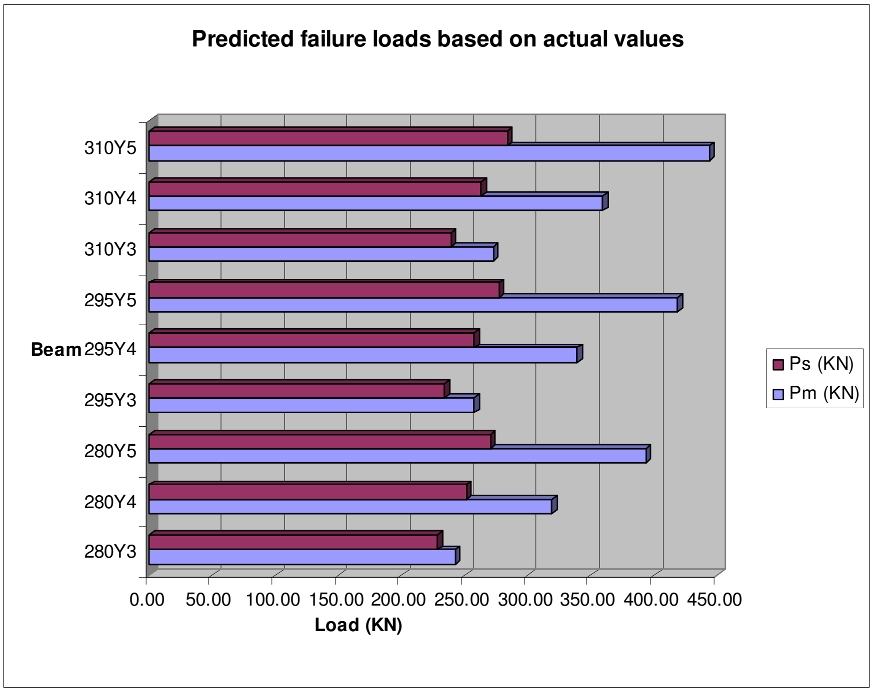

Figure 3.5.1 Predicted moment failure and shear failure loads based on design values

The P_{sc} values in *Tables 3.5.4 & 5* are the predicted failure loads for the Cobiax slabs based on previous research. Both the maximum TUD research factor (0.85) and minimum research factor (0.55) were used in the graphs (Schellenbach-Held and Pfeffer, 1999). Where the actual failure load values in column 2 of the tables exceeded the predicted German shear values, further investigation were required. So far Cobiax slab designers used the minimum shear value with 55% of the shear capacity of that of a solid slab with equal thickness and reinforcement strength and content.

In order to compare the SANS 10100, Eurocode 2 and test results, the results predicted by Eurocode2 was calculated as well, using *Equation 2.5.1*. *Table 3.5.4* and *3.5.5* display the SANS 10100 test results and EC 2 test results respectively.

Table 3.5.4 Comparison between test results and values predicted by SANS 10100

SANS10100					
fcu =	45.1	**MPa**	**Cover**	20	**mm**
fy =	558.75	**MPa**	**AY16**	201	**mm^2**
b =	600	**mm**	**γm**	1.0	
L =	1350	**mm**	**γmc**	1.0	
	Actual Failure load	**Predicted loads (kN)**			
Beam	**Pu (kN)**	**Ps (kN)**		**Psc (kN)**	
			λcob =	**0.85**	**0.55**
S280Y3	242	228	**-**	**-**	**-**
S280Y4	326	251	-	-	-
S280Y5	354	270	-	-	-
C280Y3	268	228	**C280Y3**	186	121
C280Y4	279	251	**C280Y4**	205	133
C280Y5	330	270	**C280y5**	221	143
C295Y3	259	234	**C295Y3**	191	123
C295Y4	301	257	**C295Y4**	210	136
C295Y5	343	277	**C295Y5**	226	146
C310Y3	276	239	**C310Y3**	195	126
C310Y4	271	263	**C310Y4**	215	139
C310Y5	353	283	**C310Y5**	231	150

Pu =	**Experimental failure load**
Ps =	**Failure load for an equivalent solid beam SANS 10100**
Psc =	**Failure load for a Cobiax slab = Factor x Ps**
λcob =	**Cobiax factor for shear capacity reduction**

Table 3.5.5 Comparison between test results and values predicted by EUROCODE 2

EUROCODE 2					
fcu =	45.1	MPa	Cover	20	mm
fy =	558.75	MPa	AY16	201	mm²
b =	600	mm	τRd	0.581	
L =	1350	mm	γm	1.0	
	Actual failure load (KN)	Predicted loads (kN)			
Beam	Pu (kN)	Ps (kN)	Cobiax®	Psc (kN)	
			λcob =	0.85	0.55
S280Y3	242	322	-	-	-
S280Y4	326	335	-	-	-
S280Y5	354	347	-	-	-
C280Y3	268	322	C280Y3	274	177
C280Y4	279	335	C280Y4	284	184
C280Y5	330	347	C280y5	295	191
C295Y3	259	335	C295Y3	285	184
C295Y4	301	348	C295Y4	296	191
C295Y5	343	360	C295Y5	306	198
C310Y3	276	348	C310Y3	296	191
C310Y4	271	360	C310Y4	306	198
C310Y5	353	373	C310Y5	317	205

Pu =	Experimental failure load
Ps =	Failure load for an equivalent solid beam SANS 10100
Psc =	Failure load for a Cobiax slab = Factor x Ps
λcob =	Cobiax factor for shear capacity reduction

where:

$$\tau Rd = 0.035\left(\frac{1}{1.5}\right)^{-2/3} f_{cu}^{\,2/3} \text{ , unitless}$$

From the above tables it is once again clear that SANS 10100 is more conservative in predicting shear failure. This should be noted where the actual shear failure loads of the solid samples are compared to the predicted values for solid samples. *Figure 3.5.2* and *3.5.3* show the comparisons made in *Table 3.5.4* and *3.5.5* respectively, taking λcob equal to 0.85. From *Figure 3.5.3* it can further be noted that EC 2 tends to be more conservative for a higher tension reinforcement content as well. One might argue that EC 2 is not conservative for low reinforcement content.

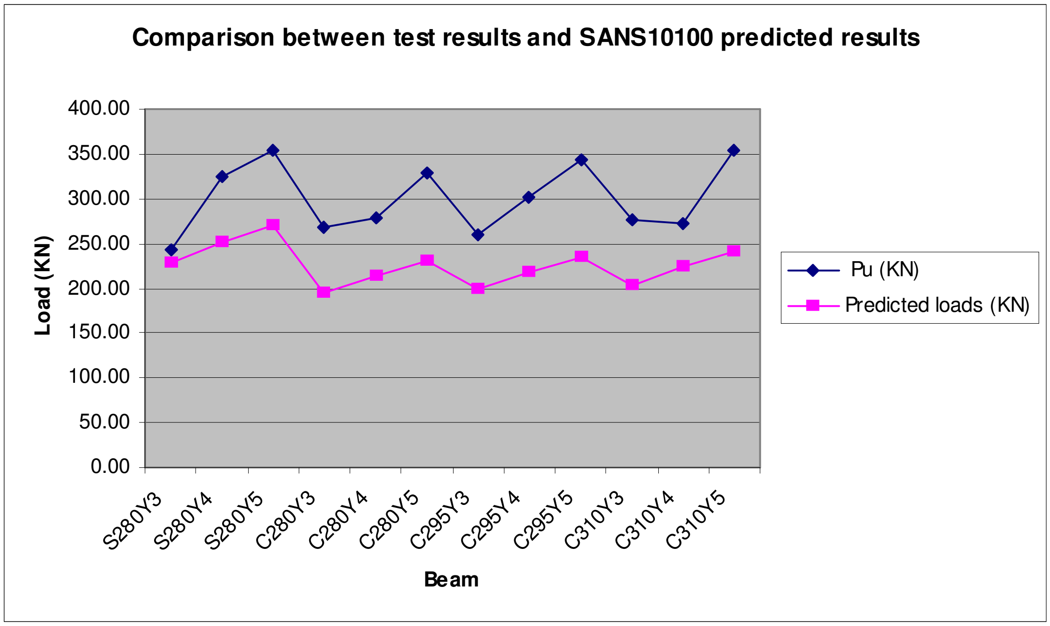

Figure 3.5.2 Comparison between predicted shear failure values and test results (SANS 10100)

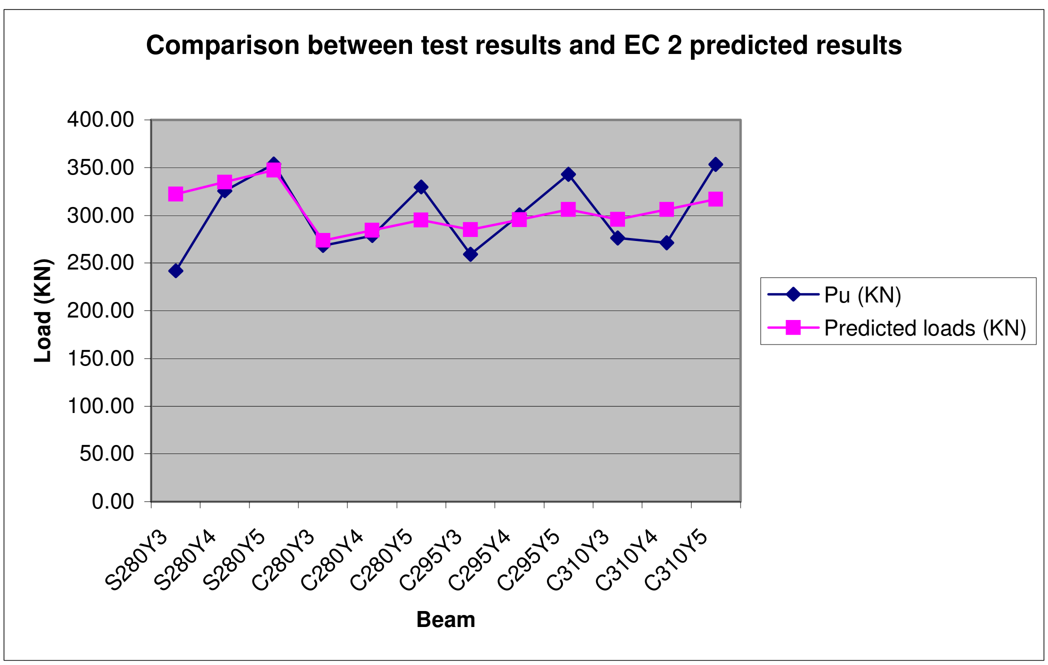

Figure 3.5.3 Comparison between predicted shear failure values and test results (EC 2)

2. Several years ago during the initial Cobiax research the steel cages currently used were not yet implemented. The fact that the Cobiax results are so high implies that the cages are contributing as shear reinforcement, in other words, increasing the shear capacity. It appears that the loss in shear capacity as a result of less aggregate interlock is compensated for by the increased capacity provided by the steel cages.

   Referring back to the experimental breaking loads (P_u) in *Table 3.5.4*, by dividing the failure load of the Cobiax sample by that of the solid sample with the same thickness and reinforcement content, 1.11, 0.86, and 0.93 are the ratios obtained. This amplifies the very essence of the shear research being done here. All three these ratios are much higher than the 0.55 ratio obtained from research in Germany where no steel cages were present in the testing samples. Therefore the steel cages must have some contribution to the shear capacity of a Cobiax slab, that has been discarded up to now.

   To verify the above statements, calculations were done according to SANS 10100 to obtain the shear resistance provided by the cages.

   The cages were fabricated using 5 mm diameter high tensile steel with a nominal yield stress of 450 MPa. The spacing of the cage bars in the vertical plane alternated between 41 mm and 159 mm. An average spacing of 100 mm was used for calculation purposes. The vertical cage bars were welded to the longitudinal bars in the cage (See *Figure 3.5.4*). Semi-spheres with cages cut in half were introduced to the sides of the samples. The longitudinal section shown below shows the true vertical cage dimensions for both the cut-in-half and full cages.

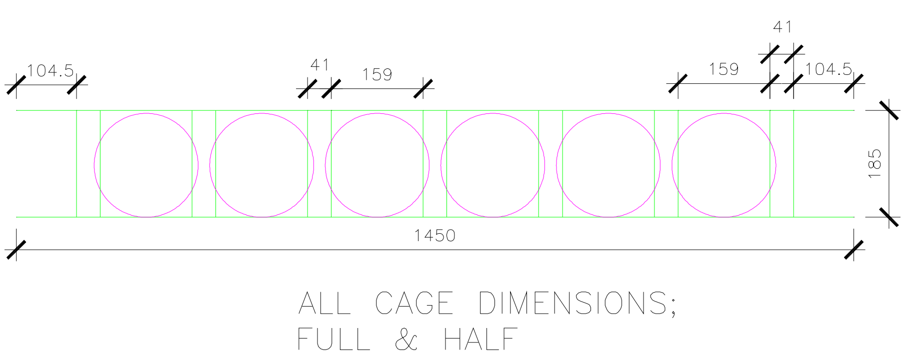

Figure 3.5.4 Cage spacing and dimensions

The maximum spacing permitted by SANS 10100 is 0.75 d = 0.75 x 252 = 189 mm. The maximum spacing of 159 mm spacing is less than this limit. SANS 10100 also requires a minimum amount of shear reinforcement calculated with $\frac{A_{sv}}{s_v} \geq 0.0012b$ where s_v = 100mm on average.

Then:

$A_{sv} = s_v*0.0012*600 = 72 \text{ mm}^2$

The shear reinforcement provided is 6 Y5 bars.

Then:

$$A_{sv} = \frac{6*\pi*5^2}{4} = 117.8 \text{ mm}^2 > 72 \text{ mm}^2$$

The shear reinforcement provided is more than what is required, therefore the only requirement not met is that the shear reinforcement must be anchored around the tension reinforcement. Yet, one can reason that some degree of anchorage is obtained via the welds of the vertical cage bars to the horizontal cage bars in the tension zone, and the horizontal bars will obtain a small degree of anchorage in this zone, which will drastically reduce when the weld fails under large loads.

Should one try to accommodate the shear resistance of these vertical cage bars, an approach could have been to subtract the shear resistance provided by the cages from the experimental results to obtain the capacity provided by the voided concrete. However, the resulting capacity will become unrealistically low when compared to earlier research. It is therefore concluded that the cages increase the shear capacity but not to the full possible value that could have been obtained by properly anchored shear links. This comment is confirmed when studying the load-deflection results that show a failure pattern tending more towards that of a brittle failure, than a ductile failure that would be expected in the presence of fully anchored shear reinforcement.

The following conclusions can be made in terms of the cages' influence:

- The cages provide additional longitudinal reinforcement which will increase the shear capacity v_c. This was conservatively ignored in preceding calculations, since it is usually very poorly anchored, taken that the spot welds, connecting the vertical cage bars to the horizontal cage bars in the tension zone of the concrete, can easily fail.

- The presence of the vertical transverse bars in the cages add to the shear capacity v_s. They have met the requirement for maximum spacing and minimum reinforcement but were not anchored around the main reinforcing bars. Because of this, the vertical bars will add capacity to the aggregate interlock, but not as much to the dowel action. Therefore, the full value v_s predicted by the design code cannot be used.

It appears from this research that the 0.55 factor currently used may be too conservative. Comparing experimental results of the 280 mm slabs, this factor appears to be closer to 0.85. If this factor is applied to the design capacity obtained for an equivalent rectangular slab, the design should be sufficiently safe as illustrated in *Figure 3.5.5*. For these results, the smallest factor of safety will be 1.77.

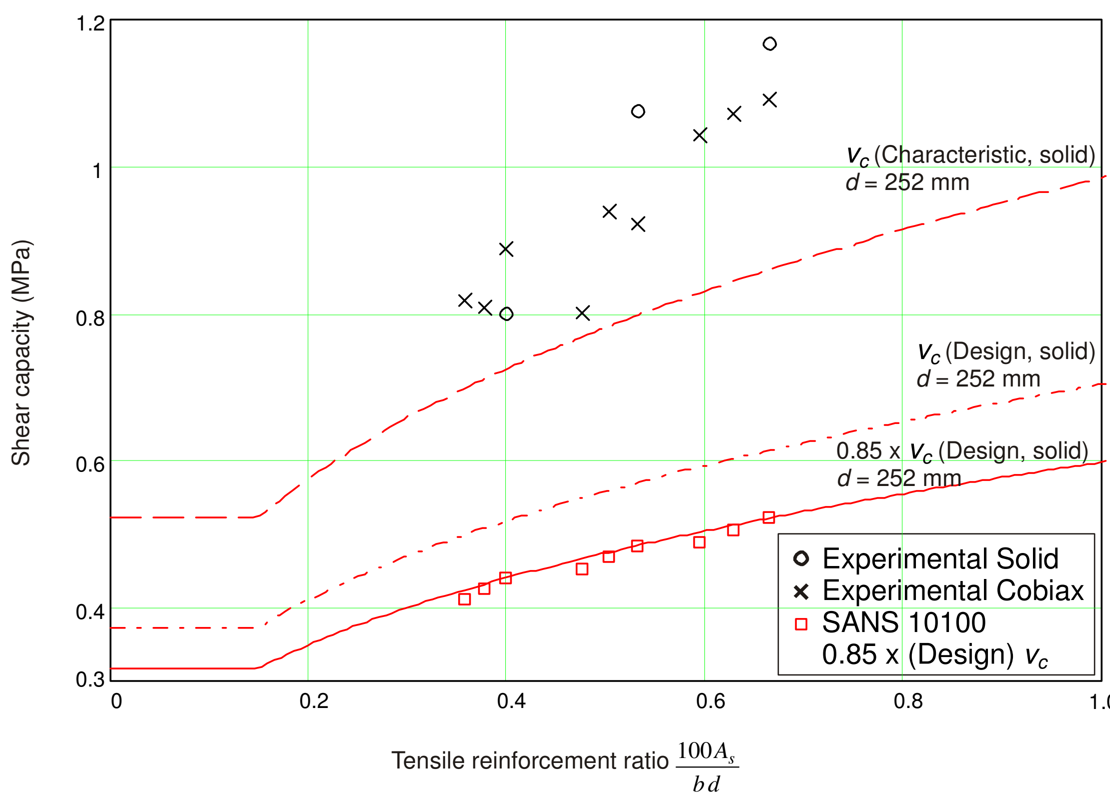

Figure 3.5.5 Design shear capacity of Cobiax slabs

Table 3.5.6 illustrates the shear resistance that fully anchored cages would have provided. *Equation 2.4.4* was used.

