THE NORWEGIAN PAVILION AT EXPO '92, DESIGN OF A FIBER REINFORCED CONCRETE STRUCTURE

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ABSTRACT

The design of the fiber reinforced concrete tube at the Norwegian pavilion at EXPO'92 is presented and discussed. The test programme performed to establish the material design parameters is explained and experience gained during the construction period is stated.

Key words: Fiber reinforced shotcrete, test programme, design, structural application

1. INTRODUCTION

The Norwegian pavilion at EXPO'92 in Sevilla in Spain was a 45 meter long concrete "tube" with a diameter of 6.66 meter. The architect wanted a relatively thin walled structure in a material that could be associated with norwegian "know how" and industry. Fiber reinforced shotcrete satisfied these criteria and was selected as the material for the tube.

The basis for the design of the concrete tube is presented in this paper together with experiences form the construction period. A sketch of the pavilion with the tube is presented in Fig 1.

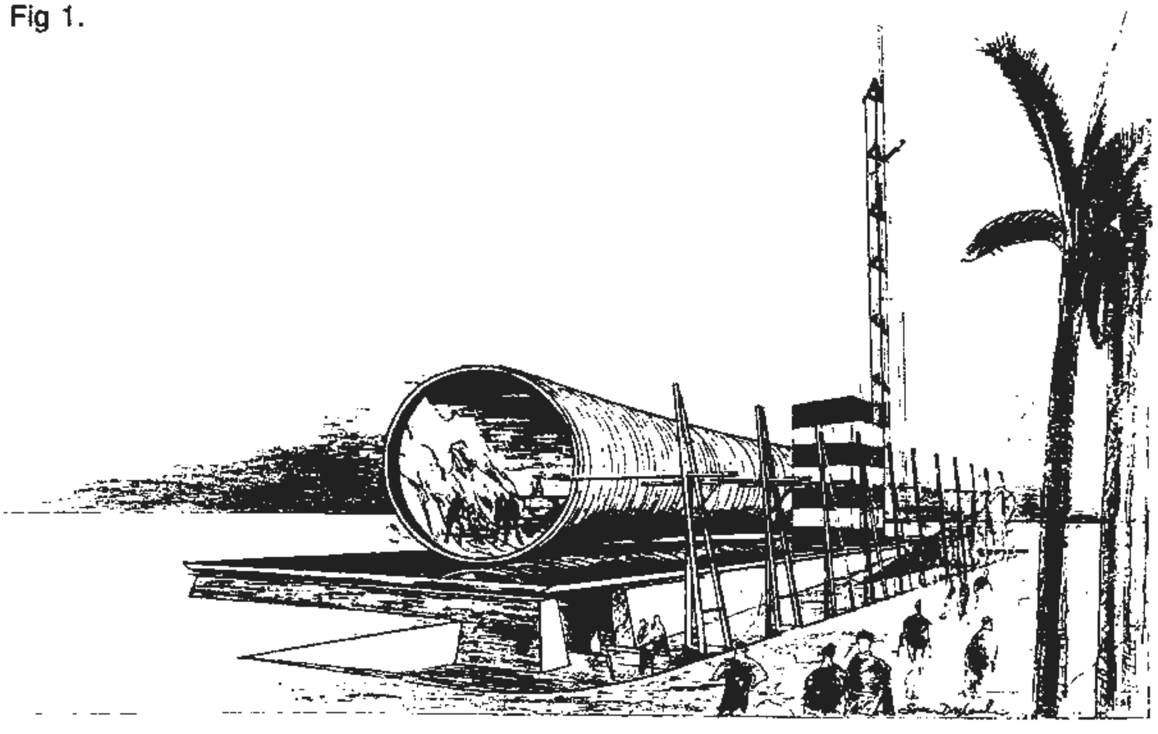

Fig 1 The Norwegian pavilion, EXPO'92

2. DESIGN PRINCIPLES FOR FIBER REINFORCED SHOTCRETE

2.1 Test parameters and test specimens

The most important parameter that was decided by the test programme was the tensile bending strength. Initial studies had shown that bending moments were significant while shear and axial forces were small in the arch of the tube.

