TESTING OF HIGH STRENGTH LIGHTWEIGHT AGGREGATE CONCRETE ELEMENTS

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ABSTRACT

The present paper deals with an experimental investigation of structural elements of high strength lightweight aggregate (LWA)-concrete exposed to bending and shear and axial compression. In addition, fatigue tests of plain high strength LWA-concrete are reported.

The tests are a part of a test programme which aim to prove the applicability of high strength LWA-concrete (C>60) for deep water structures.

The results are compared with findings in corresponding investigations of normal density (ND) concrete of high and moderate strength.

Key words: High strength concrete, lightweight aggregate concrete, tests offshore structures

1. INTRODUCTION

Lightweight aggregate concrete (LWA-concrete) has been used for many years in several types of concrete structures. Development of high strength ligthweight aggregates combined with highly developed mixing techniques have made it possible to obtain LWA-concrete with cube strength of 60-65 MPa. This high strength LWA-concrete is interesting for use in offshore structures. Low weight, and thus increased towout capacity for large gravity platforms is one advantage that can be mentioned.

2. BEAM TESTS

2.1 Test specimens and test arrangements

The tests on 22 reinforced beams were carried out to examine the behaviour of high strength LWA-concrete subjected to bending and shear.

The beams that were tested are shown in Fig. 2. The main geometrical dimensions were 150x200x2850 mm (bxhxl).

Beams nos M1-M4 were designed and loaded to obtain a failure in bending and nos S5-S10 to obtain a shear failure. For beams nos M1, M2, M3 and M4 the reinforcement ratio was 1.6%, 2.5%, 5.4% and 5.4%, respectively. 10 types of beams were tested, 2 of each type, except for S9 where 4 beams were tested.

In addition to the 22 beams 100 mm cubes, Ø 150x300 mm cylinders and 100x100x500 mm beams were cast. These specimens were used to test compressive strength, modulus of elasticity and tensile strength, i.e. splitting strength and flexural strength.

Fig. 1 shows a sketch of the test-rig. The beams were loaded upwards. In this way the weight of the beam counteracted the applied forces. This effect has been taken into account. The horizontal distance between the supports at the ends and the jacks could be adjusted.

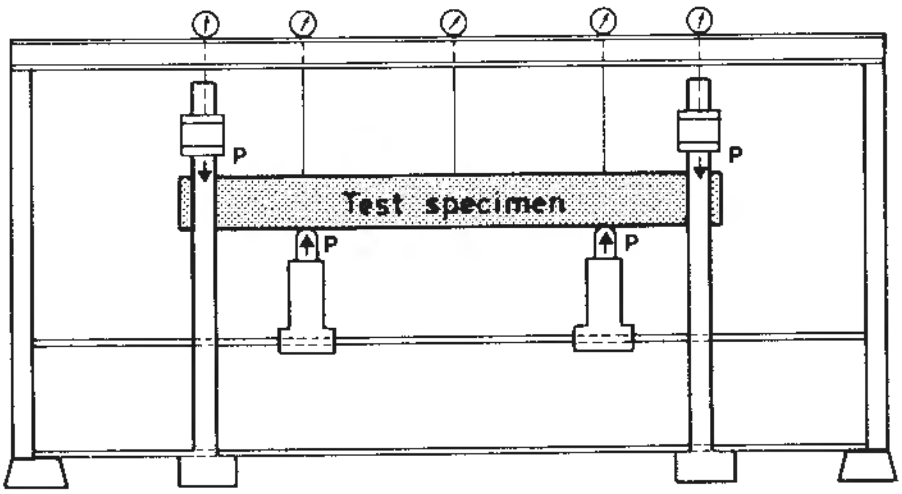


Fig. 1. Rig for beam tests.

During the tests the following parameters were measured:

- deflection of the midspan of the beam, cfr Fig. 1
- concrete compressive strain with two PL60 strain gauges on the concrete surface, cfr Fig. 3
- tensile strain in the reinforcement with one PL10 strain gauge on each reinforcing bar, cfr Fig. 3
- strain in the midspan of the beam. These strains were measured with extensometer over a 200 mm length at 14 points around the circumference of the cross-section of the beam, cfr Fig. 3