Table 3.5.6 Shear resistance of cages

SANS10100					
fcu =	45.1	MPa	Cover	20	mm
fy =	558.75	MPa	AY16 =	201	mm²
fyv =	450	MPa	AY5 =	19.63	mm²
b =	600	mm	γm	1.0	
L =	1350	mm	γmc	1.0	
sv =	100	mm	K =	0.156	

					Cage Resistance			
Solid	Height (mm)	d (mm)	Y16's	Asv (mm²)	Y5's	Asv (mm²)	Vs (KN)	Ps (KN)
280Y3	280	252	3	603	6	118	133.6	267
280Y4	280	252	4	804	6	118	133.6	267
280Y5	280	252	5	1005	6	118	133.6	267
295Y3	295	267	3	603	6	118	141.5	283
295Y4	295	267	4	804	6	118	141.5	283
295Y5	295	267	5	1005	6	118	141.5	283
310Y3	310	282	3	603	6	118	149.5	299
310Y4	310	282	4	804	6	118	149.5	299
310Y5	310	282	5	1005	6	118	149.5	299

Vs =	Shear resistance provided by cages
Ps =	Shear load resistance provided by cages = 2Vs

Comparing *Table 3.5.4* with *Table 3.5.6* it is clear that the shear resistance added to a solid slab with Cobiax cages inside should have more than doubled up the capacity of the sample strength. This can be visualised by adding the *Ps* value from *Table 3.5.4* to that of *Table 3.5.6*. The theoretical vertical point load at the centre of the beam (*Ps*) has been obtained by doubling the theoretical shear reinforcement capacity (*Vs*). This will approximately be true for a simply supported beam with a point load in the centre, where only vertical shear reinforcement has the ability to resist shear (off course this is not the case in reality, but *Ps* is nevertheless required for calculations to follow).

The question arises what the capacity would have been of Cobiax samples without cages, plus the *Ps* value in *Table 3.5.6*? Should the value be higher than the *Pu* value in *Table 3.5.4*, it would be a clear indication that some of the shear capacity of the vertical cage bars does not contribute to the shear strength, and the best reason being that these bars are not fully anchored around the tension reinforcement. At the TUD they only considered aggregate interlock, with the absence of some aggregate along a 45° angle through the Cobiax slab, to contribute to shear capacity (Schellenbach-Held and Pfeffer, 1999). This area of aggregate interlock was established as follows:

There are two full and two half spheres in a cross section as shown by *Figure 3.2.1*. This means a total area of three spheres. In the cross section, the sphere is a circle with a maximum diameter of 180mm.

$$A_{circle} = \pi r^2$$

where:

r = radius of circle

The effective area that provides aggregate interlock in a Cobiax slab is:

$$A_{eff} = bd - 3A_{circle}$$

This is for a cross section that is perpendicular to the plan view of the beam. To compensate for the extra area that will be available because of a 30 or 45° crack, a further factor has to be introduced. To be conservative, a 45° angle is assumed which will produce the smallest increase in area, therefore:

$$A_{eff} = \lambda_{area} bd - 3A_{circle}$$

with:

$$A_{circle} = \pi r^2$$

$$\lambda_{area} = \frac{1}{\sin 45^\circ} = 1.41 = \text{slope area increasing factor}$$

where:

$$r = 90mm$$

The effective shear resistance is then:

$$V_{ceff} = V_c Eff.Ratio$$

where:

$$Eff.Ratio = \frac{A_{eff}}{bd}$$

The force required to cause a V_{ceff} shear value will yield values similar to those found in *Table 3.5.4* under the $0.55P_{sc}$ column. This is simply because the effective ratio derived above will be in the vicinity of 0.55 for a worst case scenario. The TUD researchers therefore ignored the compression block and dowel-action, and only concentrated on the loss of aggregate interlock along the 45º plane of a typical shear crack (Schellenbach-Held and Pfeffer, 1999).

In *Table 3.5.7* the contribution of fully anchored vertical cage bars (P_s), the theoretical force required to break a Cobiax slab where only aggregate interlock contributes to shear resistance ($0.55P_{sc}$), and the two forces added together (Pt) are displayed. These P_t forces should have been equal to that of the actual breaking loads (P_u) of the various samples, should the vertical cage bars at all have been fully anchored around the tension reinforcement. Since the P_t values are greater than the P_u values, it shows that the vertical bars are not fully anchored.

A rough estimate of how effective the vertical cage bars are, can be obtained by the following calculation:

$(P_u – 0.55P_{sc})/P_s$

According to this calculation the vertical bars are roughly between 44% to 70% effective in shear. This conclusion should be approached with great caution, since theoretical and test results were mixed, as well as the contribution of other shear resistance parameters has been ignored, like dowel-action.

The better way to test the effectiveness of these vertical bars will be to break several solid samples with and without the cages placed inside, with no spheres present whatsoever. The contribution to shear capacity of the cages will then be clearly demonstrated from the empirical test results.

Table 3.5.7 Rough indication of the cages` shear capacity

Cobiax	Ps (kN)	0.55Psc (kN)	Pt (kN)	Pu (kN)	(Pu – 0.55Psc)/Ps
280Y3	267	121	388	268	0.553
280Y4	267	133	400	279	0.548
280Y5	267	143	410	330	0.700
295Y3	283	123	406	259	0.479
295Y4	283	136	419	301	0.582
295Y5	283	146	429	343	0.695
310Y3	299	126	425	276	0.502
310Y4	299	139	438	271	0.442

3.6. CONCLUSION

The main conclusion of this chapter is that the shear reduction factor for Cobiax flat slabs can be increased from 0.55, to at least 0.86, in accordance with the test results discussed. This increase in the shear reduction factor is accepted to be the result of the presence of the Cobiax steel cages (previously omitted at the TUD) in the test samples. Although it has been shown that the steel cages' vertical bars do not contribute as much to the shear strength as fully anchored shear reinforcement, the cages indeed increased the shear capacity of the Cobiax slabs.

Firstly the conclusion is of importance to demonstrate that the 0.55 shear reduction factor can conservatively be applied when designing Cobiax slabs in accordance with SANS 10100. Secondly this opens up the opportunity to utilise higher shear reduction factors, that might benefit the feasibility of Cobiax slabs. This second statement will require further investigation before it can be accepted and implemented into the design of Cobiax slabs.

Interesting to note from this chapter is that the EC 2 calculation for the shear resistance of slabs without shear reinforcement is less conservative than that of SANS 10100. When comparing the theoretical design code results with the laboratory test results, EC 2 tends to provide the designer with slightly more accurate results though.

The feasibility study of Cobiax flat slabs, discussed in *Chapter 4*, could be conducted with ease of mind that the utilisation of the 0.55 shear reduction factor would not compromise the integrity of a Cobiax slab design in accordance with SANS 10100.

4. ECONOMY OF INTERNAL SPHERICAL VOID FORMING CONCRETE FLAT SLAB SYSTEMS

4.1. BACKGROUND

Finding a practical method to compare costs of different slab systems is complex in the sense that the layout and application of most structures vary significantly, leaving the designer with almost endless possibilities. Many different techniques have been tried in the past, most of them with valid application in practice (refer to the work done by Goodchild (1997) described in *Chapter 2.14*). This report will focus on the most practical "real-life" design approaches, complementing the methodology that most South Africa design engineers will follow to achieve an economical design. Many assumptions will nevertheless be made to generalise the process of comparing slabs.

Two slab systems identified to be compared with a spherical void forming concrete flat-slab system (SVFS) are coffer slabs and unbonded post-tensioned slabs. Cost results for the SVFS will be based on the only existing such system in South Africa. All three slab systems have already been discussed in *Chapter 2*. The reason for their comparison with the relatively new SVFS is because they serve the same function and are well known as cost effective systems for large span slabs in South Africa. The material for the construction of these three large span slab systems is readily available in the country as well.

Same as for many other cost comparative studies on slab systems, these slabs were all modeled as shown in *Figure 4.1*. These three by three equal continuous spans provide the researcher with a relatively conservative, yet practical system, displaying both the behaviour of an internal span and external spans. Other motivation for this layout is that expansion joints will occur at distances less than 40 m apart as a good design practice to minimise crack widths. Large span systems with three continuous spans will quickly approach this 40m bench-mark, as span lengths increase.

The finite element layouts consisted of the following span lengths, based on the highest minimum and lowest maximum value generally used in practice for the three types of slab systems:

- 7.5 m
- 9.0 m
- 10.0 m
- 11.0 m
- 12.0 m

The above span lengths were all combined with three sets of load combinations each, for all three types of slab systems, derived from suggestions made by SABS 0160-1989:

1. Live Load (LL) = 2.0 kPa and Additional Dead Load (ADL) = 0.5 kPa
2. LL = 2.5 kPa and ADL = 2.5 kPa
3. LL = 5.0 kPa and ADL = 5.0 kPa

Self weight (SW) was applicable to all designs. Combination 1 was referred to as "Light Loading", combination 2 as "Medium Loading" and combination 3 as "Heavy Loading" throughout this report. Combination 1 would generally resemble the loading found on normal parking slabs, combination 2 that of normal office loading, and combination 3 that of retail buildings or office areas with single skin brick walls as internal partitions, combined with the storing of heavy equipment. Live load mainly refers to people and loose equipment on floor areas that can be moved around. Additional or superimposed dead load mainly refers to finishes, services and partitions.

4.2. MAIN ASSUMPTIONS

The following summary of assumptions for this cost study was based on common building types, design methodology, and available materials:

Cost and structural features

All designs were done using SANS 10100-01:2000 design requirements and fulfilled the requirements of minimum reinforcement, deflection and punching shear resistance. The total cost described the direct cost only, which included material, formwork, labour, site delivery, and contractor's mark-ups, but excluding VAT.

The formwork cost has been simplified by assuming normal 3 m high storeys and the construction of large floor areas where repetition was possible. No column- or drop-heads were allowed for below any of the slab systems for all models analysed, making formwork application easier and cheaper.

Column dimensions of 450 mm x 450 mm were assumed for all columns of every model and slab system analysed. All columns were assumed to be pinned to the slab soffit. This resulted in a slightly more conservative slab design, since no moments (accept minimum moments due to eccentricity) were carried by the columns. Buildings with four storeys or less were assumed, since this will result in very small differences in column and foundation costs for the different slab systems analysed.

The models such as displayed in *Figure 4.1* were all completely surrounded by expansion joints, allowing the slab to stop near the centre of the edge columns. All span lengths were measured from centre to the centre of columns.

Material properties

The concrete cube strength for all models was taken to be 30 MPa. The elasticity of concrete was taken equal to 26 GPa. Reinforcement yield strength was set equal to 450 MPa.

Loading

Dead Load (DL) consisted of SW and ADL. Only one load combination was considered for ultimate limit state (ULS), namely 1.2DL + 1.6LL. Serviceability limit state (SLS) had factors 1.1DL + 0.6LL. The 0.6 factor was used due to the fact that 60% of the live load was taken as permanent loading when estimating long-term deflections. This 60% is a good estimate, supported by SABS 0160 design code. All 45 models were loaded with these ULS and SLS load combinations, and to simplify the cost comparisons, no pattern loading was introduced to any of the models.

Deflection

In accordance with SANS 10100 the maximum long-term deflection allowed for concrete structures is span/250. In accordance with SABS 0160 the maximum deviation for any slab or beam may not exceed 30 mm or span/300, whichever is the lesser, where this deviation can be measured to the top or bottom of the slab's horizontal position of zero deviation. These requirements were fulfilled by insuring that no long-term deflection exceeded span/250 or 60 mm, whichever is the lesser, where the 60 mm had been obtained from a maximum deviation (precamber) to the top of 30 mm, plus the maximum allowed downward deviation of 30 mm.

It should be clear that the final downward deviation described in SABS 0160 refers to differential deflection. The "span" can therefore refer to the distance between any two points, with the resulting maximum difference in vertical displacement along a line between these two points. The points of zero deflection are the columns. The line between two points can therefore conveniently be taken on a diagonal line or orthogonal line between two columns. The deflection on an orthogonal line runs along a column band, usually with a small differential deflection. The diagonal line will generally contain the maximum differential deflection, and when it is divided by its maximum deflection, it normally yields the smaller value, closer to the 250 limit, therefore being the critical case to consider. The Cobiax company interpreted the DIN 1045-1 code in such a manner that they decided to base their span/deflection criteria on the diagonal span between columns (CBD-MS&CRO, 2006).

The L/d ratios described in SANS 10100 should not be confused with the other criterion of span/250. The "L" in the L/d ratio criterion refers to the critical span, which is usually the longer of the two orthogonal spans of a flat slab panel. This is a different application than that found in SABS 0160. Neither codes discuss the deflection limits very clearly, and experience shows that various engineers have different interpretations of deflection limits.

Long-term deflections were not calculated according to the formulae of code requirements, but rather the general rule of thumb were applied by multiplying the short-term elastic deflections with a factor. Experience shows that this factor usually varies between 2.5 and 4.0 according to most design engineers, and will depend on the type of aggregate, the curing of the concrete, temperature exposure, loading of the slab, and on so forth. These elements will in turn result in the creep and shrinkage of the concrete, causing long-term deflections to occur. A factor of 3.5 was assumed for all slab types in this report.

It should be noted that the aim of this report is not to investigate long-term deflection behaviour of different slab types, and therefore the factor is used. Interesting enough, from analysis run by Prokon software for post-tensioned slab design, the output of this software indicated a long-term deflection factor between 3.0 and 3.5 to be quite applicable to all span and load ranges of post-tensioned slabs. Although no special verification of Prokon software was attained for long-term deflection results in prestressed beams, the software had been utilised by the majority of structural engineers throughout South-Africa for more than a decade. The fact that constructed prestressed beams and slabs that had been designed using Prokon did not yield any problems that the public was made aware of, justifies at least that the deflection predictions of Prokon were either correct or conservative.

Table 4.1 displays deflection results for the three slab types compared in this chapter. These deflection results were obtained from various Strand7 finite element analysis output contours. The finite element analysis methodology will be discussed later in this chapter.

As mentioned, the span from any column to column, divided by the maximum deflection along that line, may not result in a value lower than 250. The maximum deflection along a diagonal line between two columns in *Figure 4.1* will be larger than that of the shorter span length in an x or y direction (referred to as "Span" in *Table 4.1*). After investigating the span/deflection (span/x or diagonal/x) ratios, the worst case scenarios had been listed in *Table 4.1*. These ratios were always critical (smallest) along the diagonal span for coffer and Cobiax slabs, yet both scenarios had to be listed for post-tensioned slabs.