"Plates" of fiber reinforced concrete were produced and test specimens for bending strength testing, axial tensile strength and compression strength were sawn from these plates.

The plates were produced in three layers of concrete, each 4 to 5 cm thickness to a total thickness of 14 cm, which was equal to the thickness of the arch of the tube. The plates were produced by shotcreteing and stored in-door until tested. They were not covered by plastic or similar.

2.2 Concrete mix design

The following mix design was used in production:

Cement P30	525 kg/m^3
Silica	40 kg/m^3
Sand 0-6 mm	1400 kg/m^3
Betokem P	3 l/m^3
Sikament 100	3-6 l/m^3
Water	230 l/m^3
Accelerator max	10 l/m^3
18mm EE-fiber	90 kg/m^3

2.3 Test programme

2.3.1 Bending strength

The bending strength of the fiber concrete was determined from one point load testing on beams (100x100x600mm) sawn from the produced plates (ref. section 2.1). The load was applied at the middle of the 500 mm span. Some control beams (100x100x850mm) with two point load testing at distance 250 mm of the 750 mm span were included to see if the one point test was representative for a moment distributed over a longer distance.

The measured bending strength of the test specimens was considered to be about 10 to 15% higher than expected on the actual structure due to the difference in height (100 mm in test compared to 140 mm actual).

The most important parameter in the bending test programme was the age at testing. The required curing time before the scaffolding could be removed was decided from these tests.

2.3.2 Compressive strength

The compressive strength was determined from cubes sawn from the tested beams, ref. section 2.3.1. Three cylinders were also tested to determine the modulus of elasticity.

As utilization of the compression strength was low in design, this parameter was not paid special attention during testing.

2.3.3 Tensile strength

The tensile strength was determined from "beams" (100x100x550mm) sawn from the produced plates (ref. section 2.1). The tensile strength was considered important to establish a design criteria for pure tensile loading on the structure and was also used for simple shear calculations.

2.3.4 E-modulus and creep deformations

A limited study of the E-modulus of the concrete was performed. This modulus was used as input to an FE-model used for calculation of sectional forces and deformations.

The E-modulus in compression was determined form drilled cylinders (diameter 153 mm and height 265 mm). In addition, the "tensile" E-modulus was determined form the tensile tests (ref. section 2.3.3) and the "bending" modulus from the bending tests (ref. section 2.3.1). By comparing the three moduli, a reasonable good estimate for the expected deformations could be established.

A simple creep test was also performed by applying a permanent load of 400 kg for 14 days at mid span of the 600 mm beams. This load induced a maximum bending stress of 3 Mpa at mid section.

2.4 Test results

2.4.1 Bending strength

Test results from the bending tests on the small 600 mm beams are presented as a function of age at testing in Fig. 2. The curve for "max load 1" in this figure is calculated from the maximum recorded load during the test. The curve for "max load 2" is the highest "stable" load after the peak load for max load 1 had been passed. The difference between max load 1 and 2 is illustrated in Fig 3 which is the recorded plot of a typical load-deformation curve during testing.

It is assumed that max load 1 is representing the strength of the mortar, while max load 2 represents the bending strength for the fiber/mortar interaction after the mortar has cracked. As can be seen from Fig. 2, bending strength from max load 1 are reduced as the time increases and is nearly equal to max load 2 at 14 days. It is assumed that this is caused by development of micro cracking in the mortar due to shrinkage.