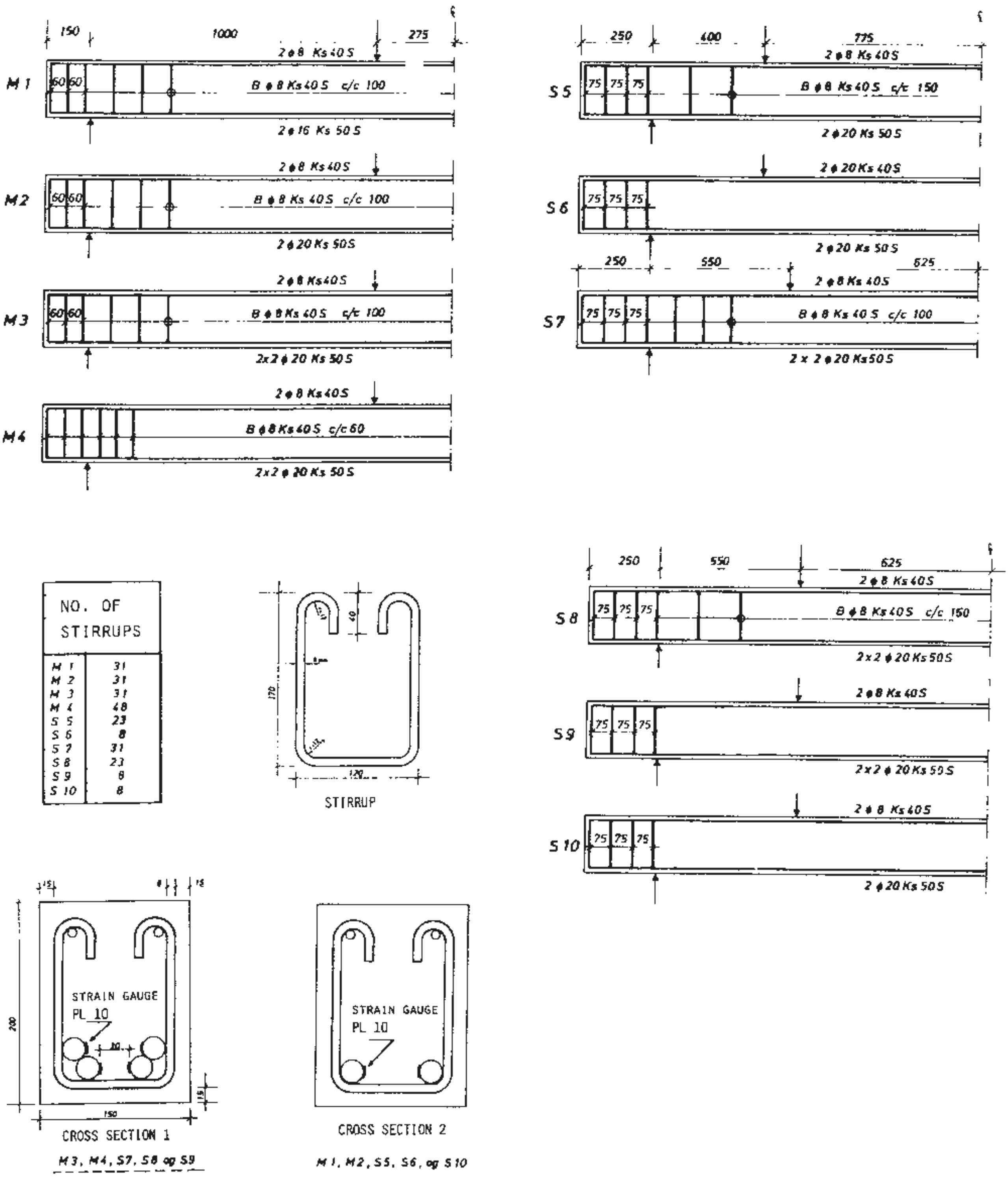
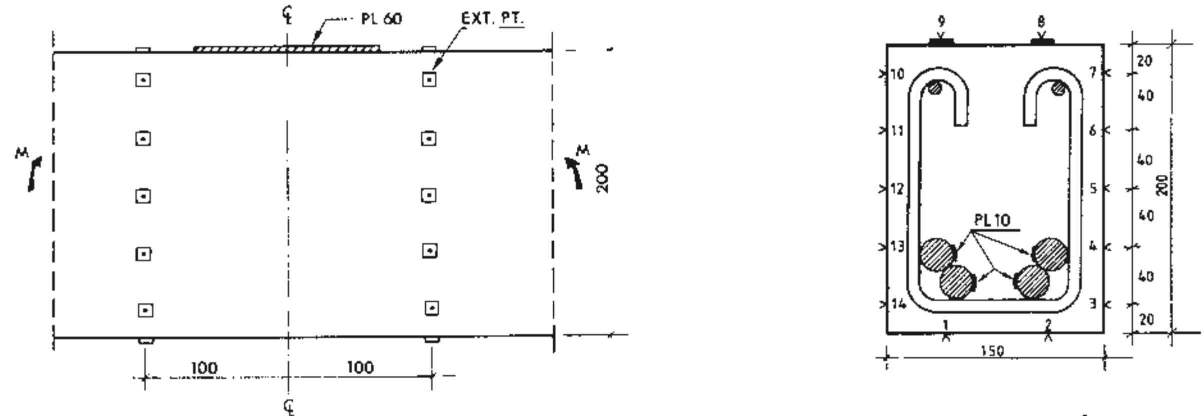


Fig. 2. Test beams.

Table 1.

Batch no.	Age days	Cube strength MPa	Cylinder strength MPa	E-modulus MPa	ε_c at max.stress o/oo	Splitting tensile strength MPa	Flexural tensile strength MPa
1	36	76,3	69,9	24 967	3.19	3,50	4,19
	41	80,8				3,90	
2	31	81,0					
	32	79,2				3,73	
	33	77,5	68,5	25 101	3,36		3,20
3	31	74,6					
	35	77,0					
	36	76,3	70,8	26 430	3,07	3,45	3,59
4	37	71,5					
	38	76,6	67,3	25 509	3,09	4,03	3,72
	41	74,2					
	Mean:	76,8	69,1	25 501	3,18	3,72	3,68
	St.dev.	3,7 %	2,2 %	2,6 %	4,2 %	6,7 %	11,1 %

The use of mean values and standard deviation are not theoretically correct since the mechanical properties are not measured at the same age.