Table 4.1: Deflections

Load	Span (mm)	Cobiax Deflection (diagonal) (mm)			Coffer Deflection (diagonal) (mm)		
		Elastic	Long-term	Diagonal/x	Elastic	Long-term	Diagonal/x
7.5m light load	7500	7.6	27	399	5.3	19	572
7.5m medium load	7500	10.2	36	297	6.8	24	446
7.5m heavy load	7500	7.7	27	394	9.4	33	322
9m light load	9000	13.5	47	269	10.8	38	337
9m medium load	9000	13.3	47	273	14.0	49	260
9m heavy load	9000	9.3	33	391	11.4	40	319
10m light load	10000	15.6	55	259	16.4	57	246
10m medium load	10000	13.9	49	291	12.8	45	316
10m heavy load	10000	11.0	39	367	11.3	40	358
11m light load	11000	16.1	56	276	15.0	53	296
11m medium load	11000	14.8	52	300	12.6	44	353
11m heavy load	11000	11.8	41	377	16.4	57	271
12m light load	12000	16.9	59	287	14.6	51	332
12m medium load	12000	15.9	56	305	17.8	62	273
12m heavy load	12000	13.8	48	353	23.1	81	210
Load	**Span (mm)**	**Post-tension Deflection (diagonal) (mm)**			**Post-tension Deflection (normal) (mm)**		
		Elastic	Long-term	Diagonal/x	Elastic	Long-term	Span/x
7.5m light load	7500	7.7	27	394	6.0	21	357
7.5m medium load	7500	9.2	32	328	7.2	25	298
7.5m heavy load	7500	11.3	40	268	8.7	30	246
9m light load	9000	10.3	36	353	8.0	28	321
9m medium load	9000	11.9	42	306	9.4	33	274
9m heavy load	9000	13.5	47	269	10.5	37	245
10m light load	10000	11.4	40	354	8.9	31	321
10m medium load	10000	12.9	45	313	10.0	35	286
10m heavy load	10000	14.1	49	287	11.0	39	260
11m light load	11000	12.7	44	350	9.9	35	317
11m medium load	11000	13.9	49	320	11.1	39	283
11m heavy load	11000	15.9	56	280	12.4	43	253
12m light load	12000	14.9	52	325	11.9	42	288
12m medium load	12000	15.7	55	309	12.6	44	272
12m heavy load	12000	12.7	44	382	10.2	36	336

Reinforcement

Cover on reinforcement was taken to be 25 mm for all slabs, satisfying safe fire protection requirements of over 2 hours fire exposure. In contrast with the assumptions of Goodchild (1997) for his analysis discussed in *Chapter 2.14,* reinforcement content for all models was based on the reinforcement provided and not those required. The reinforcement provided was always kept to a minimum, but never allowed to be less than the SANS 10100 minimum reinforcement specifications. Curtailment and lap lengths (SABS 0144, 1995) were provided for by multiplying the total reinforcement per m^2 of slab area by 1.1, therefore allowing for 10% extra reinforcement. In practice this 10% would normally represent the correct amount of reinforcement very well.

A good designer will try to design the reinforcement as such that the reinforcement provided is always more than the reinforcement required, yet kept to a minimum. A better simulation of the reality can be obtained for the use of a cost analysis, by using this amount of reinforcement provided, rather than the exact amount required. The reinforcement content chosen for each slab was therefore approximately 5 percent more than the amount required. It should be borne in mind though that it is not practically possible to read off the exact amount of reinforcement required when interpreting a finite element contour plot.

Spacings of reinforcement provided were also kept to standard spacings such as 125 mm or 300 mm increments for example. In areas where top and bottom reinforcement occurs, the spacings were set to have the same increment to simplify construction.

The three tables in *Appendix A* show the reinforcement areas as provided for all the models. Using the 7.5m span scenario as an example, typical finite element output displays of the models' required reinforcement content are shown in *Appendix B, C and D.*. These were obtained ustilising Strand7 (2006) software, with the plate elements set up in accordance to SANS 10100 criteria for the direct calculation of reinforcement using Wood-Armer moments.

The 7.5 m span Cobiax slab with light loading was used to demonstrate the accuracy of the reinforcement contours in *Appendix B*. This was done by comparing the top and bottom reinforcement in the y-direction with a MathCad generated contour plot (by Dr John Robberts, 2007) based on the gauss point values obtained from Strand7. The plots from the Strand7 concrete module are very similar to those generated by Dr John Robberts's program, and therefore one can assume the reinforcement results to be quite accurate.

The Strand7 finite elements used for al three slab systems consisted of rectangular eight-noded plate elements. Each plate element represented a finite volume of the concrete slab. Applicable concrete properties and plate thicknesses were applied to all plate elements for the various scenarios.

The "B" and "T" in *Appendix B-D* mean "Bottom" and "Top" reinforcement respectively. Only the slightly more conservative *y*-direction of reinforcement is displayed, since the internal lever arms between the compression block and tension reinforcement were smaller in this direction for all the models. The same amount of steel provided for this y-direction was provided for the x-direction.

The steel provided as displayed in *Appendix A* was based on the following assumptions:

- Bottom reinforcement for column strips was taken to be a mm^2/m value read at a position measured in the *x*-direction, one sixteenth of the span away from the y-direction line connecting column centers. This was done for both edge and internal spans, and bottom reinforcement was provided according to these values. The maximum reinforcement contours at the above positions (usually closer to midspan) were used.
- The same has been done for the middle strips, but the steel content was read at a position five sixteenths away from the y-direction line connecting column centers.
- The bottom steel was taken to be continuous over the whole slab in both directions.
- At the same distances away from the y-direction column line as for bottom steel, but measured right on top of the x-direction column line connecting internal columns, the amount of top steel could be found for column and middle strips.
- Top steel were stopped at a distance of 0.3 times the span length past a line connecting the column face, accept for coffer slabs, where minimum reinforcement was required according to SANS 10100 throughout the 100 mm topping. Cobiax in Europe claims that no minimum reinforcement is required in the midspan (compression) region of a Cobiax slab (CBD-MS&CRO, 2006) due to the fact that the top flange thickness rapidly increases to the full slab depth between voids, being thin only for a small area above each sphere.
- The reinforcement spacings of column and middle strips were allowed to have different spacing increments, since there is no practical reason why these spacings should be the same, as long as the top and bottom steel had the same increments.
- No reinforcement spacing was taken smaller than 100 mm or larger than 300 mm centre to centre.

- Coffer slab tension reinforcement had to be grouped in the webs. The steel provided was based on two or four bars with a specific diameter in the bottom of each web, matching the required steel displayed in *Appendix C*.

4.3. FORMWORK

Appendix E shows the formwork cost analysis done by Jan Kotze (2007) at Wiehahn Formwork (Pty) Ltd for both Cobiax flat-slabs and coffer slabs. All formwork material, delivery on site and labour were included in this analysis, but VAT excluded. The analysis was based on large slab areas where repetition of formwork usage resulted in 5 day cycle periods for both flat-slab (Cobiax and post-tensioned slabs) and coffer formwork. The assumption is based on the presence of an experienced contractor on site and no delays in the supply of the formwork.

A 450 mm thick Cobiax flat-slab with 315 mm diameter spheres was compared with a 525 mm thick coffer slab with 425x425x900 coffers and a 100 mm topping. This comparison resembles the average formwork conditions for the three slab systems, assuming that since formwork designs are conservative, the formwork costs will vary only slightly for different slab and coffer depths. The 425 mm deep coffer mould is also known as the most commonly used and available coffer in South Africa.

In *Appendix E* the total nett rate for the post-tension and Cobiax flat-slab formwork will be R64/m^2. The total net rate for a coffer slab will be R114/m^2. These rates are displayed in *Table 4.6*. Therefore coffer formwork will be approximately R50/m^2 more expensive than flat-slab formwork for large slab areas. For small projects this difference will increase due to the fact that the first cycle or two for coffers takes longer, resulting in an extended hire period.

4.4. COBIAX SLABS

<u>Punching Shear</u>

Eight-noded rectangular plate finite element models were created for all three load combinations and five span lengths, resulting in 15 models for the Cobiax slabs alone. *Figure 4.1* displays square areas around the columns which result in approximately 25% of the total slab area. These areas will remain solid to accommodate shear greater than 55% of that of a solid slab's v_c-value, with the same thickness as the specific Cobiax slab under investigation (see *Chapter 3* for a discussion).

The Cobiax solid zones obtained from a Strand7 analysis are shown in *Appendix F*, where the white areas around the columns are to be left solid (i.e. spheres omitted), and the remaining area should be supplied with the applicable Cobiax spheres (where the applied shear is lower than $0.55v_c$). Only one such analysis is shown in *Appendix F*, since all Cobiax models for the different scenarios resulted in almost exactly similar shear contour patterns. Comparing *Figure 4.1* with the Cobiax plot in *Appendix F*, it is clear that the size of the square solid zones assumed in *Figure 4.1* simulate the real solid zone scenario quite accurately, and therefore the Cobiax models in this report can be used with confidence.

The slab thicknesses of Cobiax slabs were mainly determined by using the punching shear design software of Prokon, set up to fulfill SANS 10100 requirements. The vertical column reaction resulting from the ULS loading combination was obtained for an internal column, using Strand7 software. A simplified punching shear design was then performed by entering this vertical load and other material factors into the Prokon punching shear software.

Chapter 3 indicated that SANS 10100-01 is more conservative than EC 2 for punching shear requirements, and compared to the test results maybe a bit too conservative. The ultimate shear (v) may not exceed $2v_c$. Enough tension reinforcement had to be added over the column zone to cross the critical shear perimeters, to prevent the utilisation of uneconomically thick slabs. The more tension reinforcement, the higher the value of v_c. The Prokon punching shear calculation output for an internal column is displayed in *Appendix G*, using the 7.5m span scenario as an example for the Cobiax models.

The area of punching reinforcement could be found from *Appendix G* type output, and then multiplied by the length of half a shear clip for the specific slab thickness, to calculate the volume of punching reinforcement for one column. This volume could in turn be multiplied by the 7850 kg/m^3 to obtain the steel weight in kg. The weight could then be multiplied by the total number of columns, taking into account that the eight edge columns are “half” columns and four corner columns are “quarter” columns. This means that only half a shear zone exists for edge columns and only quarter a shear zone exists for corner columns.

Lastly this total steel weight for the punching reinforcement could be divided by the total slab area for the specific model (see *Figure 4.1*), resulting in a very low steel content per m^2, usually being far less than 1.0 kg/m^2 for most of the models. Therefore one can conclude that punching reinforcement will only contribute to a very small percentage of the total reinforcement content. Nevertheless this approximated punching reinforcement was added to the reinforcement content displayed later in this report in *Tables 4.7 to 4.9*.

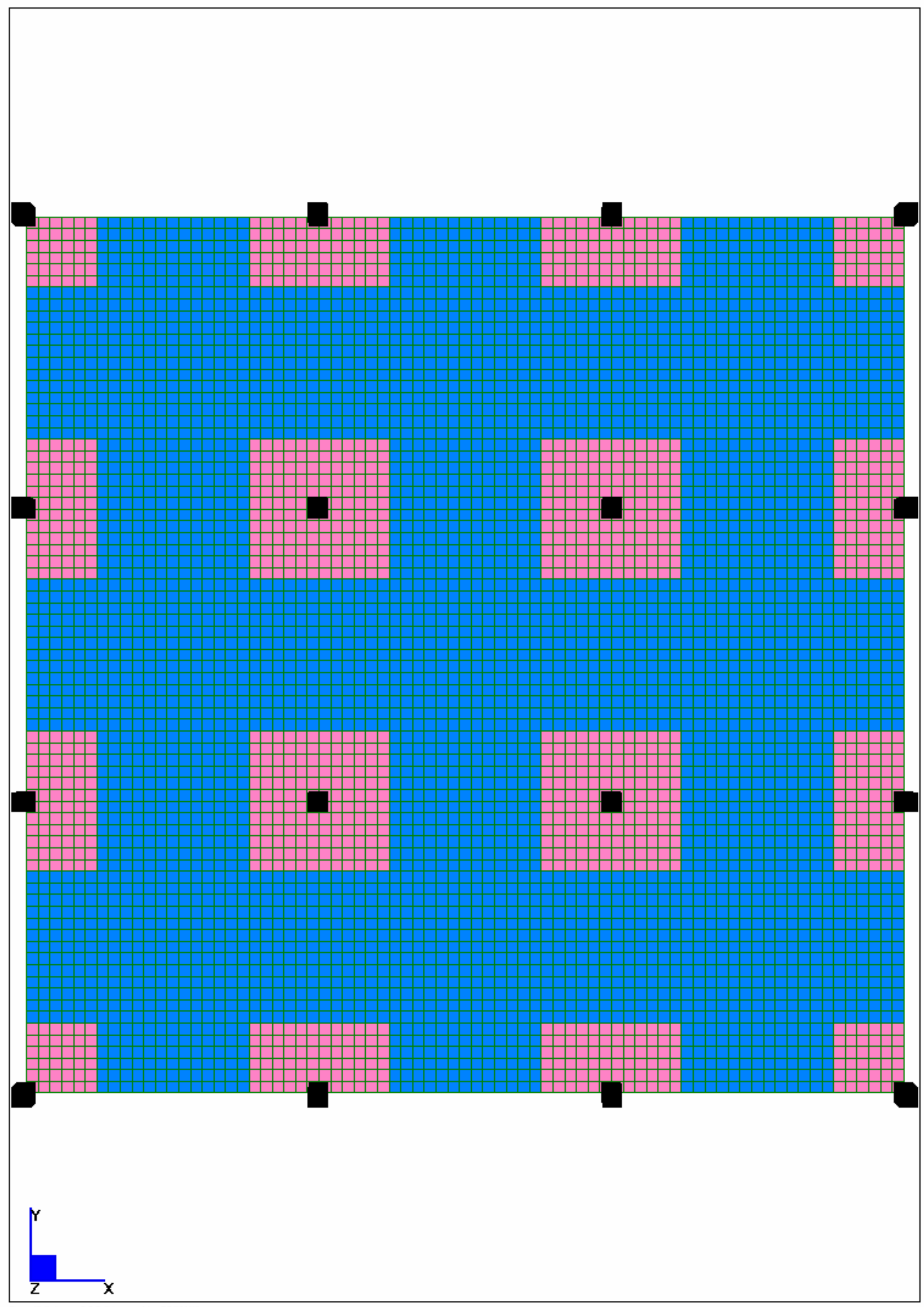

Figure 4.1: Cobiax and Coffer slab solid zone layouts

Deflection

A stiffness reduction factor had to be calculated for all types of Cobiax slabs. The formula for elastic deflection calculation is:

$$deflection = \frac{kwL^4}{EI}$$

Where:

k = a factor depending on the support conditions of the specific span
w = SLS load
L = span length
E = elasticity of concrete
I = second moment of area, in other words the stiffness of the slab

Whether a stiffness reduction factor is applied to either the E or the I value in the formula above will make no difference. The E-value of the pink areas (voided zones) of the Cobiax models (see *Figure 4.1*) were simply reduced by the stiffness reduction factors in *Table 4.2* for the applicable slab thickness. By also adding an upward load over these voided zones for reduction in dead load, obtained from *Table 4.2*, one could obtain the correct elastic deflection values for any Cobiax slab.

The reduction in dead load was simply the displaced concrete weight (25 kN/m^3) as a result of the hollow spheres in the voided areas, which differs for all different sizes of spheres. The calculation of the stiffness reduction factors are more complicated though.

Figure 4.2 shows a section through a Cobiax slab on the left hand side, displaying only two spheres, cut exactly where the diameter is greatest. This section will be exactly the same for the perpendicular direction. Should half a sphere be taken to perform calculations with, an x-distance can be calculated to the centroid of the hemisphere, where x = 3r/8, with al symbols explained in *Table 4.2*. With the formula for a circle (Pythagoras) $r^2 = x^2 + y^2$ one can easily obtain the y-value.

Section A-A in *Figure 4.2* was taken at the x-position, displaying a new cross section on the right hand side of the figure. This cross-section is representative of the voided part of the Cobiax slab when calculating the second moment of area. In *Table 4.2* I_s is calculated with the formula $I = bh^3/12$ and represents the second moment of area of a flat slab with no Cobiax void. $I_c = \pi r^4/4$ represents the second moment of area of a circle with radius y. I_c can then be subtracted from I_s and then divided by I_s to provide a ratio of the stiffness of a voided slab to that of a solid slab.

Along an imaginary line through the cetroids of the spheres in a Cobiax slab, 90% of that line will be inside the spheres (voids) and 10% of that line will run through solid zones (regions between spheres). Due to the gradual change in void size and thus cross-section of the slab along the line, one may assume that the stiffness of a Cobiax slab will be given by combining the voided zone's (90%) and the solid zone's (10%) stiffnesses to obtain an average stiffness. Stiffness reduction factors follow in *Table 4.2*, which complement those obtained at the Technical University of Darmstadt (TUD) very well, where both empirical tests, as well as theoretical calculations were performed.

It should be noted that one can simply adjust the slab thickness in *Table 4.2* to obtain a new stiffness ratio, but that one cannot use this excel program to calculate the stiffness for different vertical positions of the spheres within the slab thickness. For the purposes of this report it was assumed that the spheres were all placed mid-height in the slab.

Multiplying the E-value of 26 GPa with this stiffness reduction factor as explained earlier, will then provide the designer with a new E-value (see *Table 4.2*) for the purpose of deflection calculations with either hand calculation methods or finite element software.