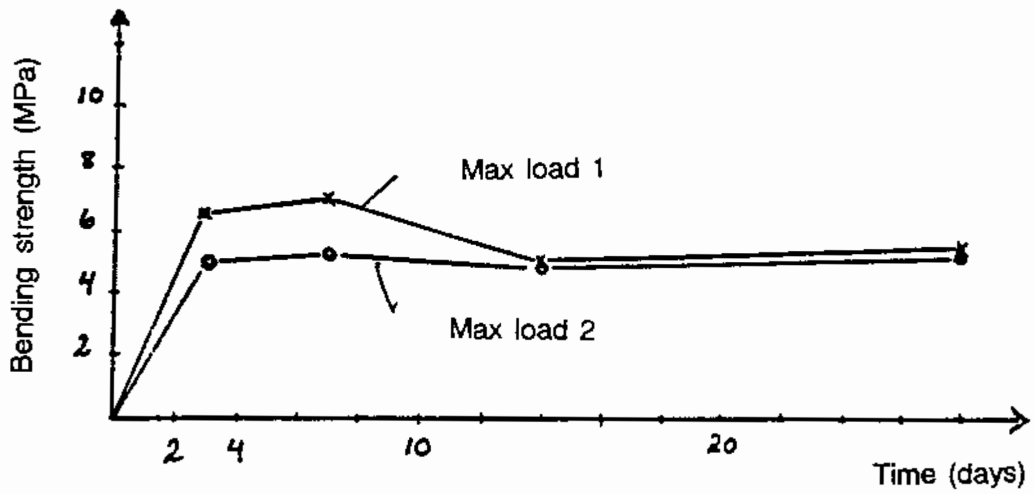

Fig 2 Bending strength as function of age at testing

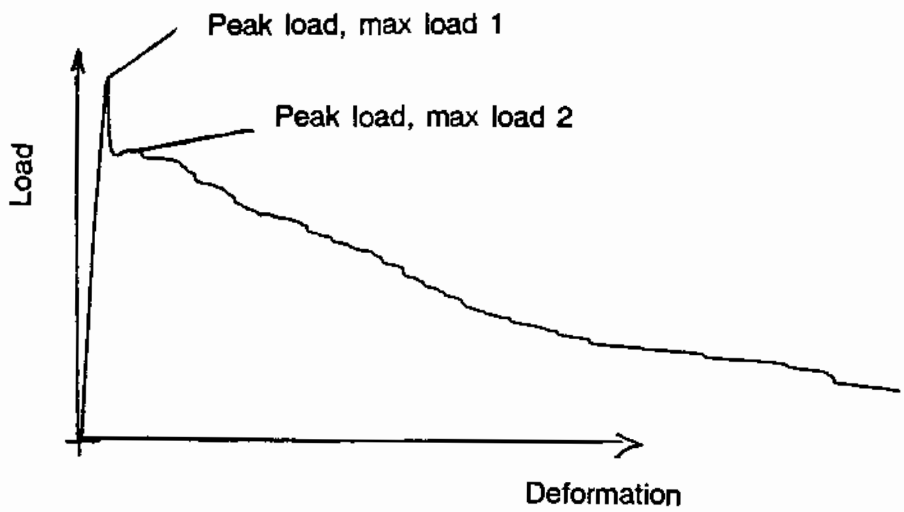

Fig 3 Illustration of max load 1 and 2, typical load-deformation curve

The design bending strength was determ'ned based on the curve for max load 2. It can be seen from Fig 2 that the bending strength based on max load 2 was close to independent of the age of the specimens if these were more than three days old. The tests also showed that the difference between short (600 mm) and long (850 mm) beams was insignificant.

2.4.2 Compression strength

The compression cube strength as function of age is presented in Fig. 4.