Longitudinal section. Midspan

Cross section
Midspan

Fig. 3. Instrumentation of midspam of beams. Strain gauges and extensometer points.

The strain registration from the strain gauges were carried out by a Brüel & Kjær amplifier and printed onto paper.

2.2 Mechanical properties

Liapor was used as aggregate. The mean density of the LWA-concrete was 1916 kg/m.

The results from testing of cubes and cylinders are shown in Table 1. The beams were casted in four batches and these batches are numbered 1-4 in Table 1.

It is interesting to note the ratio between cylinder strength and cube strength that was a mean value of 0.90 (mean).

It is important to notice that all cubes and cylinders were stored together with the beams under wet sacks and polyethylene foil. As can be seen from Table 1 the age at testing varies between the specimens. This is because the specimens were tested at the same age as the beams. All test results refer to mean value of these specimens.

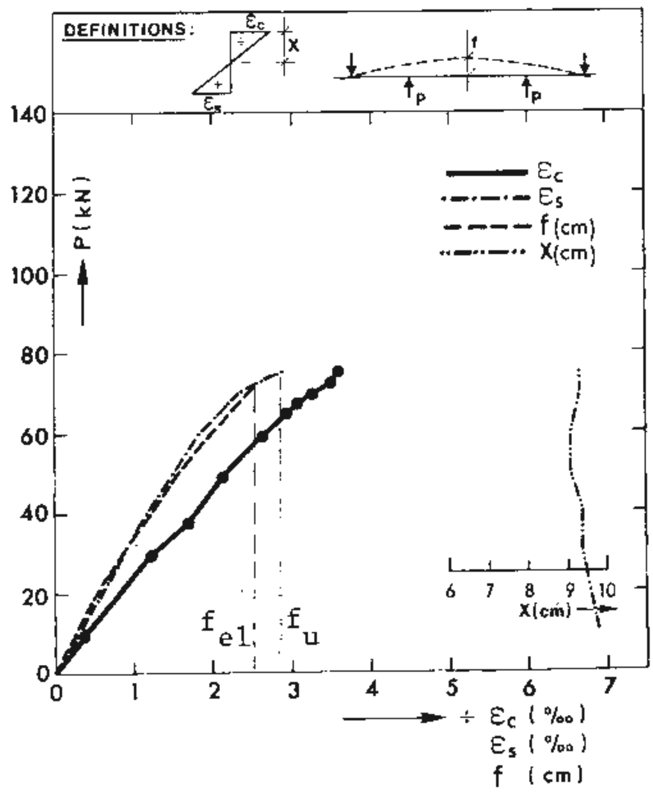
Test results from waterstored 100 mm cubes are shown in Table 2.

Table 2. Waterstored cubes.

Batch no	Cube strength, MPa	
	7 days	28 days
1	60,4	72,7
2	56,0	72,6
3	60,2	69,4
4	55,1	67,9

2.3 Beam tests

The results from tests were illustrated as shown in Fig. 4 for beam no M4-A. This beam was over-reinforced and the failure was characterized by crushing of the concrete in the compression zone.



Beam no: M4-A
Ultimate load: P_u = 75.2 kN

ε_c = concrete compressive strain
ε_s = steel tensile strain
f = deflection at midspan
x = height of compression zone

Fig. 4. Test results beam M4-A.

The detailed test results from the 22 beams will not be presented here because of the limited space. However, some information on the main items will be presented.

2.3.1 Cracks

The flexural tensile strength and shear strength were calculated from the tests by noting at which load the different types of cracks occurred.

The calculated values of the flexural tensile strength, σ_{cr}, had a mean value of 4.4 MPa. This was about 20% higher than the mean σ_{cr} measured on 100x100x500 mm beams. However, the first cracks in the beams will usually occur at a lower load level than when observed,and it seems probable that the small unreinforced beams are more sensitive to "drying-out-effects" than the larger reinforced beams.

The shear stresses, τ_{scr}, were calculated on the basis of a parabolic distribution over the cross-section. The calculated mean value of τ_{scr} were 2.2 MPa.

2.3.2 Strains

The ultimate concrete compressive edge strain, ε_{cu}, was measured to 3.61 o/oo (mean) for the over-reinforced beams and 3.45 o/oo (mean) for the under-reinforced beams.

The maximum compressive strain measured on Ø 150x300 mm cylinders was 3.18 o/oo for centric load.

The latter value is rather high compared to NW-concrete of the same grade but the E-modulus of LWA-concrete is lower than for a corresponding NW-concrete.

The stress-strain diagrams for the LWA-concrete measured on cylinders were linear with a small curvature short before failure. This behaviour is similar to the behaviour of high strength NW-concrete except from the previously mentioned lower E-modulus of LWA-concrete.

The measured ε_{cu} from the beam tests shows that a compressive edge strain of 3.5 o/oo as used for design for NW-concrete also should be applicable for LWA-concrete of the strength used in these tests.

2.3.3 Ultimate limit state. Combined actions of flexural moment and shear

To present the results from the beam tests in a "condensed" manner it was chosen to look at the combined actions of flexural moment and shear and compare the LWA-concrete with other relevant test series.