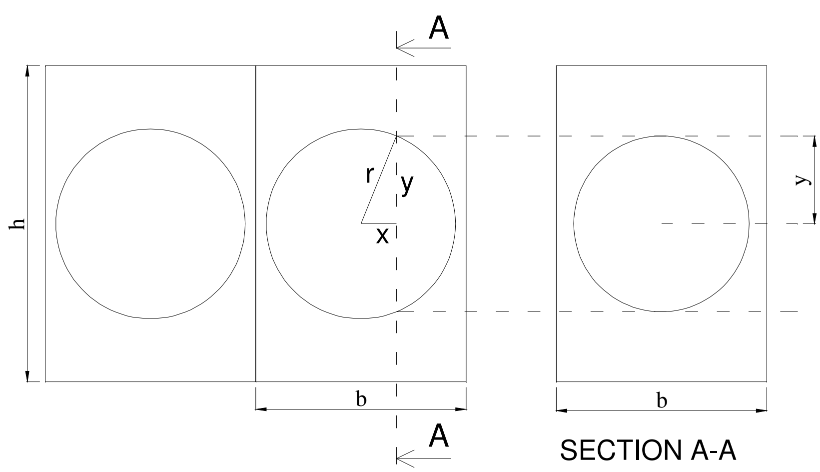

Figure 4.2: Cobiax stiffness calculation method

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Table 4.2: Cobiax Stiffness Reduction Factors

Sphere Diameter – 2r (mm)	180	180	225	225	270	315
Slab Thickness – h (mm)	280	300	340	360	400	450
Sphere Spacing c/c – b (mm)	200	200	250	250	300	350
Radius - r (mm)	90	90	112.5	112.5	135	157.5
Centroid hemisphere - x (mm)	33.8	33.8	42.2	42.2	50.6	59.1
New radius - y (mm)	83.4	83.4	104.3	104.3	125.1	146.0
Is solid (mm^4)	3.66E+08	4.50E+08	8.19E+08	9.72E+08	1.60E+09	2.66E+09
Ic circle (mm^4)	3.81E+07	3.81E+07	9.29E+07	9.29E+07	1.93E+08	3.57E+08
Sphere factor (Is-Ic)/Is	0.896	0.915	0.887	0.904	0.880	0.866
Solid factor Is/Is	1.0	1.0	1.0	1.0	1.0	1.0
Sphere %	0.9	0.9	0.9	0.9	0.9	0.9
Solid %	0.1	0.1	0.1	0.1	0.1	0.1
Stiffness Reduction Factor	0.91	0.92	0.90	0.91	0.89	0.88
E-value of Concrete for Strand7 (GPa)	23.566	24.021	23.345	23.763	23.182	22.858
Reduction in dead load (kPa)	1.909	1.909	2.386	2.386	2.863	3.340

Sphere Diameter (mm)	315	360	360	405	450
Slab Thickness (mm)	460	500	520	570	620
Sphere Spacing c/c (mm)	350	400	400	450	500
Radius - r (mm)	157.5	180	180	202.5	225
Centroid hemisphere - x (mm)	59.1	67.5	67.5	75.9	84.4
New radius - y (mm)	146.0	166.9	166.9	187.7	208.6
Second moment of area for the solid region between voids - Is solid (mm^4)	2.84E+09	4.17E+09	4.69E+09	6.94E+09	9.93E+09
Second moment of area for the voided region - Ic circle (mm^4)	3.57E+08	6.09E+08	6.09E+08	9.75E+08	1.49E+09
Sphere factor (Is-Ic)/Is (stiffness ratio of the average voided cross-sectional area in terms of a fully solid cross-section)	0.874	0.854	0.870	0.860	0.850
Solid factor Is/Is (stiffness ratio of a fully solid cross-sectional area in terms of a fully solid cross-section)	1.0	1.0	1.0	1.0	1.0
Sphere % (percentage of all possible cross-sections through the slab that will obtain internal voids)	0.9	0.9	0.9	0.9	0.9
Solid % (percentage of all possible cross-sections through the slab that will be fully solid)	0.1	0.1	0.1	0.1	0.1
Stiffness Reduction Factor	0.89	0.87	0.88	0.87	0.87
E-value of Concrete for Strand7 (GPa)	23.058	22.580	22.960	22.714	22.497
Reduction in dead load (kPa)	3.340	3.817	3.817	4.294	4.771

Referring back to *Table 4.1,* since the diagonal/x ratios are reasonably larger than 250 for the case of Cobiax slabs, it demonstrates that punching shear has governed the calculation of the slab thickness for all span lengths and load combinations, especially for the heavier load combinations. In the TUD deflection was found to be the governing factor, since their punching shear requirements are not as strict as in South Africa, combined with the use of Halfen® Shear Stirrups in Europe. These two factors allow for thinner Cobiax slabs to be used, resulting in more economical designs, still being within the maximum deflection specifications.

Horizontal Shear Resistance

The cold joint in a Cobiax slab due to the two pour system needs some investigation. Laboratory tests done in the TUD confirmed that a Cobiax slab constructed with two pours will behave the same as a slab with no cold joint. This is probably the best way to confirm the effective horizontal shear capacity, which is obtained by friction at the surface of the cold joint and the vertical cage bars passing through the cold joint. A concrete slump between 120 mm and 140 mm will generally result in easier workability of the first concrete layer of a Cobiax slab, and are therefore strongly recommended for this layer.

In South Africa a decision has been made to continue with the Cobiax cages into the solid zones, to act as reinforcement chairs separating top and bottom reinforcement. Both the solid and voided zones of a Cobiax slab will be performed in two pours. Since the vertical shear from an ultimate limit state (ULS) loading condition is used in the formula for horizontal shear calculation, the critical position for the testing of horizontal shear will be where the punching shear reinforcement is discontinued and only the cages continue. This position where the relevant vertical shear (in the y-direction) can be obtained is shown in *Appendix H*, on the line where the white zone changes to a coloured zone.

The contour plot in this appendix is for the 12 m span Cobiax slab exposed to heavy loading. The highest vertical shear will exist for this slab, as well will the vertical cage bars be the furthest apart, providing the least shear resistance of al slabs investigated for the purpose of this dissertation. The large spacing of vertical cage bars is due to the largest Cobiax sphere size (450 mm diameter) used for this 620 mm thick slab.

TMH7 Part 3 (1989) is a South African code that provides a method to test the longitudinal shear capacity at horizontal cold joints. *Section 5.4.2.3* provides formulae for this shear resistance. The V_l value for ultimate vertical shear force per meter width referred to in this section is obtained from the well-known formula:

$V_1 = VAy/I$

Where:

V = 375 kN/m, which is the vertical shear at the critical position for a meter width.

A = 1m x (0.62 – 0.1)m = 0.52m^2, or

A = 1m x 0.1m = 0.1m^2,

where a 100 mm first pour height and 1 m slab width are assumed, and A is the area either below or above the cold joint.

y = (0.62/2)m – (0.52/2)m = 0.05m, or

y = (0.62/2)m – (0.1/2)m = 0.26m,

where y is the distance from the centroid of slab area either above or below the cold joint, measured to the centroid of the area of the total slab thickness.

Therefore:

Ay = 0.52m x 0.05m = 0.026m^3, or

Ay = 0.1m x 0.26m = 0.026m^3,

which should be exactly the same.

I = $bh^3/12$ = [1m x (0.62m)^3]/12 = 19.861 x 10^{-3} m^4

Then:

V_1 = 1x(375 x 0.026)/19.861 x 10^{-3} = 491 kN/m,

for a 1 m length along the span of the slab.

This V_1 value should in accordance with TMH7 Part 3 not exceed the lesser of:

$k_1 f_{cu} L_s$ = 2700 kN/m (or N/mm)

$v_1 L_s + 0.0007 A_e f_y$ = 593 kN/m (or N/mm)

Where:

$k_1 = 0.09$ for surface type 2 described as a surface where laitance removal was performed with air or water, and no other surface treatment conducted.

$f_{cu} = 30$MPa which is the characteristic cube strength of concrete

$L_s = 1000$mm which is the width of the shear plane (or cold joint)

$v_1 = 0.45$MPa which is the ultimate longitudinal shear stress in the concrete taken from *Table 30* of *TMH7 Part 3*, for surface type 2.

$A_e = 16 \text{ x } \pi d^2/4 = 452 \text{ mm}^2$,

which is the area of anchored reinforcement per unit length crossing the shear plane, and where *d* is the vertical cage bar diameter. This unit length was taken to be 1 m when calculating V_1. For 450 mm diameter Cobiax spheres, 16 cage bars of 6 mm diameter each will cross this shear plane for every square meter of slab area.

$f_y = 450$MPa which is the characteristic strength of the cage reinforcement.

TMH7 Part 3 as well as *SANS 10100* stipulates that the minimum reinforcement crossing the shear plane should be:

0.15% x Area of contact = 0.0015 x 1 m^2 = 1500 mm^2

This value is greater than that of A_e, and therefore the vertical cage reinforcement is insufficient. A simple investigation will show that only 6 mm diameter cage bars for the 180 mm and 225 mm diameter Cobiax spheres will exceed the 1500 mm^2/m^2 minimum horizontal shear reinforcement requirement. These sphere sizes include all Cobiax slabs up to 360 mm thickness. For thicker Cobiax slabs the minimum horizontal shear reinforcement requirements will not be satisfied.

The spacing of the vertical cage reinforcement bars may not exceed the lesser of four times the minimum thickness of the second concrete pour or 600mm. The maximum spacing of these bars is less than 500 mm for all sizes of Cobiax cages, and therefore this requirement of *TMH7 Part 3* is met.

Since the TUD laboratory tests showed the Cobiax slabs to be safe, one might question whether this was also true for slab thicknesses exceeding 360 mm, which will not meet the minimum horizontal shear reinforcement requirements. Also, whether or not the cage reinforcement is truly fully anchored, remains unclear and needs further investigation.

A counter argument may be that almost no vertical shear rebar will be required through the cold joint, since the code requirements are based on precast members that may be a couple of days old before receiving a topping, while the second pour of a Cobiax slab generally follows within four hours of the first pour. This will allow for less differential creep and shrinkage to take place at the cold joint, which will limit the reduction in shear strength on this plane.

A South African solution will be to increase the cage reinforcement thickness for Cobiax slabs thicker than 360 mm. Setting $A_e = 1500$ mm^2 for a 1 m^2 area of cold joint and then dividing A_e by the area of a single cage bar, choosing different bar diameters, will indicate the number of these bars required to cross the 1 m^2 area. The following number of bars will satisfy minimum horizontal shear reinforcement requirements for different reinforcement diameters through a 1 m^2 area:

- 53 bars for 6 mm diameter bars
- 30 bars for 8 mm diameter bars
- 19 bars for 10 mm diameter bars
- 14 bars for 12 mm diameter bars

The number of bars crossing a 1 m^2 area for different Cobiax cages are:

- 100 bars for 180 mm diameter Cobiax sphere cages
- 64 bars for 225 mm diameter Cobiax sphere cages
- 44 bars for 270 mm diameter Cobiax sphere cages
- 33 bars for 315 mm diameter Cobiax sphere cages
- 25 bars for 360 mm diameter Cobiax sphere cages
- 20 bars for 405 mm diameter Cobiax sphere cages
- 16 bars for 450 mm diameter Cobiax sphere cages

The above summary clearly shows that the following cage reinforcement diameters are required:

- 6 mm diameter bars for 180 mm and 225 mm diameter Cobiax sphere cages
- 8 mm diameter bars for 270 mm and 315 mm diameter Cobiax sphere cages
- 10 mm diameter bars for 360 mm and 405 mm diameter Cobiax sphere cages
- 12 mm diameter bars for 450 mm diameter Cobiax sphere cages

Although this would result in a very practical solution for satisfying the minimum horizontal shear reinforcement requirements for Cobiax slabs, unfortunately it will increase the cost of the Cobiax item.

4.5. COFFER SLABS

Punching Shear

Eight-noded finite element plate models were created for all three load combinations and five span lengths in Strand7, resulting in 15 models for the coffer slabs alone. *Figure 4.1* displays square areas around the columns which result in approximately 25% of the total slab area. These areas will remain solid to accommodate shear that cannot be resisted by the webs of the coffers alone.

The coffer solid zones obtained from a Strand7 analysis are shown in *Appendix F*, where the white areas around the columns are to be left solid, and the remaining area should be supplied with the applicable coffer moulds. One can limit the solid zones to approximately 25% of the slab area, and simply add some shear stirrups in the webs where additional shear is required. For the 10 m span slab model under light loading in *Appendix F* one would typically have to add shear stirrups in just over half a meter of web length away from the solid zone. This will only be required in some areas of the slab and the rest of all the webs can be left without stirrups. The example in *Appendix F* was the most critical case of all coffer models analysed, having the largest solid zones. Comparing *Figure 4.1* with the coffer plot in *Appendix F*, it is clear that the size of the square solid zones assumed in *Figure 4.1* simulate the real solid zone scenario quite accurately, and therefore the coffer models in this report can be used with confidence.

The same procedure used for Cobiax slabs was used for coffer slab punching shear design, utilising Prokon software. The Prokon punching shear calculation output for an internal column is displayed in *Appendix I*, using the 7.5m span scenario as an example for the coffer models. Here punching reinforcement also made up a very small percentage of the total reinforcement content. Nevertheless this approximated punching reinforcement was added to the reinforcement content displayed later in this report in *Tables 4.7 to 4.9*. The same reasoning for obtaining valid results from a Prokon design discussed in Chapter 4.2, applies here.

<u>*Deflection*</u>

The slab thicknesses of coffer slabs were mainly governed by deflection. Coffer thicknesses could only be 425, 525 and 625 mm thick, where commonly available coffer sizes with 100 mm toppings had been used, which is the maximum allowable topping. This provides a strange non-constant long-term deflection variation between coffer slabs with different span lengths and loading conditions. The deflection became too severe for the 12 m span coffer slab under heavy loading (see *Table 4.1*). One can also show that from 13 m span lengths, even for the light load combination, no commonly available coffer slab will meet the deflection requirements. Therefore one can assume that the use of coffer slabs ends with approximately 12 m lengths, unless a special coffer mould with increased depth, or post-tensioning in combination with the coffers, is used.

A stiffness reduction factor had to be calculated for all types of coffer slabs. The same approach was followed as that used for Cobiax slabs. The E-value of the pink areas (voided zones) of the coffer models (see *Figure 4.1*) was simply reduced by the stiffness reduction factors in *Table 4.3* for the applicable slab thickness. By also adding an upward load over these voided zones for reduction in dead load, obtained from *Table 4.3*, one could obtain the correct elastic deflection values for any coffer slab.

The reduction in dead load was simply the displaced concrete weight (25 kN/m^3) as a result of the coffer voids outside the solid regions, which differs for the different sizes of coffer moulds.

In *Table 4.3* I_{solid} was calculated with the formula $I_{solid} = b_f(A+h_f)^3/12$ and represented the second moment of area of a flat slab with no coffer voids. I_{coffer} was equal to a T-section's second moment of area, with a tapering web (calculated with areas A_1, A_2 and A_3). The stiffness reduction factor here was directly obtained by calculating the I_{coffer}/I_{solid} ratio. Unlike with Cobiax slabs, the change along the span length to a totally solid section does not happen gradually, but very suddenly, and therefore it would be dangerous to assume that part of the span along the coffers will have the stiffness value of a completely solid slab.

Multiplying the E-value of 26 GPa with this stiffness reduction factor as explained earlier, will then provide the designer with a new E-value (see *Table 4.3*) for the purpose of deflection calculations with either hand calculation methods or finite element software. *Figure 4.2B* explains the symbols used in *Table 4.3* for the coffer system.

Table 4.3: Coffer Stiffness Reduction Factors

Coffer Type (+100mm topping)	900x900x325	900x900x425	900x900x525
A - Coffer height (mm)	325	425	525
B – Web width at soffit of topping (mm)	258	298	338
B_{av} – Average web width (mm)	193	213	233
C – Minimum web width at bottom (mm)	128	128	128
h_f - Flange Thickness (mm)	100	100	100
b_f - Flange Width (mm)	900	900	900
A_1 – Flange area of section (mm²)	90000	90000	90000
A_2 – Web area of section (mm²)	41600	54400	67200
A_3 – Tapering web area of section (mm²)	10562.5	18062.5	27562.5
y - Centroid from bottom (mm)	295.2	357.5	417.7
I_{coffer} - Second moment of area (mm^4)	2.00E+09	3.84E+09	6.56E+09
I_{solid} (mm^4)	5.76E+09	1.09E+10	1.83E+10
Stiffness reduction factor = I_{coffer}/I_{solid}	0.35	0.35	0.36
E-value of Concrete for Strand7 (GPa)	9.037	9.203	9.316
Reduction in dead load (kPa)	4.875	6.025	7.125

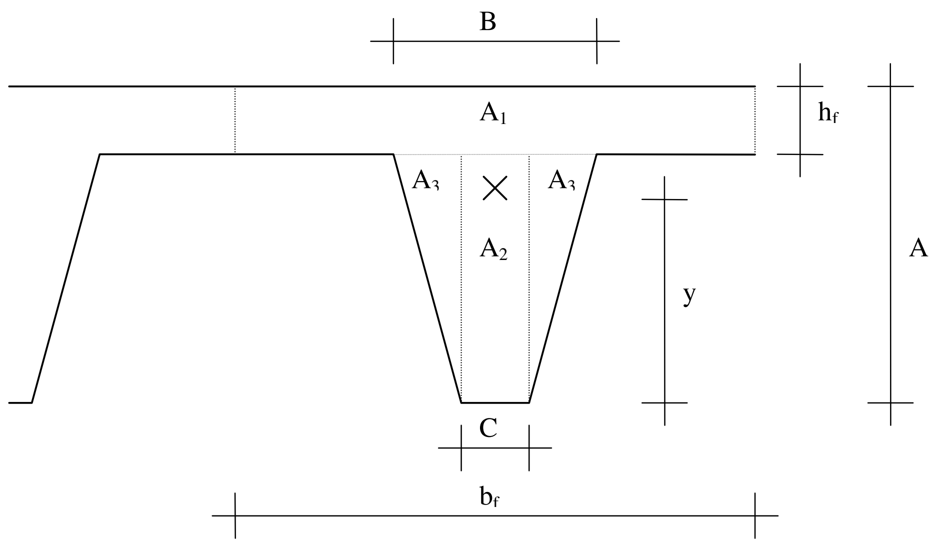


Figure 4.2B: Coffer system

4.6. POST-TENSIONED SLABS

Punching Shear

Eight-noded finite element plate models were created in Strand7 for all three load combinations and five span lengths, resulting in 15 models for the unbonded post-tensioned slabs alone. Punching shear reinforcement was designed with the help of the Prokon Captain software, and results are displayed only for the 7.5m span scenario in *Appendix J*. The presence of the cables in the slabs contributed significantly to shear resistance, making very thin slabs possible. Punching shear reinforcement made up a very small percentage of the total reinforcement content. Nevertheless this approximated punching shear reinforcement was added to the reinforcement content displayed later in this report in *Tables 4.7 to 4.9*.