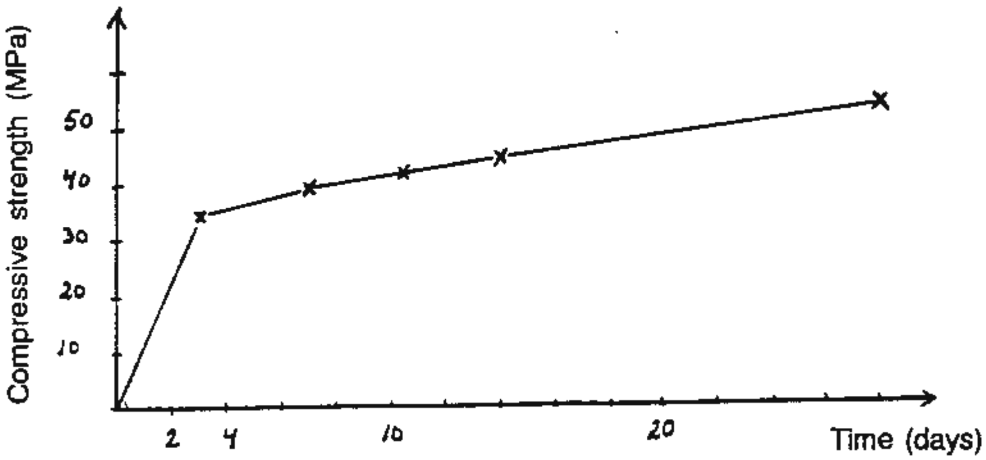

Fig 4 Compression cube strength as function of age

2.4.3 Tensile strength

The mean recorded tensile strength was 2,38 MPa with a standard deviation of 0,08 MPa at the age of 14 days.

2.4.4 E-modulus and creep deformations

Recorded mean E-modulus in compression was 24.5 GPa. Corresponding mean value in tension was 11,2 GPa. It is assumed that the low E-modulus in tension was caused by micro cracking caused by drying and shrinkage prior to testing.

Calculated E-modulus from the bending tests was in the range of 21 to 22 GPa at 3 and 7 days. For tests performed at 14 and 28 days respectively the E-modulus was calculated to 16-17 GPa. Again, micro cracking caused by drying and shrinkage prior to testing was considered the reason for the reduced values.

The creep test results showed that the time dependant deformations were significant and strongly dependant of the age at load application. When loaded at an age of 3 days, the creep deformations at 11 days of loading were close to 7 times larger than the initial elastic deformation. When loaded at an age of 14 days, the creep deformations were only three times initial deformations during the same load interval.

Deformations at mid span of the beam (average of three tests) as function of time in shown in

Fig 5.

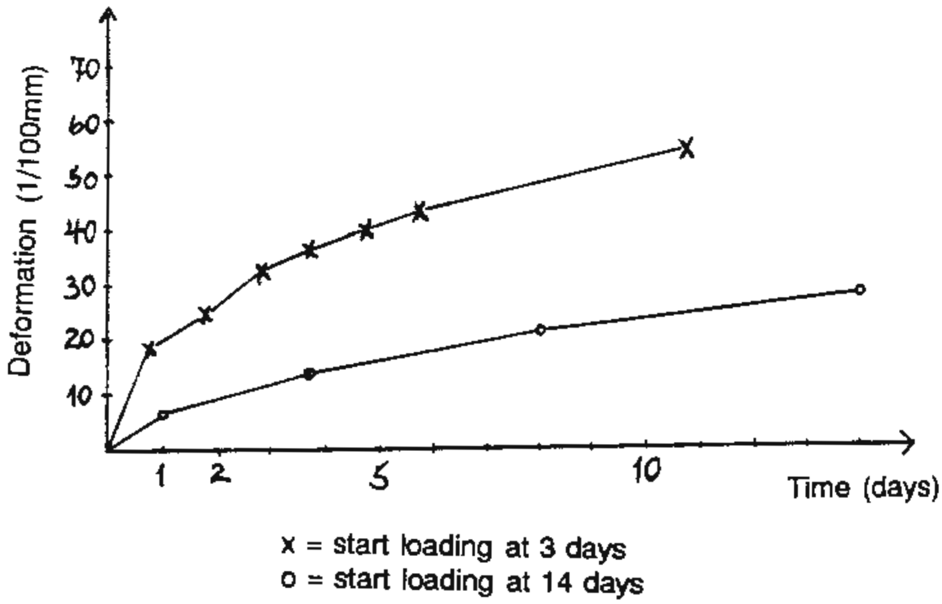

Fig 5 Creep deformations at mid span of test beams for different age at loading as function of time

2.5 Establishing design strength

2.5.1 Design bending strength

The test results from testing at an age of 14 days were used as the basis for the design bending strength. This was considered reasonable as it was a requirement to the actual structure that it should be supported by scaffolding during this 14 days period and was not allowed to carry any load.