The ratios M_u/M_d and V_u/V_d were calculated for all 22 beams. M_u is the bending moment measured at the failure of the beam in disregard of type of failure, and V_u has to be understood in the same way. M_d and V_d are calculated design moment and shear capacity according to the Norwegian Code (NS3473) /10/. The design concrete stress, f_c, is found by linear extrapolation according to the formula:

$$f_c = \frac{1}{\gamma_m} (0,4f_{cj} + 6)\text{MPa} \quad , \quad \gamma_m = 1.25.$$

It is emphasized that f_{cj} is the <u>measured</u> mean value of compression strength from cubes stored together with the beams, ie <u>not</u> waterstored. Effects caused by differences between characteristic cube strength, f_{ck}, (referring to a certain grade of concrete, eg C60 or C65) and the used f_{cj}, has not been discussed.

The shear strength values, f_v, for shear calculations have also been extrapolated from NS3473 on basis of f_{cj}. The results are plotted on Fig. 5.

Fig. 5 shows that the results from the tests on the LWA-concrete are very similar to the results from other tests.

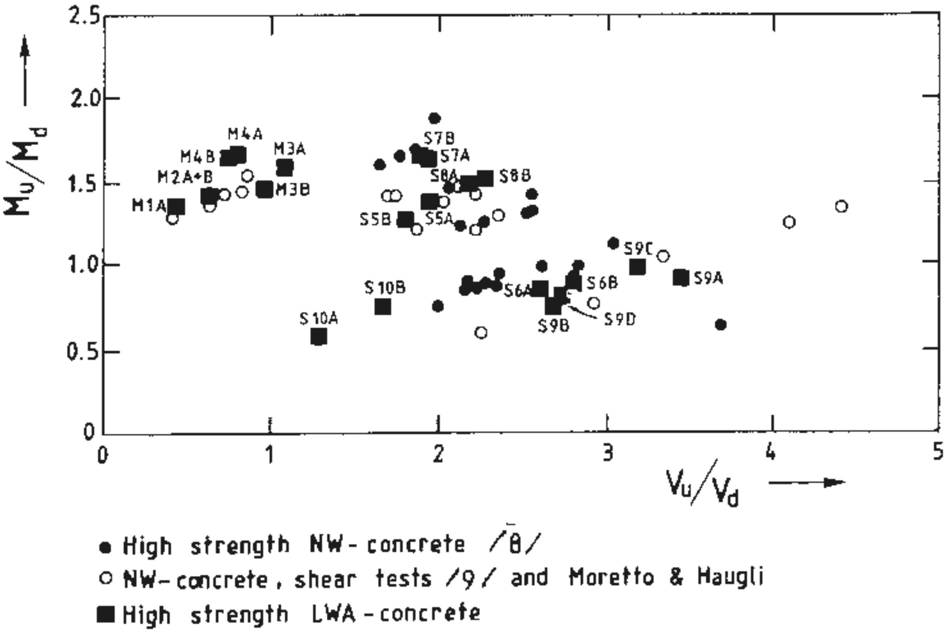


Fig. 5. Combined actions from moment and shear.

Beams nos S10A and S10B have a considerably lower V_u/V_d-ratio than the other beams without stirrups. All the other beams including those of NW-concrete are gathered outside the area defined by the lines $V_u/V_d = 1.8$ and $M_u/M_d = 1.25$. If we draw the lines $V_u/V_d = 1.25$ and $M_u/M_d = 1.25$ all the results from the LWA-concrete beams will be outside this area, which means they satisfy NS3473 with $\gamma_m = 1.25$. However, S10A and S10B are not designed according to codes like NS3473 /10/ and CEB /11/ because these codes, and others, require a minimum shear reinforcement.

It must be noticed that the performed tests are short-time tests. Effects caused by long time loads (creep) would normally lead to a lower failure load.

The results from S10A and S10B show that the limitation of effective longitudinal reinforcement in NS3473, 5.2.3: $V_d = f_v(bd + 75\ A_s) \leq 2\ f_v bd$ is justifiable. Because if A_s is used with its actual value, the ratio V_u/V_d would become less than 1.25 (γ_m). S10A and S10B are the only beams with A_s = 628 mm^2 and a/h = 550 /200. The other beams with this shear span had A_s = 1256 mm^2. None of the NW-beams without stirrups had a ratio of reinforcement as low as S10A and S10B, therefore it is difficult to compare these two beams directly with the other beams in Fig. 5. However, the low ultimate load for S10A and S10B might be an indication for reducing the design shear stress for LWA-concrete compared to NW-concrete of the same grade.

Regarding the over-reinforced beams, the test results from the LWA-concrete showed a mean value of M_u/M_d = 1.60, st.dev. 5.2%. Similar values from other tests with NW-concrete beams showed in Table 3.

Table 3. Results from other tests.

Number of beams	Grade (mean)	Ratio M_u/M_d	St.dev. %
14	~C15	1,73	12,5
29	~C20	1,65	8,5
25	~C25	1,54	6.2
15	~C30	1,59	6,0
15	~C35	1,54	8,5
6	~C43	1,51	7,4
2	~C90-95	1,49	-

For NW-concrete it seems like M_u/M_d is decreasing with increasing compressive strength. Because of the limited number of over-reinforced LWA-beams it is difficult to draw conclusions but the results indicate that the strain/stress connection in the compressive zone for LWA-concrete beams may be described in a better way than the parabolic/linear relation used in NS3473. A more triangular stress block at ultimate conditions would be more in line with the stress-strain diagram measured on LWA-concrete cylinders.