Deflection

Appendix K discusses the cable design methodology of every post-tensioned slab model in detail, using a Mathcad software program mainly developed by Dr John Robberts. Only the calculations for the light load scenario on the 7.5m span slab system are displayed as an example. The cables were banded (100 mm c/c spacings) in the x-direction and uniformly distributed in the y-direction. The equivalent loads on the slabs after long-term losses occurred (*Appendix K*) were applied to the slabs for both directions of cables. The forces were applied in the form of uniform distributed loads (UDL), being downward over supports and upward away from supports over distances as calculated in *Appendix K*. The application of this UDL significantly reduces deflections.

By taking the cables to balance only 70% of the dead load, which is an average value that designers may use, one can assume a class 3 structure in accordance with TMH7 Part 3 (1989) as a result, where additional normal reinforcement will be critical to carry the remainder of the loads.

The results in *Appendix K* were tested against those obtained in Prokon, and very similar deflections and equivalent loadings were obtained. The slab thicknesses were determined with a formula that would normally suggest a thickness satisfying punching, deflection and vibration requirements. Punching shear requirements dictated slab thicknesses for the lighter loadings, and deflection that of heavy loading (see *Table 4.1*). The deflections seen in this table were both for the maximum obtained on a diagonal line between columns, and that for a normal span length, between two columns.

As from 12 m span lengths, heavy loading on post-tensioned slabs causes the slab thickness to be dictated by punching shear requirements. Post-tensioned slabs rapidly increase in thickness beyond 12 m spans and become uneconomical due to unacceptable volumes of concrete, also resulting in heavier columns and foundations. The number of cables also becomes excessive for spans greater than 12 m, causing congestion of cables to occur.

Post-tension content

The cost of post-tensioning was calculated as displayed in the *Appendix K* example, and from there a cost per kg could be established as displayed in *Table 4.4*, resulting in an average cost for the post-tensioning content displayed in *Table 4.6*.

Table 4.5 was used to create *Figure 4.3*. This figure displays the difference in post-tensioning content for different span lengths and load intensities. The increase of post-tensioning weight versus increase in span length ratio was almost linear.

Table 4.4: Calculation of Post-tension cost per kg of tendons and anchors

Load	**Cost (R/m^2)**	**Weight (kg)**	**Weight (kg/m^2)**	**Cost (R/kg)**
7.5m light load	49	659	1.3	38
7.5m medium load	65	878	1.7	38
7.5m heavy load	81	1098	2.2	38
9m light load	63	1252	1.7	37
9m medium load	80	1581	2.2	37
9m heavy load	96	1911	2.6	37
10m light load	74	1830	2.0	36
10m medium load	95	2343	2.6	36
10m heavy load	110	2709	3.0	36
11m light load	85	2577	2.4	36
11m medium load	104	3141	2.9	36
11m heavy load	120	3625	3.3	36
12m light load	97	3514	2.7	36
12m medium load	114	4130	3.2	36
12m heavy load	124	4484	3.5	36
				37

Table 4.5: Post-tension Content

Load	Span (m)	Weight (kg/m²)
Light	7.5	1.3
	9	1.7
	10	2.0
	11	2.4
	12	2.7
Medium	7.5	1.7
	9	2.2
	10	2.6
	11	2.9
	12	3.2
Heavy	7.5	2.2
	9	2.6
	10	3.0
	11	3.3
	12	3.5

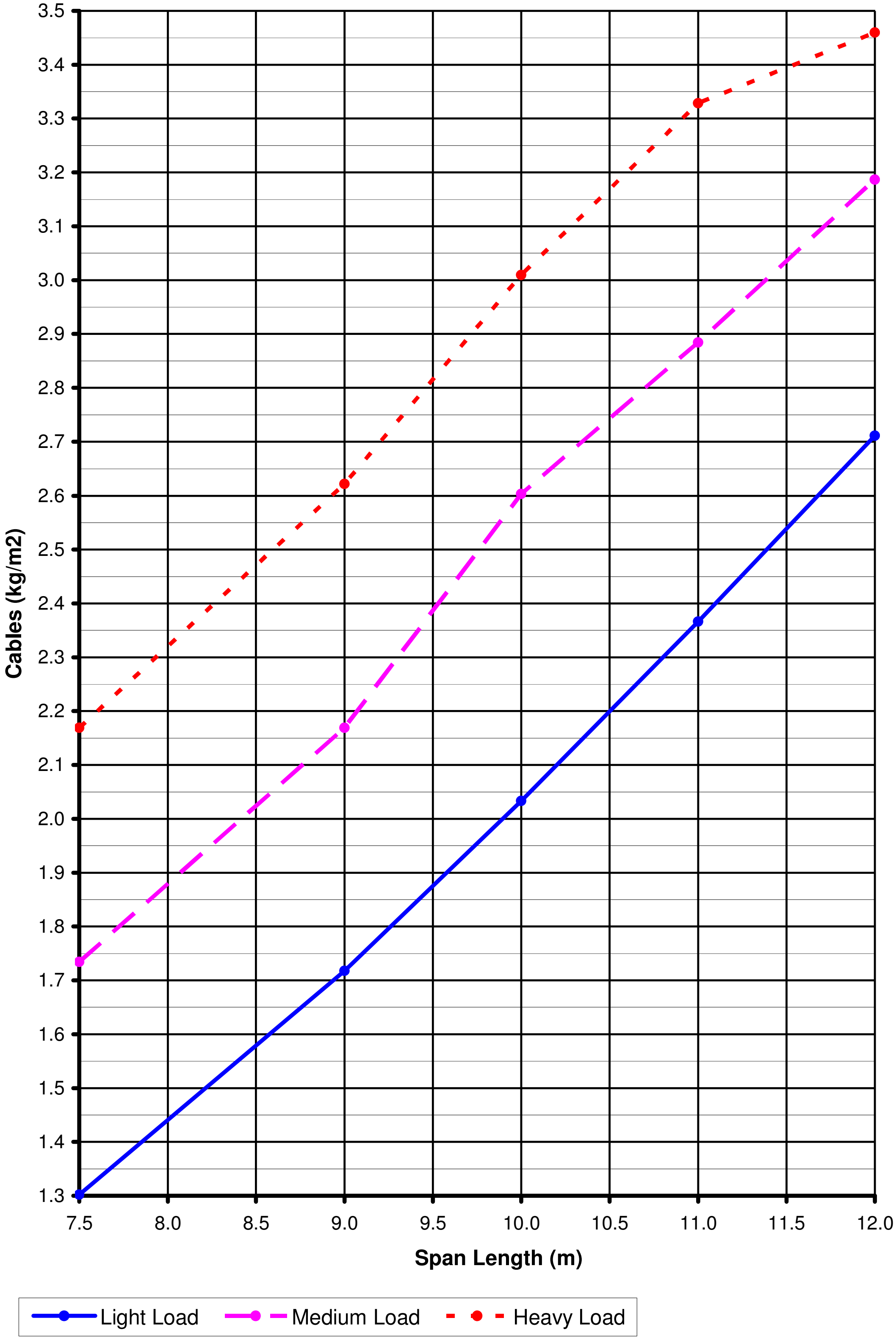


Figure 4.3: Post-tension Content

4.7. RESULTS

The values used in *Table 4.6* can vary from location to location in South Africa, and therefore only resemble the average rates for materials during December 2007. These values were based on various engineers', contractors' and quantity surveyors' opinions.

The values in *Table 4.6* were used to create *Tables 4.7 to 4.9*, where the concrete content of coffer and Cobiax slabs were calculated, assuming 25% of the slab to be solid. *Table 4.7* contains the results for light loading, *Table 4.8* for medium loading, and *Table 4.9* for heavy loading. These tables were used to generate the graphs in *Figures 4.4 to 4.15*, which were scrutinised to explain the economy of the different slab systems for different loadings and span lengths.

Table 4.6: Material Cost 2007

Concrete (R/m³)	1100
Reinforcement (R/kg)	9.50
Cost Post-tension (R/kg)	36.50
Flat-slab Formwork (R/m²)	64
Coffer Formwork (R/m²)	114
Cobiax Component	
Cobiax sphere diameter (mm)	(R/m²)
180	139
225	140
270	150
315	186
360	215
405	233
450	240
*NOTES Costs exclude VAT Costs include: - Delivery on site - Labour - Reinforcement cages and spheres (Cobiax) - 10% contractor's mark-up (Cobiax) - Cables, sleeves & anchors (Post-tension)	

Table 4.7: Cobiax, Coffer & Post-tensioned Slab Cost Comparison - Light Load

Additional Dead Load = 0.5 kPa

Live Load = 2.0 kPa

Span (m)	Concrete (m^3/m^2)			Reinforcement (kg/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	0.223	0.279	0.220	16.8	13.3	15.9
9.0	0.243	0.279	0.270	23.8	19.5	21.3
10.0	0.268	0.279	0.310	28.4	24.5	24.9
11.0	0.314	0.344	0.350	30.6	28.5	30.2
12.0	0.360	0.411	0.380	35.3	31.1	33.6
Span (m)	Slab Thickness (mm)			Cost (R/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	280	425	220	608	547	504
9.0	300	425	270	696	606	626
10.0	340	425	310	769	653	715
11.0	400	525	350	851	763	822
12.0	460	625	380	981	861	900

Table 4.8: Cobiax, Coffer & Post-tensioned Slabs Cost Comparison - Medium Load

Additional Dead Load = 2.5 kPa

Live Load = 2.5 kPa

Span (m)	Concrete (m^3/m^2)			Reinforcement (kg/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	0.223	0.279	0.230	22.0	16.5	20.9
9.0	0.268	0.279	0.280	29.8	24.6	28.2
10.0	0.314	0.344	0.325	31.6	28.6	29.8
11.0	0.360	0.411	0.370	35.1	31.2	35.7
12.0	0.405	0.411	0.410	42.8	37.1	36.8
Span (m)	Slab Thickness (mm)			Cost (R/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	280	425	230	657	578	578
9.0	340	425	280	783	655	719
10.0	400	525	325	860	765	800
11.0	460	625	370	979	863	915
12.0	520	625	410	1132	919	981

Table 4.9: Cobiax, Coffer & Post-tensioned Slabs Cost Comparison - Heavy Load

Additional Dead Load = 5.0 kPa

Live Load = 5.0 kPa

Span (m)	Concrete (m^3/m^2)			Reinforcement (kg/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	0.288	0.279	0.250	29.1	26.6	34.1
9.0	0.350	0.344	0.310	33.0	31.3	43.4
10.0	0.385	0.411	0.360	39.3	34.6	47.4
11.0	0.441	0.411	0.400	45.0	43.1	47.6
12.0	0.477	-	0.510	51.3	-	47.6
Span (m)	Slab Thickness (mm)			Cost (R/m^2)		
	Cobiax	Coffer	Post-tension	Cobiax	Coffer	Post-tension
7.5	360	425	250	798	674	742
9.0	450	525	310	948	790	913
10.0	500	625	360	1076	895	1020
11.0	570	625	400	1210	976	1078
12.0	620	-	510	1316	-	1204

Concrete content

Figure 4.4 and *Figure 4.8* indicated the Cobiax slab system to provide the greatest concrete savings for the light and medium load conditions respectively. Due to the rigidness of coffer slab thicknesses, it can be seen in *Figure 4.4* that coffer slabs had the highest concrete content for light loading, from where post-tensioned slabs required slightly more concrete between 9 m and 11 m spans, but then again coffers the most for 12 m span slabs. The dots, instead of lines, used in the graphs for concrete content and slab thickness of coffer slabs were due to the fact that a line could never represent coffer slabs, having only three possible slab depths.

For medium loading (*Figure 4.8*) the concrete content of post-tensioned slabs almost matched those of the Cobiax slabs, and coffer slabs showed to be the heaviest. For heavy loading (*Figure 4.12*) the concrete content of Cobiax and coffer slabs will be approximately the same, with coffer slabs delivering no results for the 12 m span design as explained in earlier discussions. Interesting is to note that for the case of heavy loading, the post-tensioned slabs will be the lightest slab system.

Only direct material cost benefits were taken into account when looking at the concrete content. It should be borne in mind though that, especially for high buildings, lighter slab systems can result in enormous cost savings on support and foundation structures. The Cobiax system clearly displays this benefit for light loading, and therefore might be an attractive slab option for multi-level car park structures.

<u>*Reinforcement content*</u>

For light and medium loading conditions (*Figure 4.5 and 4.9*), coffer slabs will require the least reinforcement due to its larger slab depth, and therefore greater internal lever arms between the compression block and tension steel of the section. Post-tensioned slabs have the benefit of the cables balancing a great percentage of the total load, and therefore Cobiax slabs end up requiring the most reinforcement for light and medium loading conditions.

Figure 4.13 showed that for heavy loading conditions coffer slabs still require the least reinforcement, but in this case, post-tensioned slabs the most. This scenario occurred due to the fact that because of the high live load, a much smaller percentage of the total load has been balanced by the cable forces. The thin post-tension slabs therefore resulted in sections with small internal lever arms, requiring a lot of tension reinforcement. Due to the rapid increase in thickness of post-tensioned slabs close to 12 m spans for heavy loading, in order to resist punching effects, the thicker post-tension slab for a 12 m span had a greater lever arm. This explained why the reinforcement content was less than that of Cobiax slabs for these conditions.

<u>*Slab thickness*</u>

For all loading conditions (see *Figures 4.6, 4.10 and 4.14*) post-tension slabs had the smallest slab thicknesses and coffer slabs the largest. Although this was not taken into account for this cost study, again for high buildings with multi-level floors, Cobiax and post-tension slabs may have cost benefits in terms of vertical services and construction material required such as brickwork. Finishes to buildings with excessive heights can also result in high costs.

<u>*Direct material cost*</u>

The graphs comparing costs of the different slab systems, as displayed in *Figures 4.7, 4.11 and 4.15*, showed that the Cobiax system will be the most expensive and coffer slabs the cheapest for all loading conditions over large span lengths. *Table 4.6* clearly states what this cost study took into account, mainly being direct material costs. As earlier mentioned many other costs should also be taken into account to obtain a true display of the cost effectiveness of a slab system. Sadly in South

Africa very few quantity surveyors, contractors and engineers go the distance to calculate the indirect cost effects of different slab systems.

Preliminary cost estimates

The graphs in *Figures 4.3 to 4.15* can be used by designers to do preliminary cost estimates. For concrete content and slab thickness, should the spans differ, e.g. 9 m by 11 m spans, the designer should always take the reading on the graph for the largest span length to accommodate deflection requirements, thus 11 m in this example. Reinforcement and post-tensioning content may be obtained from a reading at the average span length, for this example at a 10 m span.

Take for instance the medium loading condition for a slab with a 9 m by 11 m column grid. From *Figure 4.8* at an 11 m span length, the concrete content for a Cobiax slab system will be 0.36 m^3/m^2. The slab thickness (*Figure 4.10*) for this same scenario will be 460 mm, also for an 11 m span length, and the reinforcement content (*Figure 4.9*) approximately 31.6 kg/m^2, taken at the average span length of 10 m. A 460 mm thick Cobiax slab will require 315 mm Cobiax spheres, resulting in a Cobiax component cost (see *Table 4.6*) of $R186/m^2$ in the year of 2007. Flat-slab formwork will be required, costing $R64/m^2$. Using the cost rates in *Table 4.6*, the total cost per square meter for this scenario will be:

$$0.36 \text{ x } R1100 + 31.6 \text{ x } R9.5 + R186 + R64 = R946.20/m^2$$

More conservatively the designer can read the cost directly from *Figure 4.11*, at the highest span length of 11 m, which will indicate a cost of R978.93 for this system. This will overestimate the more likely cost of the slab by 3.5%.

The preliminary cost and quantity estimates for Cobiax slabs can not be established at this time in South Africa with the Cobiax preliminary design graph in *Figure 2.14.1*, since this graph was based on European design standards that are much less strenuous on shear requirements, as well as assuming the use of Halfen shear reinforcement and 35 MPa concrete cylinder strength (43.75 MPa cube strength). These factors will cause Cobiax slabs not to be dominated by punching requirements, but rather deflection requirements, making much thinner Cobiax slabs possible. The designer should refer to the figures in this chapter only for South African Cobiax slab cost estimates.