10 tests were performed at age 14 days, 5 on the 600 mm beams and 5 on the 850 mm beams. The mean value was 5.06 MPa and the standard deviation was 0,54 MPa, all based on "max load 2", ref Fig. 3. The characteristic strength was calculated to 4.14 MPa, i.e. mean value minus 1,7 times the standard deviation according to the Norwegian Concrete Code NS 3473, section 16.5 /1/.

Then the material coefficient had to be decided. According to /1/ 1,4 is the standard material coefficient for concrete. /1/ also states that if the tensile strength of the concrete is governing for the failure mode, the material coefficient should be doubled. Due the high volume per cent of steel fibre reinforcement (1,1%) this criteria was relaxed as a 50% increase of the material coefficient was required for concrete with "insufficiently anchored" reinforced.

Consequently the material coefficient 2,1 was selected and the design bending strength was determined to be 2,0 MPa. From a trial, full scale model made in Norway prior to the construction work started in Spain, some bending tests were performed on sawn beams from the tested structure. Bending strength in the range of the characteristic strength was calculated from these tests.

2.5.2 Design compression strength

The design compression strength was determined according to Table 5 in NS 3473 /1/ as for ordinary concrete.

2.5.3 Design strength in tension

The same assumptions as made for the bending strength was used to determine the tensile strength. 1,1 MPa then became the design tensile strength.

The shear capacity was then checked according to NS 3473 /1/ by assuming A_s equal to zero. The utilization of the shear capacity was low in all sections of the structure, and this subject was not paid special attention.

2.5.4 E-modulus used in design

Bending moments were the dominating load response on the fiber reinforced concrete. E-modulus calculated from bending test on "cracked" concrete beams, i.e. 17 MPa, were therefore selected as input to the FE analysis.

3 DESIGN OF THE TUBE

The base of the tube were made of ordinary reinforced concrete. This part covered 80° of the circle (ref. Fig 6). This base carried the main part of all live loads and distributed the weight of the fiber-reinforced shotcrete to the supports. Splicing bars were placed at the intersection between the ordinary concrete and the shotcrete.

Apart from some ordinary reinforcement bars above openings and in longitudinal direction towards the ordinary cast concrete, the shotcrete was un-reinforced. The design was based on the parameters described in section 2.5

The detailed design showed that the bending design strength of the shotcrete was exceeded in critical areas. This could not be compensated for by increasing the thickness of the arch. It was therefore decided to use prestressing cables placed on the outer face of the tube after the circular tube was completed. These cables induced a hoop compression forces in the ring direction of the tube and thereby limited the tensile stresses due to local bending moments.

After the design was completed, a full scale section of the tube was built in Oslo and load tested to failure. The tested section had no prestressing cables, but was otherwise close to identical to the actual structure. The test showed that the fiber reinforced concrete had a brittle failure with no "warning" before collapse. The fibers were unable to redistribute any stresses and "hinges" developed rapidly at critical sections and made the arch unstable.