2.3.4 Ductility

The ductility of the LWA-beams that failed in flexure was compared with test results from beams of high strength NW-concrete (f_{ck} ~ 90-100 MPa).

The ductility was calculated as the ratio between the deflection of the midspan of the beams at failure and at the final point of linear elastic behaviour, see Fig. 4, f_u and f_{el}, respectively. This ratio f_u/f_{el} is called "ductility factor". The results are shown in Fig. 6.

ρ/ρ_b is the ratio between the actual reinforcement ratio, ρ, and the reinforcement ratio at balanced reinforcement, ρ_b. ρ_b depends on the basis for calculation which here were a triangular stress block with edge stress 0.85 f_{cj} and edge strain 3 o/oo.

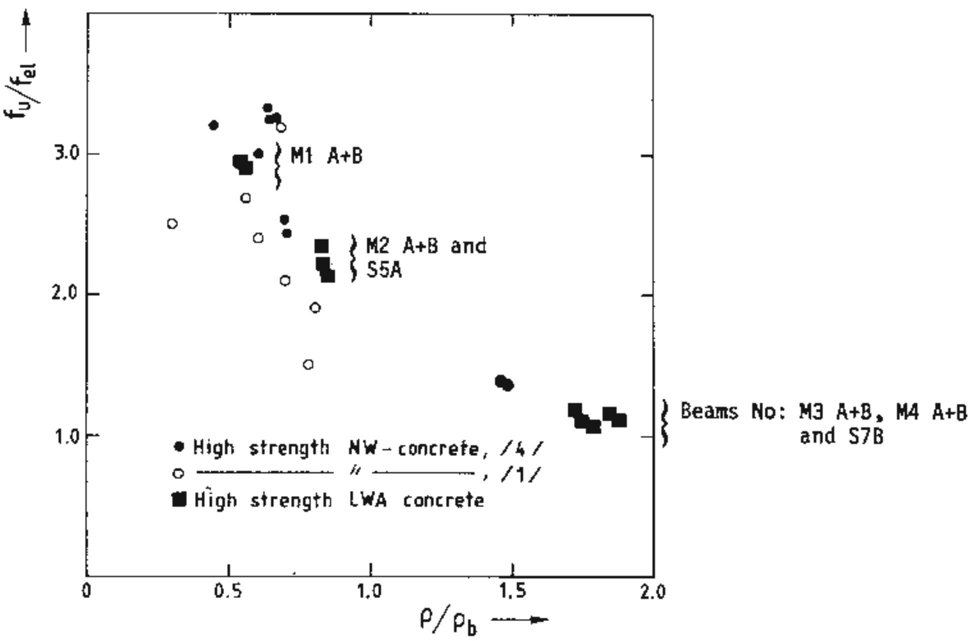


Fig. 6. "Ductility index".

2.3 Conclusions and summary of beam tests

- The relationship between ultimate moments, M_u, from the tests on the LWA-beams and the moment capacity according to NS3473, M_d, is of the same order as the beams of NW-concrete. The M_d-values are calculated on the basis of linear extrapolation of the design values in NS3473.

- The ultimate shear loads, V_u, show a rather large scatter for beams without transverse reinforcement. This scatter is due to the effect of a favourable crack development that might lead to the formation of compressive arches with large capacity in the shear zone. This effect is difficult to predict.

  A discussion of moment and shear as a combined action indicates that the high strength LWA-concrete acts as normal weight concrete when the ultimate load from the tests, V_u, is compared to the ultimate shear capacity calculated according to NS3473.

  However, the shear tensile strength from these tests is lower for the LWA-concrete than for NW-concrete of the same grade.

- Calculations of the ultimate moments based on ε_{cmax} = 3.5 o/oo, σ_{cmax} = 0.9 f_{cj} and a triangular stress block give a good approach to the M_u from the tests. (ε_c = 3 o/oo and σ_c = 0.85 f_{cj} also give satisfactory results).

- For beams that had a flexure type of failure the ultimate concrete compressive strain, ε_{cu}, was 3.61 o/oo (mean) with st.dev. 8.1% for over-reinforced beams. For under-reinforced beams the numbers were ε_{cu} = 3.45 o/oo (mean) with st.dev. 12.5%.

- The "ductility index", defined as the ratio f_u/f_{el}, decreases with increasing amount of reinforcement and is similar to the ductility index for NW-concrete.

3. TESTING OF COLUMNS

The aim of the column tests was to examine LWA-concrete elements subjected to axial compression. Properties of particular interests were: bearing capacity, load/strain relationship and failure modes.

The specimens and test performance were chosen in accordance with previous investigations on normal-density concrete (ND) of high and moderat strength, which also served as a basis of reference, /2/ and /3/.

3.1 Test spesimens and test arrangement

The test series consisted of six columns with dimensions 250x250 x2000 mm, shown in Fig. 7.

The columns were denoted S1, S2,..,S6 depending on reinforcement ratio and grade of reinforcement, see Table 4.

Table 4. Reinforcement of columns.