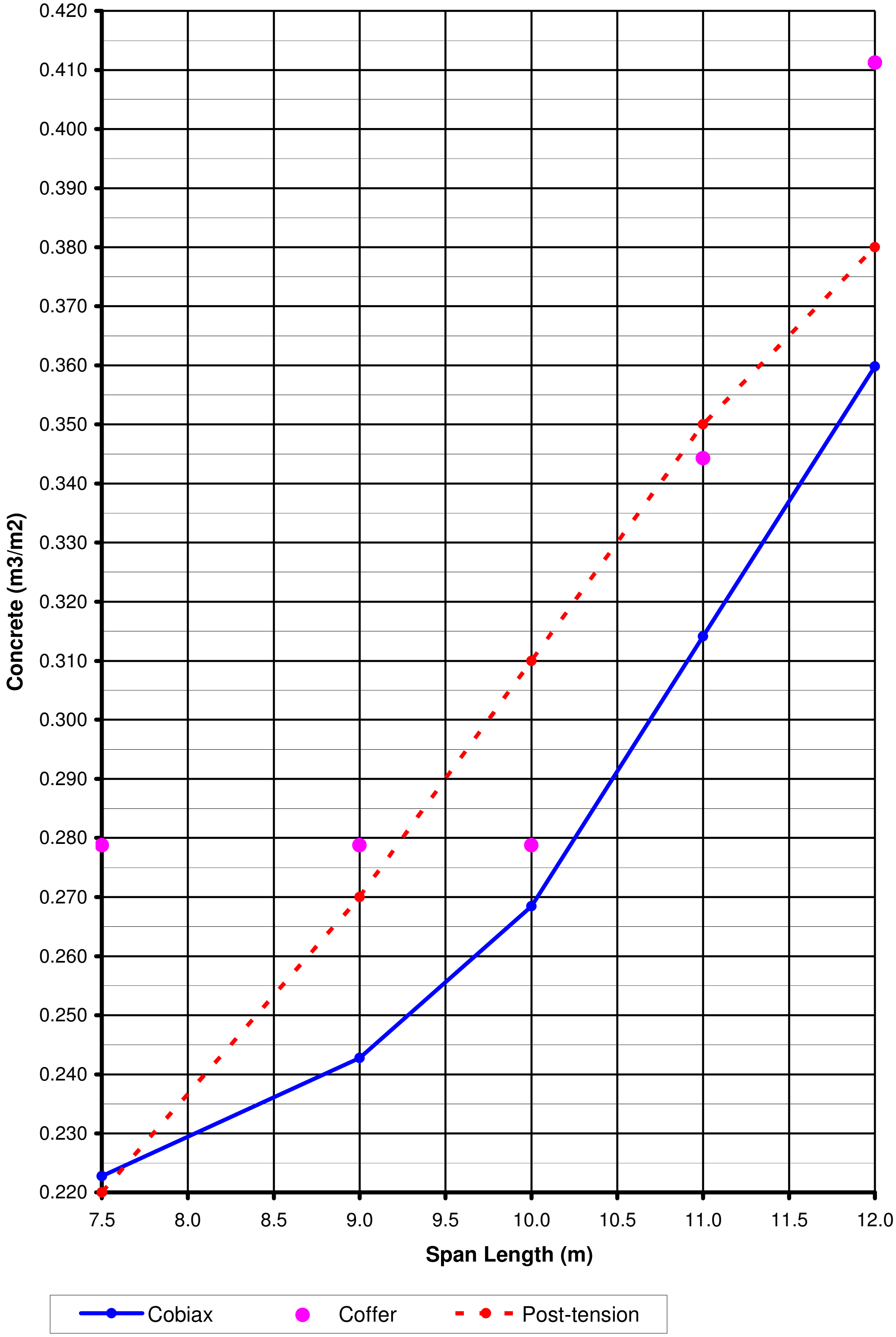


Figure 4.4: Concrete Content of Slab Systems [SDL=0.5kPa & LL=2.0kPa]

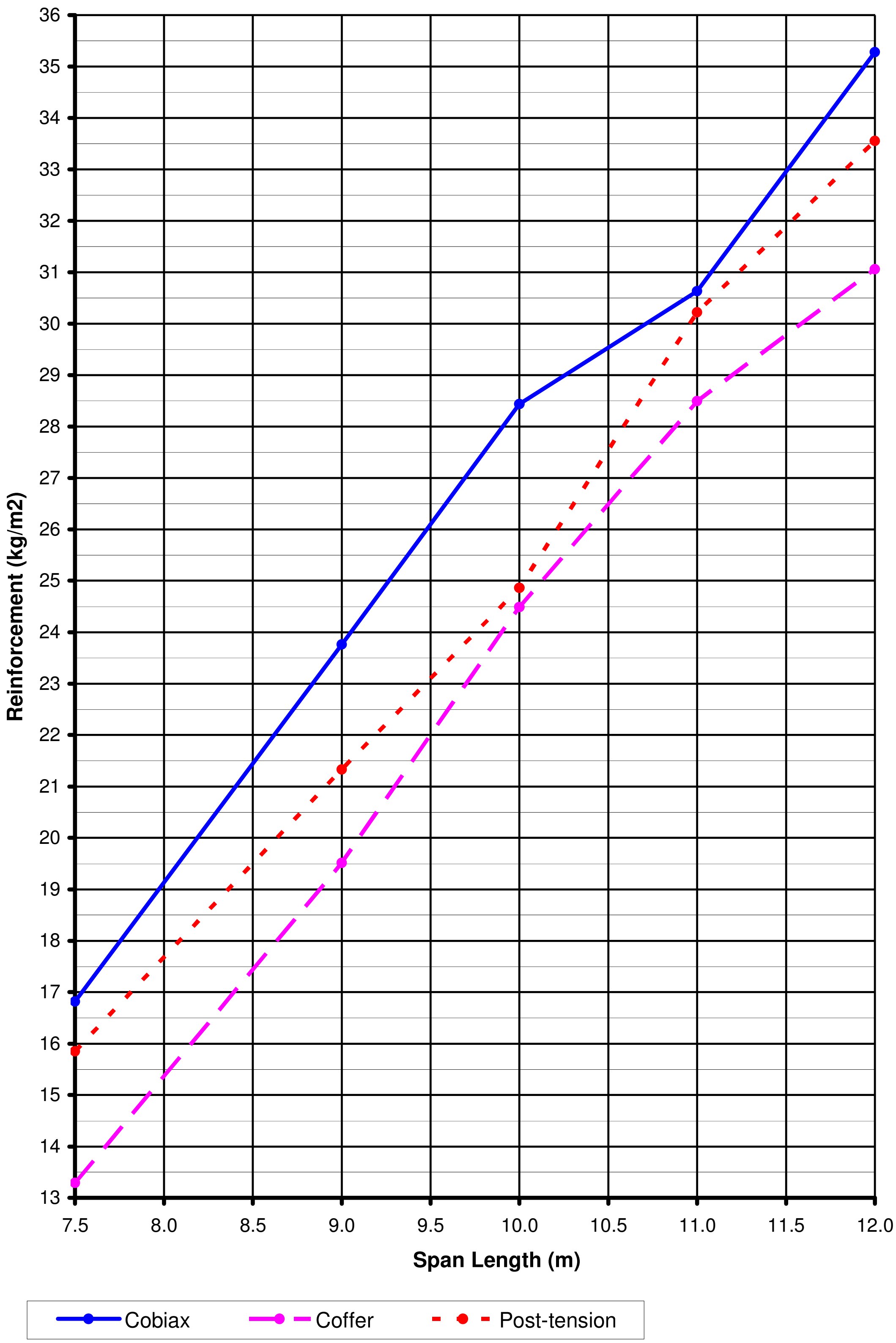


Figure 4.5: Reinforcement Content of Slab Systems [SDL=0.5kPa & LL=2.0kPa]

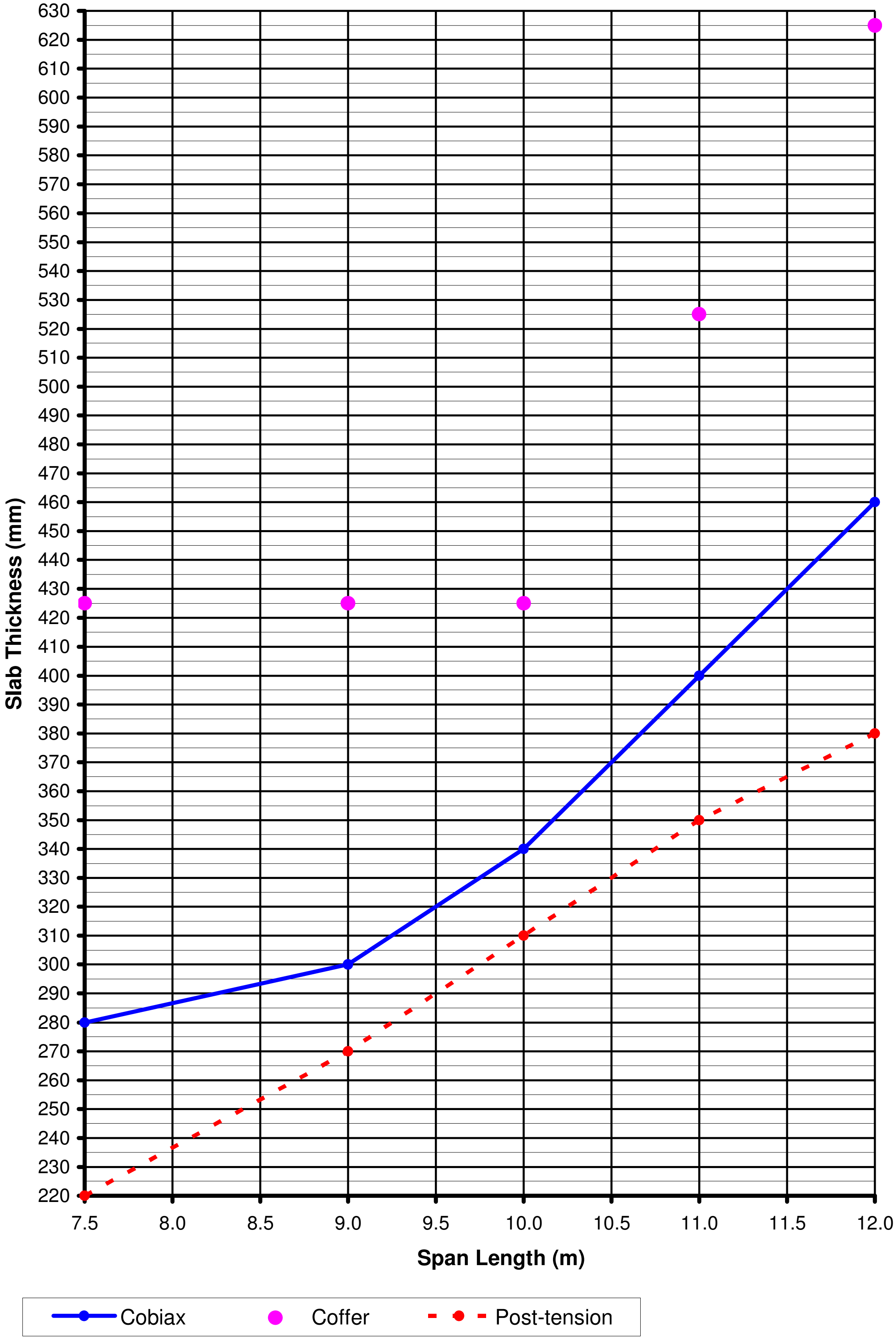


Figure 4.6: Slab Thickness of Slab Systems [SDL=0.5kPa & LL=2.0kPa]

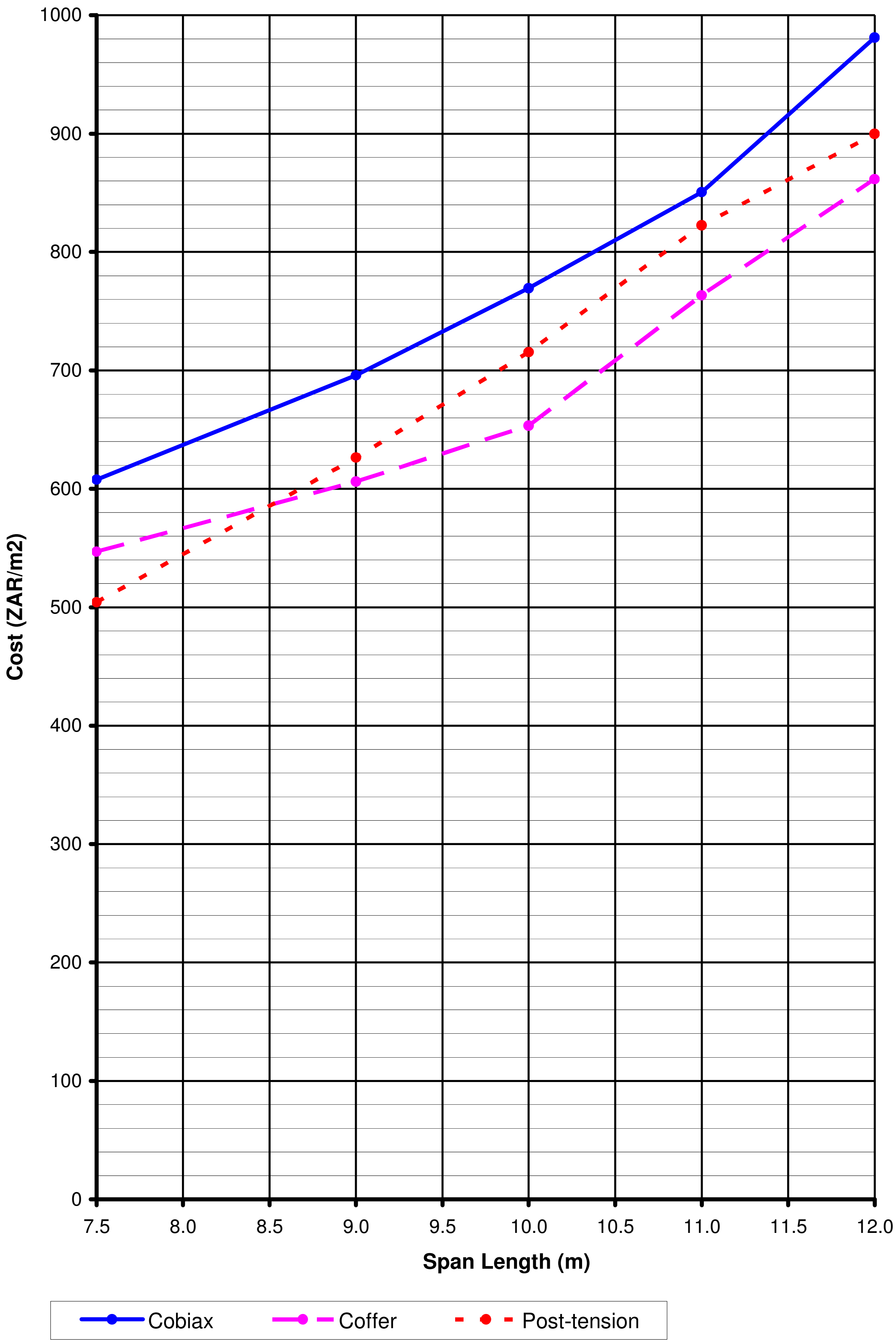


Figure 4.7: Cost of Slab Systems [SDL=0.5kPa & LL=2.0kPa]