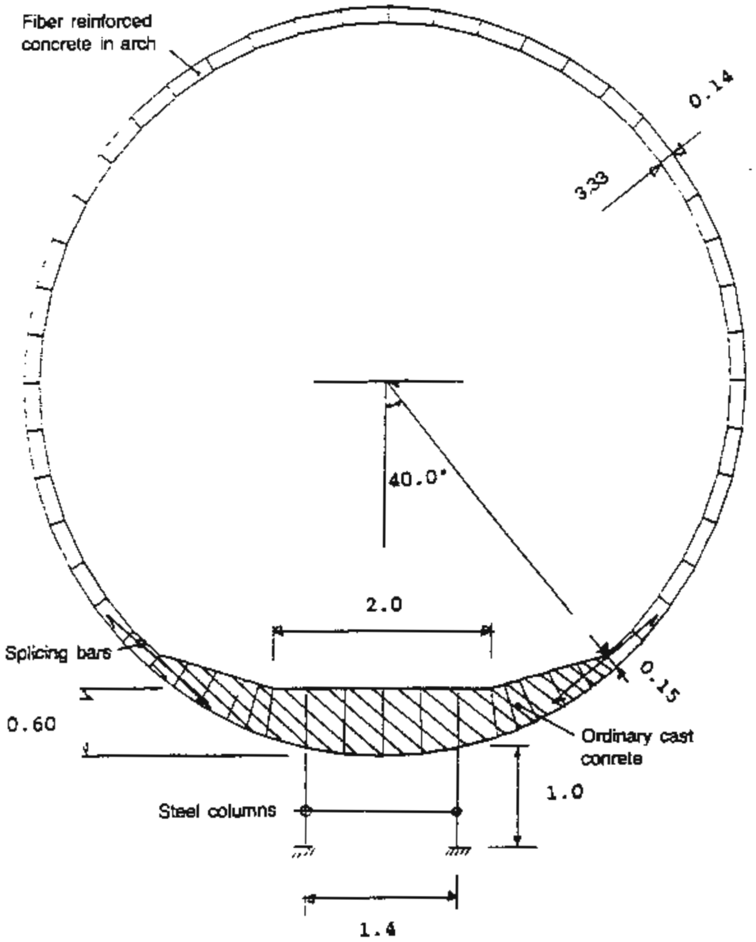

Fig 6 Typical section of concrete tube, from FE-analysis

Based on these test results it was decided to utilize only 1,0 MPa of the bending tensile strength in ULS design. The amount of prestressing tendons had to be increased accordingly.

4 CONSTRUCTION OF THE TUBE

During construction in Sevilla, some cracks developed at the "equator line" of the tube while the structure was still supported by scaffolding before the tendons were stressed. The cracks were at the highest stressed ares in the arch and the fibers were unable to re-distribute the cracks to a wider area. It was assumed that the cracks were caused by temperature gradients and shrinkage.

Considerable cracking also occurred in the fiber reinforced arch perpendicular to the longitudinal direction of the tube. The cracks occurred in all casting joints, which were positioned in 9 meters intervals. Again shrinkage was considered to cause the problems. High content of cement, early drying in high temperature and no coarse aggregate made the tube arch very sensitive to shrinkage. All cracks were satisfactory repaired by epoxy injection.

It was also difficult to control the thickness of the arch during shotcreteing. "Pins" with length equal to the arch thickness were cast into the concrete and used as "spies" for thickness control.

Care had also to be taken when concrete was sprayed in the upper part of the arch. Some concrete was then reflected from the face and fell down as "dry" material on the faces below. If this material was not removed it would be covered by concrete when the next layer was sprayed on the lower faces and would then form a soft, weak layer inside the material.

5 SUMMARY AND CONCLUSIONS

Based on the experience during design and construction of the fiber reinforced concrete tube in Sevilla the following conclusions and recommendations are made:

1. To utilize the tensile and bending strength of fiber reinforced concrete in design all "types" of deformations must be fully controlled, i.e. shrinkage, creep and temperature.

2. In addition to determine design strengths for bending, tension and compression, the ductility of the fiber concrete has to be considered. The ductility should preferably by so high that the tensile strength of the material is not reduces after cracking of the concrete matrix.

3. The production must be carefully planned to avoid weak and soft zones within the material, weak casting sections and to get full control of sectional thickness.

4. If ordinary reinforcement are placed in shotcrete sections, care must be taken to avoid "shadows" behind bars. Reduced bond between reinforcement and shotcrete must be expected.

5. The quality of the work performed in shotcrete concrete is strongly dependant on the skill of the man holding the spray. Therefore the documented experience and skill of this person is very important. The material coefficient used in design of concrete that is so dependant of the labor skill should be higher than the coefficient used for ordinary concrete work.

The concrete tube in Sevilla was successfully fabricated and performed without problems during its service period. A bold and original idea was realized through a close cooperation between the architects having the idea and the contractors doing the job.

6 REFERENCES

/1/ NS 3473; Concrete structures. Design rules. 4th Edition Nov. 1992.