COLUMN	REINFORCEMENT	YIELD STRESS (MPa)	BATCH
S1	0		TT 74
S2	0		TT 88
S3	8Ø16 (2.57%)	400	TT 74
S4	12Ø16 (3.86%)	400	TT 74
S5	8Ø16 (2.57%)	600	TT 74
S6	12Ø16 (3.86%)	600	TT 74

The concrete mixture gave a mean compressive cube strength of approximately 72 MPa after 28 days of curing. The modulus of

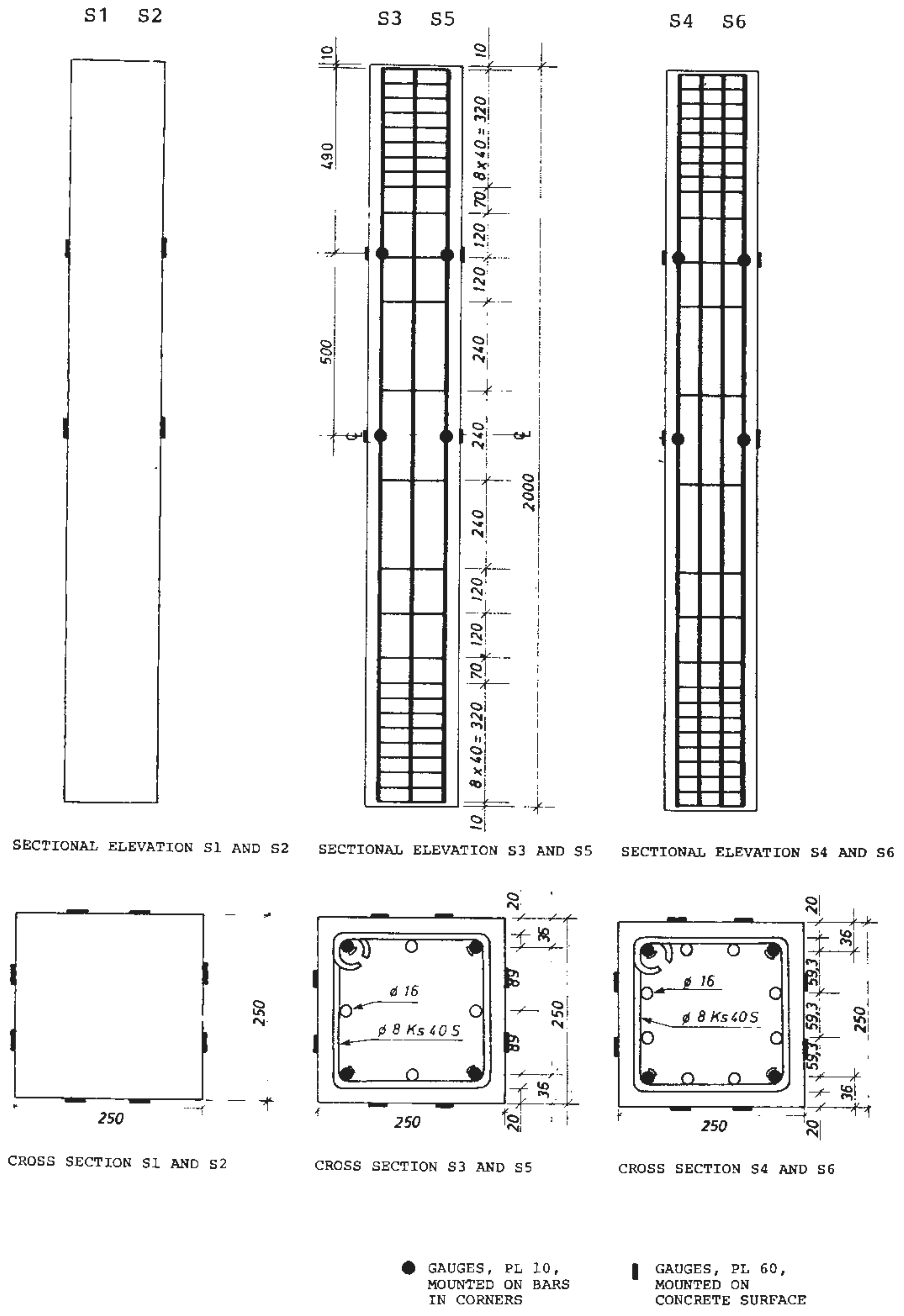


Fig. 7. Columns.

elasticity, measured on 150x300 mm cylinders, was about 25 000 MPa. The stress/strain relationship showed a fairly linear behaviour up to failure.

The test columns and corresponding material test specimens as cubes 100x100 mm and cylinders 150x300 mm were all stored under wet cloth until the day of test preparation.

The reinforcement used, Table 4, was of grade Ks40 and Ks60 (yield strength of 400 and 600 MPa respectively). Grade 60 was used particularly to examine the application of a higher steel quality, that is a higher yield strain, in LWA-members.

The amount of reinforcement exceeded what is recommended as a minimum according to the Norwegian Code NS3473 sec. 9.4.1 (ρ_{min} = 1.1 - 1.7%).

To prevent failure caused by transverse forces in the columns during testing, stirrups ∅ 8 mm Ks40 were mounted in both ends, regarding S3-S6.

The tension/strain relationship for the longitudinal reinforcement was tested on 16 mm bars. The tests revealed a rather unlinear behaviour for Ks40 in what should be the elastic region. The yield stresses were somewhat greater than the prescribed grades Ks40 and Ks60 should promise.

The test set up is shown in Fig. 8. A hydraulic spindle press of type Losenhausen with a capacity of 5000 kN was used. The loading machine is operated manually and registration of loads were done by oil manometer readings and by means of oil pressure transducers.