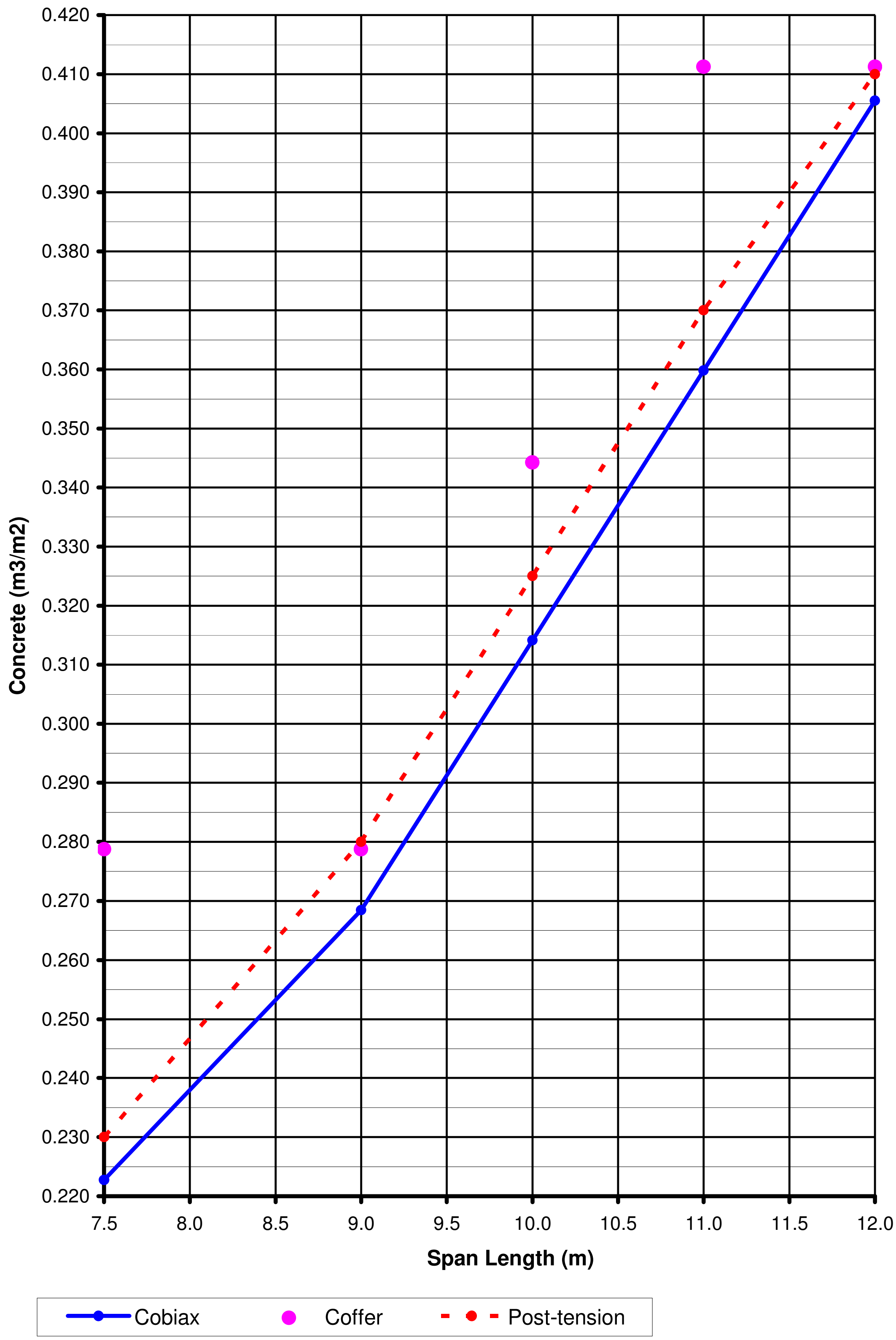
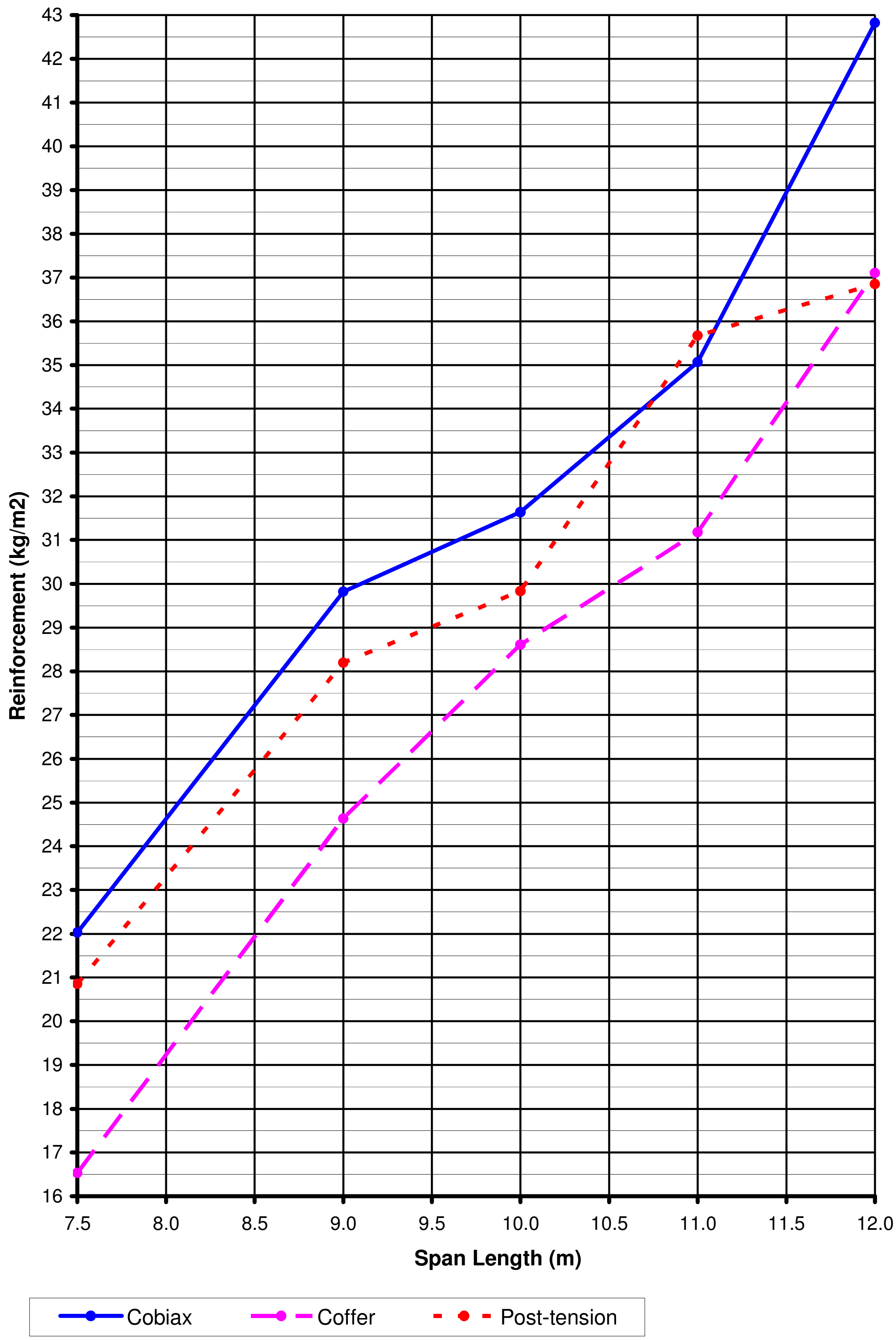


Figure 4.8: Concrete Content of Slab Systems [SDL=2.5kPa & LL=2.5kPa]



<u>**Figure 4.9: Reinforcement Content of Slab Systems [SDL=2.5kPa & LL=2.5kPa]**</u>

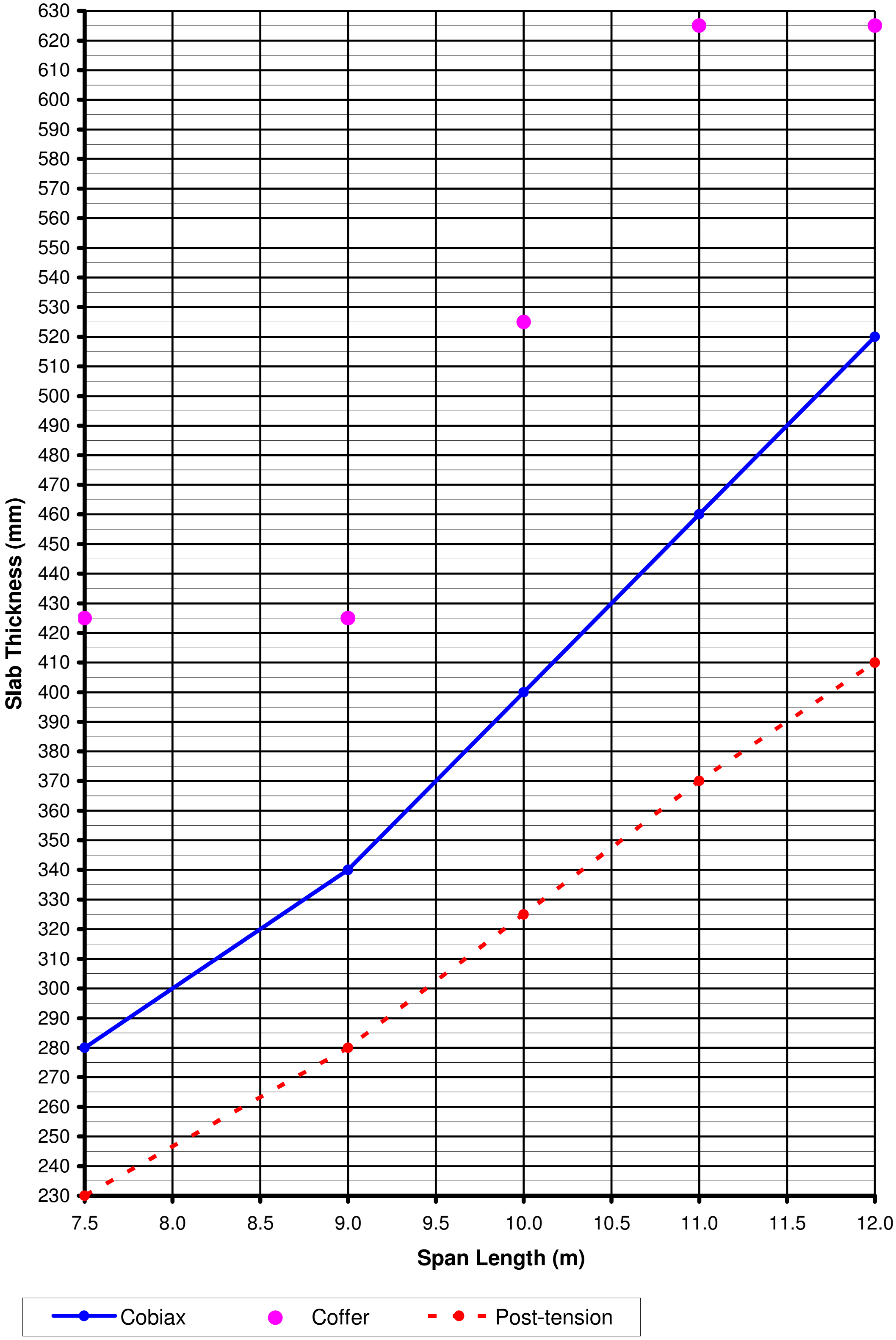


Figure 4.10: Slab Thickness of Slab Systems [SDL=2.5kPa & LL=2.5kPa]

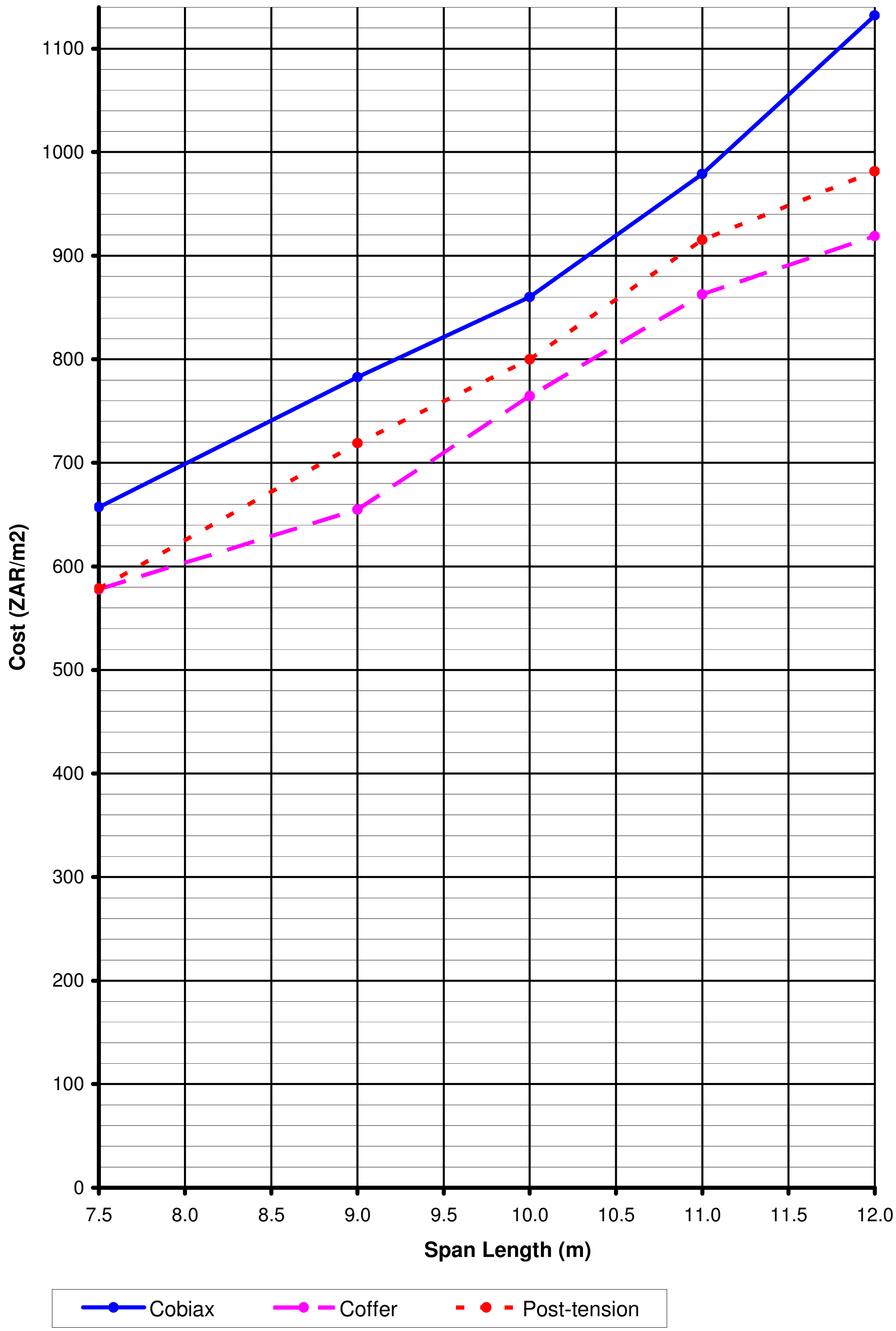


Figure 4.11: Cost of Slab Systems [SDL=2.5kPa & LL=2.5kPa]

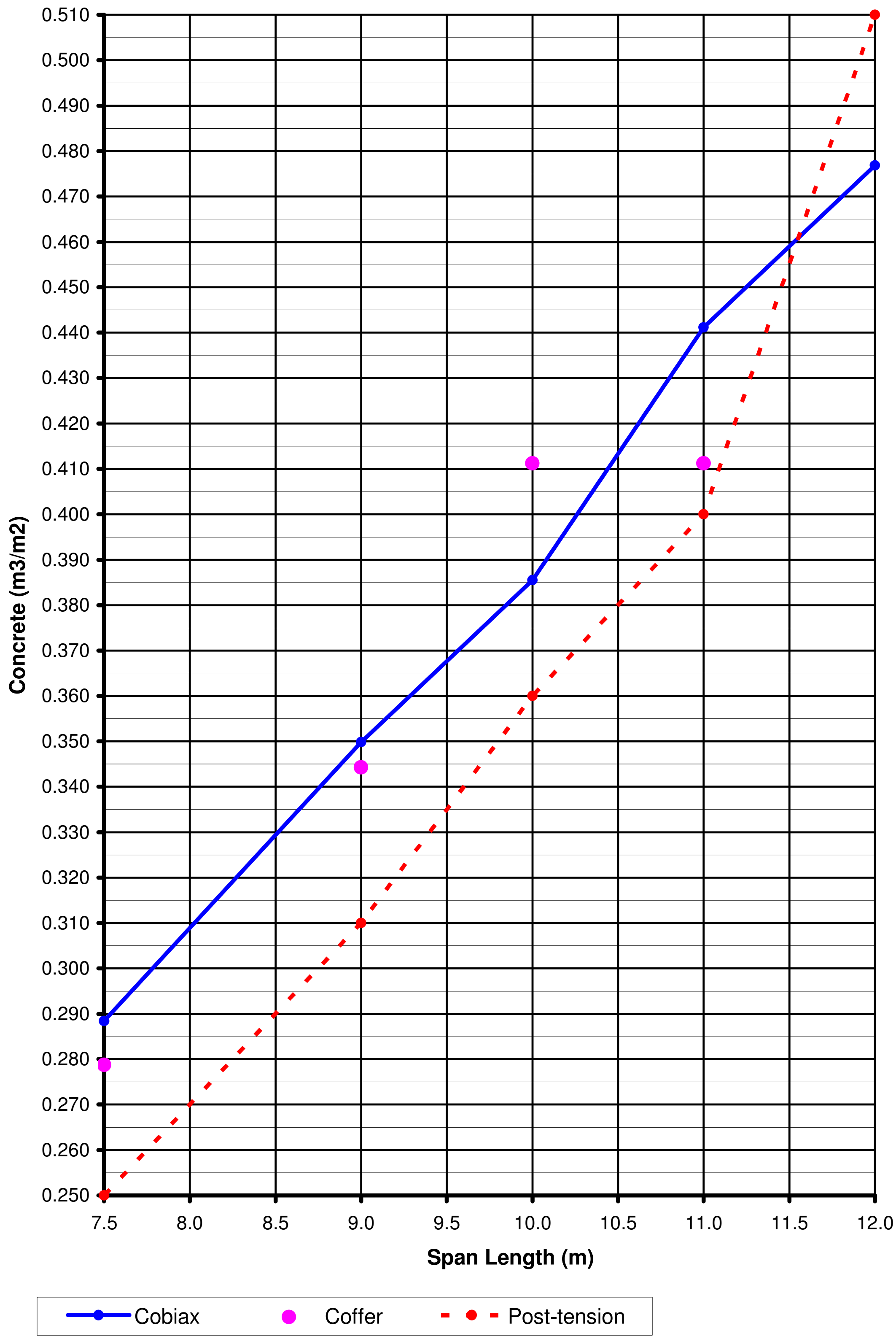


Figure 4.12: Concrete Content of Slab Systems [SDL=5.0kPa & LL=5.0kPa]

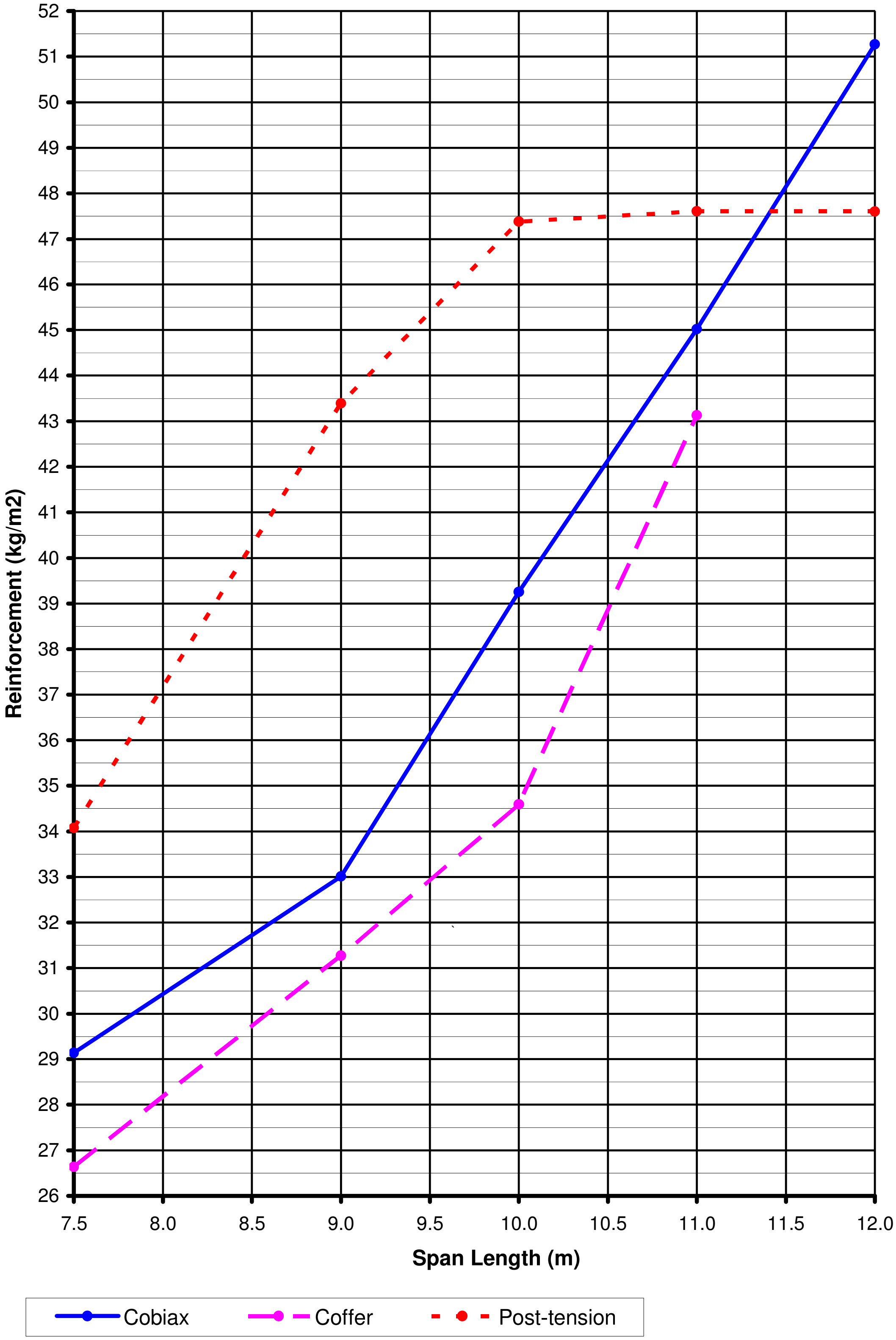
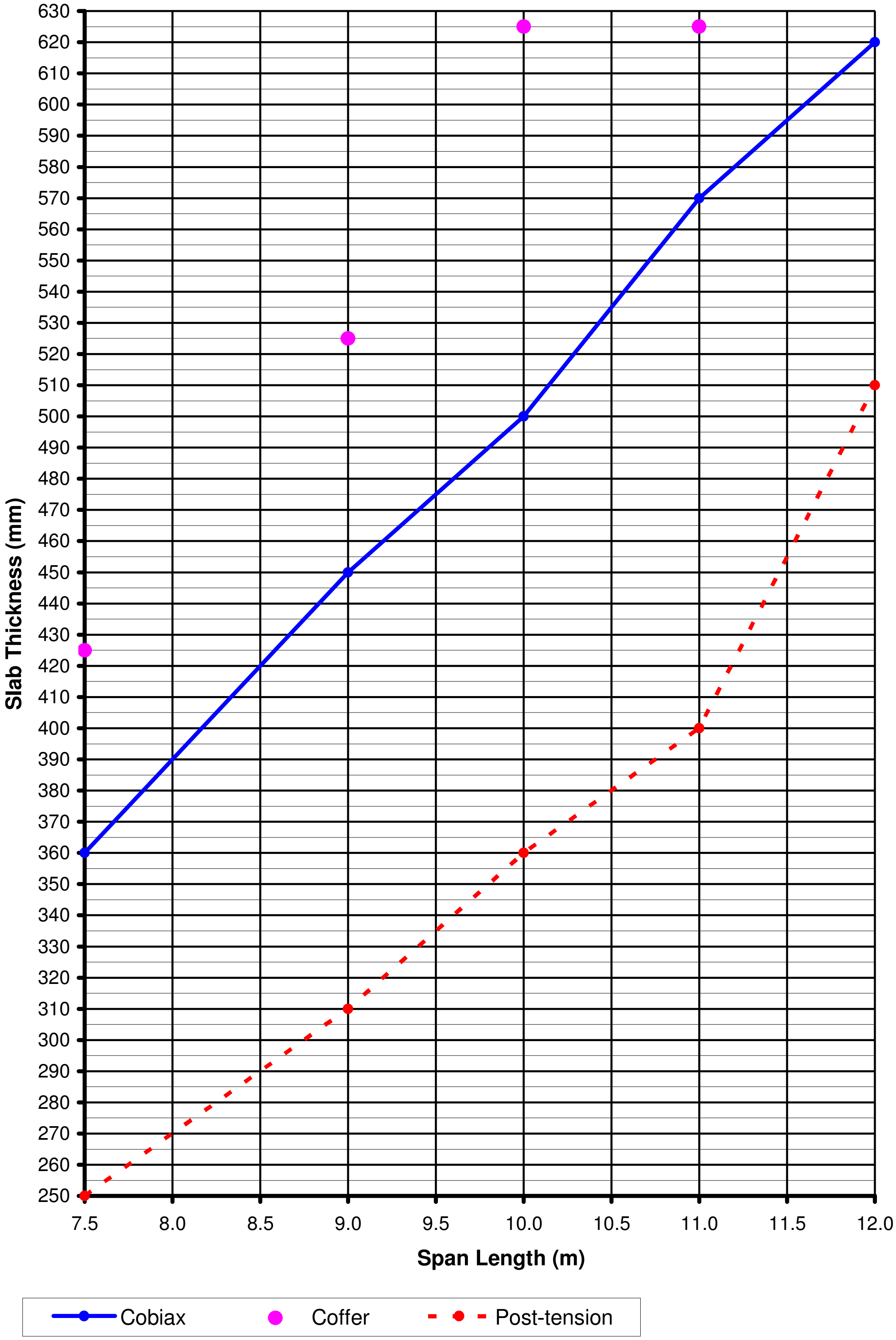


Figure 4.13: Reinforcement Content of Slab Systems [SDL=5.0kPa & LL=5.0kPa]



<u>Figure 4.14: Slab Thickness of Slab Systems [SDL=5.0kPa & LL=5.0kPa]</u>

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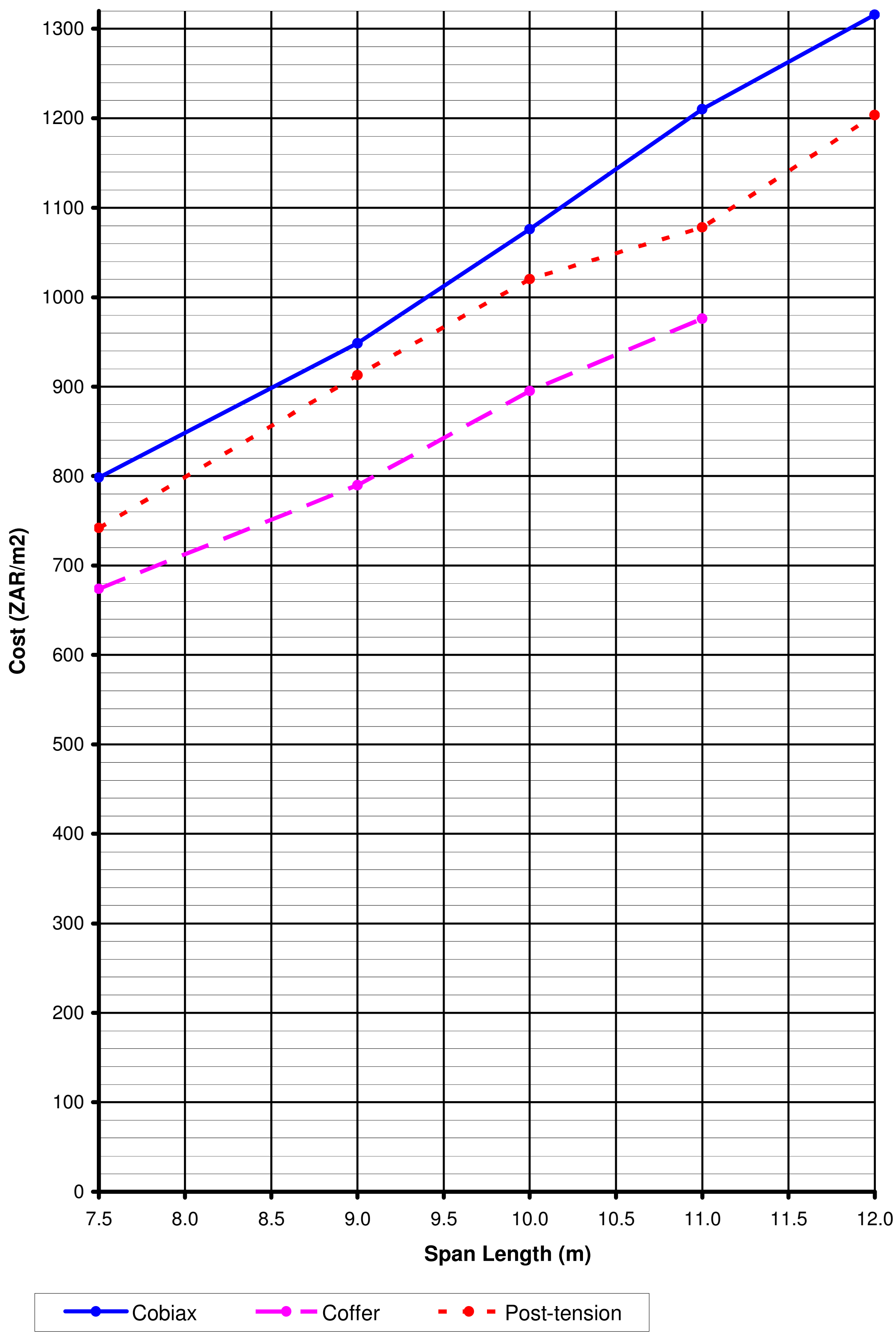

Figure 4.15: Cost of Slab Systems [SDL=5.0kPa & LL=5.0kPa]

5. CONCLUSIONS AND RECOMMENDATIONS

The cost effectiveness of flat-slabs with internal hollow spherical void formers was investigated in this report. The only system of this kind that is available in South Africa is the European Cobiax system. The report did not only investigate the cost relationship of Cobiax to that of other large span slab systems in the country, but also how the SANS 10100-01:2000 concrete design code applies to the system.

Experimental test at the University of Pretoria showed the Cobiax system to operate at higher shear resistance levels than obtained in German research. This was mainly due to the fact that German researchers conservatively ignored the additional shear capacity of the Cobiax steel cages. The shear reduction factor of 0.55 can be taken as 0.85 according to the laboratory tests done in South Africa, yet further research is required to establish whether this a higher factor will result in cost benefits. The probability that a higher factor is likely to be applicable should rather serve to ease the mind of the design engineer after applying the 0.55 factor to v_c.

In order to establish a more exact factor due to the contribution of the Cobiax cages, further test should be performed on Cobiax slabs, comparing solid samples, with solid samples plus cages, with samples containing both the hollow spheres and their cages. The design code formula for normal shear reinforcement will not be applicable for the vertical bars of the cages, since these cages are not fully anchored. Therefore testing samples as described above will display much more trustworthy results. Such testing is already in progress in Germany.

The conditions at the cold joint due to the two pour system with Cobiax slabs will have sufficient horizontal shear capacity according to the results in this report, but will not fulfill the minimum horizontal shear reinforcement requirements, unless the cages are made up of the following reinforcement diameters:

- 6 mm diameter bars for 180 mm and 225 mm diameter Cobiax sphere cages
- 8 mm diameter bars for 270 mm and 315 mm diameter Cobiax sphere cages
- 10 mm diameter bars for 360 mm and 405 mm diameter Cobiax sphere cages
- 12 mm diameter bars for 450 mm diameter Cobiax sphere cages

The cost comparison that was performed included Cobiax, coffer and unbonded post-tensioned flat-slabs, where all systems were considered to have no drop or column heads. Span lengths ranged from 7.5 m to 12 m spans, and light to heavy loading for normal commercial buildings only was applied. Strand7 finite element analysis software was used to perform the analysis on 45 different models for the various slab types and conditions.

This was done to establish the cost effectiveness of the Cobiax flat-slab system in South Africa. The costs were based on concrete, reinforcement, formwork, post-tension (for post-tensioned slabs alone), and Cobiax (for Cobiax slabs alone) content. These costs were mainly based on direct slab material costs, and the quantity surveyor should investigate other economical implications when choosing a specific slab system as well.

The Cobiax system, based on the direct cost of the slab, turned out to be more expensive than the other two systems for all conditions. This can partly be remedied by introducing Halfen shear links in South Africa, since punching shear dominates Cobiax slab thicknesses, that increases costs. One would rather have thinner slabs dominated by deflection requirements. Halfen shear stirrups will allow the designer to use thinner Cobiax slabs for the same conditions, being much more effective in shear resistance than the normal South African shear clips, and easier to install. This will not only reduce the concrete content of Cobiax, but also the weight on columns and foundations, as well as the overall building height. Cost savings will be a result. The designer can also rather use the Eurocode 2 design code for his slab design, since the requirements for shear resistance are less strenuous than that of SANS 10100.

Cobiax nevertheless shows a cost benefit for light loading such as car parking levels. Due to less concrete content than other slab systems investigated, indirect cost savings can be considerable in terms of column and foundation types and sizes, especially for high, multilevel parking buildings.

Cobiax will also be a very efficient system when dealing with long span slab systems with complex column gridlines and openings in the slabs, or scattered columns. Here coffers and post-tensioning might be difficult to apply.

Due to the thinner slab thicknesses of Cobiax and post-tensioned slabs, lower overall building heights are possible. Especially with high multi-storey buildings, there can be considerable savings on vertical construction material (such as brickwork), services and finishes, by reducing the building's total height.

Cobiax slabs can be safely designed with the SANS 10100-01:2000 design code in combination with suggestions made in this report. Although the direct material cost of Cobiax slabs is higher than coffer and post-tensioned slabs, the Cobiax system can be utilised especially with the construction of high multi-level buildings, with large cost benefits as a result.

4.8. CONCLUSION

After modelling various equal span three-span by three-span slab models in Strand7 with eight-noded rectangular finite element plate elements, a cost comparison between Cobiax (representing SVFS in South Africa), coffer, and unbonded post-tensioned slabs could be executed in accordance with SANS 10100 (2000) and TMH7 Part 3 (1989). The cost comparison included direct material costs only, taking the effect of various span lengths between 7.5 m and 12 m and uniformly distributed load applications into account.

The SVFS (Cobiax) system resulted in the most expensive large span slab system, with coffer slabs being the cheapest for almost every span length and load application scenario, this mainly being the result of the high Cobiax component costs (Cobiax cages and spheres).