As a matter of security, the test rig was covered with wooden plates during testing.

Steel collars treated with epoxy were mounted on the ends of the test specimens in order to prevent splitting damage of the ends. To achieve a good distribution of the axial forces on the ends, porous wooden plates served as interface to the column ends.

The test columns were instrumented with electrical strain gauges in two sections in the distance of 500 and 1000 mm from top of the columns as shown in Fig. 7. Four reinforcing bars were instrumented in each section and additional strain gauges were mounted at the concrete surface in the same section.

The datalogging was achieved using a Solartron 3530 Orion datalogging system. The load and strain recordings were printed directly and simultaneously stored on magnetic tape. This made it possible later on to transmit data to a NORD 500 computer for handling and plotting of results.

The loads were attached in steps, by percent of expected load at failure, see Table 5.

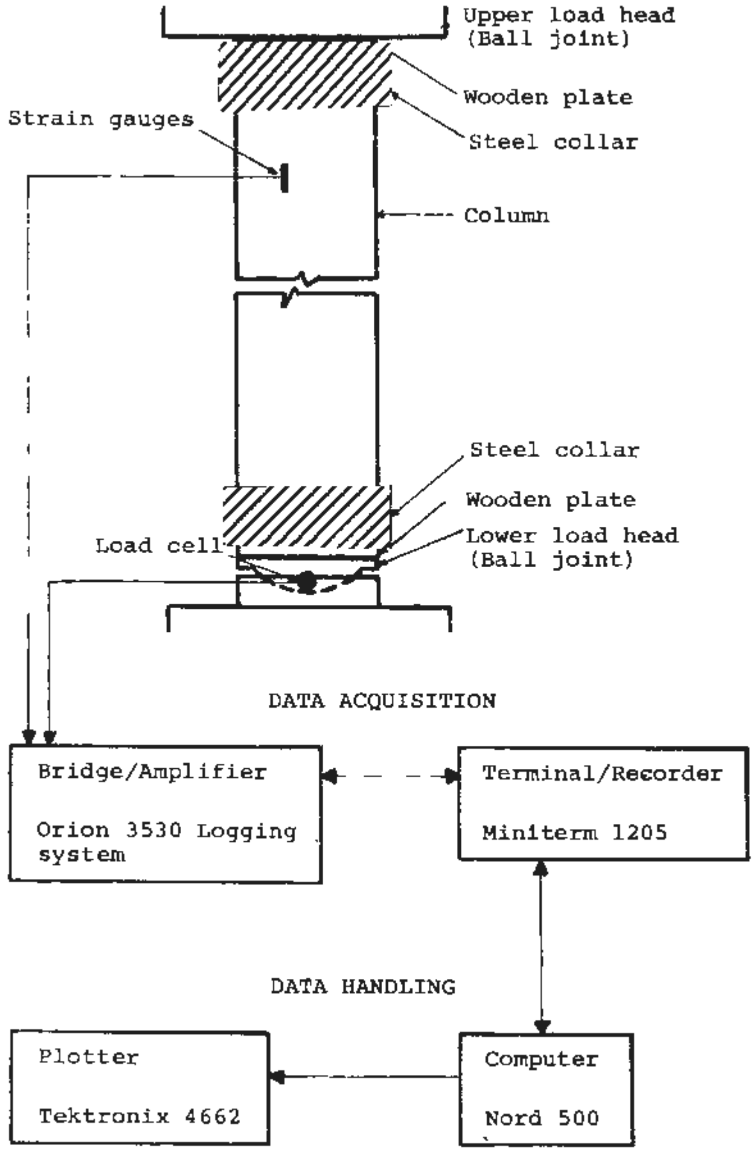


Fig. 8. Test set up.

The expected failure loads were estimated according to

$$P_{ef} = 0.8\,(A_c - A_s)f_{cyl} + A_s \cdot f_y$$

where A_c, A_s = area of concrete and reinforcement respectively

f_{cyl}, f_y = concrete cylinder strength and yield strength of reinforcement

When reaching a load level, the load was held constant for about 2 - 2.5 minutes. Thereby a load sequence from zero to failure load lasted about 45 minutes.

Before the columns were tested, a load of 30% was attached to judge the centricity.

3.2 Test results

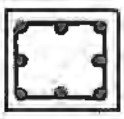
3.2.1 Failure loads

The failure loads are given in Table 6 in addition to the last recorded strains on gauges mounted on the concrete surface and the corresponding applied loads.

Table 5. Load sequence.

LOADLEVEL	1	2	3	4	5	6	7	8	9	10	11	...
% of expected failure load	0	30	50	75	80	85	87.5	90	92.5	95	97.5	...

Table 6. Failure loads.

	COLUMN	FAILURE LOAD (kN)	FAILURE STRAIN (0/00) i)
	S1 S2	4060 4160	3.07 (4000) 3.25 (4160)
	S3 S5	4345 4830	2.72 (4275) 2.94 (4650)
	S4 S6	4925 5085	3.30 (4925) 2.71 (5085)

i) Mean value of last measured strains on gauges mounted on concrete surface, applied load in parenthesis

3.2.2 Load/strain relationship

Figs. 9 and 10 give typically load/strain relationship for unreinforced (S1) and reinforced (S4) columns.

The strains are taken from the upper instrumented section. The gauges that gave the extreme upper and lower strains are shown as dotted lines and gives an indication of the unavoidable strain gradients that occurred under loading.

Fig. 10 shows that there was a good compatibility between concrete and reinforcement.

3.2.3 Failure modes

The columns failed without pressage in a brittle manner, particularly the unreinforced S1 and S2. Figs. 11 and 12 show typically S1 and S4 after failure.

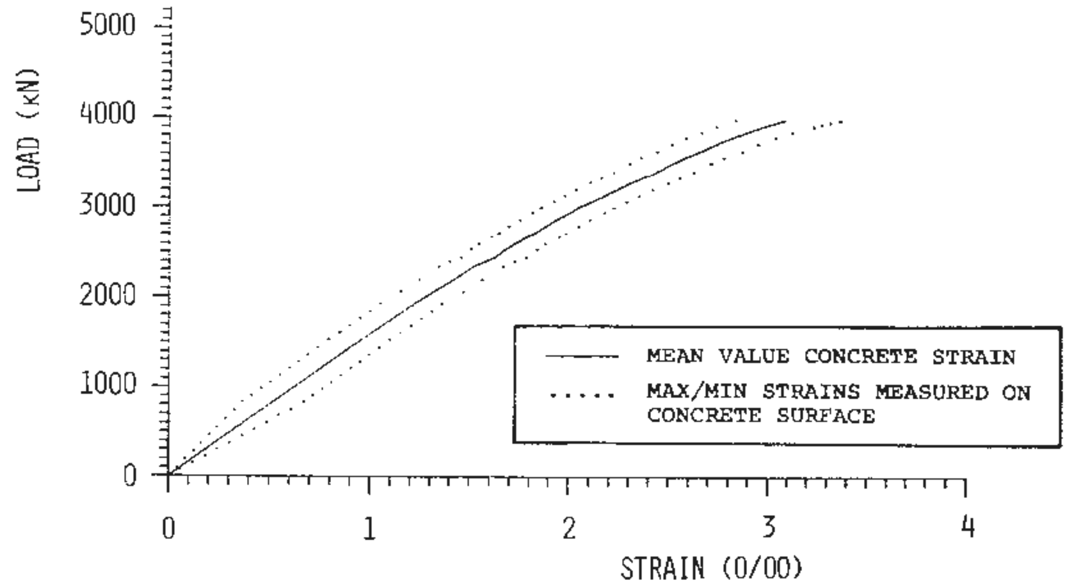


Fig. 9. Load/strain relationship for columns S1, unreinforced.

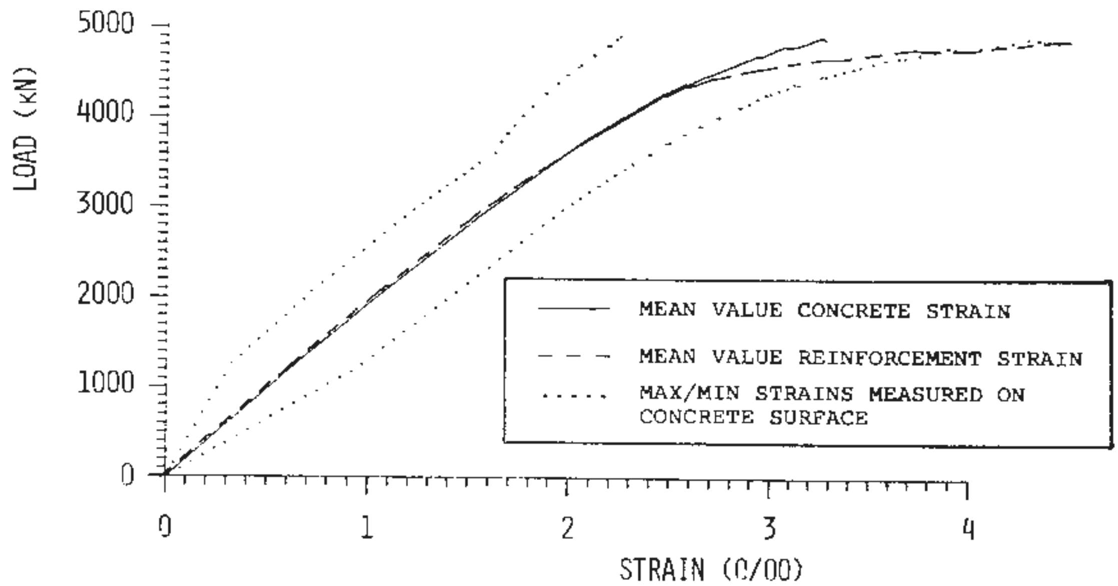


Fig. 10. Load/strain diagram column S4, 12Ø16 Ks40.

3.3 Discussions and conclusions of column tests

3.3.1 Failure strength and capacity of columns

Fig. 13 shows the relationship between registered and calculated bearing capacity. The calculations are done in accordance with the Norwegian Code NS3473, sec. 5.1.3, which says:

$$N = \frac{0.85}{\gamma_m} (f_{cn}A_c + f_yA_s) \quad , \quad \gamma_m = 1.25$$

where: f_{cn}, the nominal design strength is calculated according to the formula:

$$f_{cn} = 0.4\ f_{ck} + 6\ \text{MPa}$$

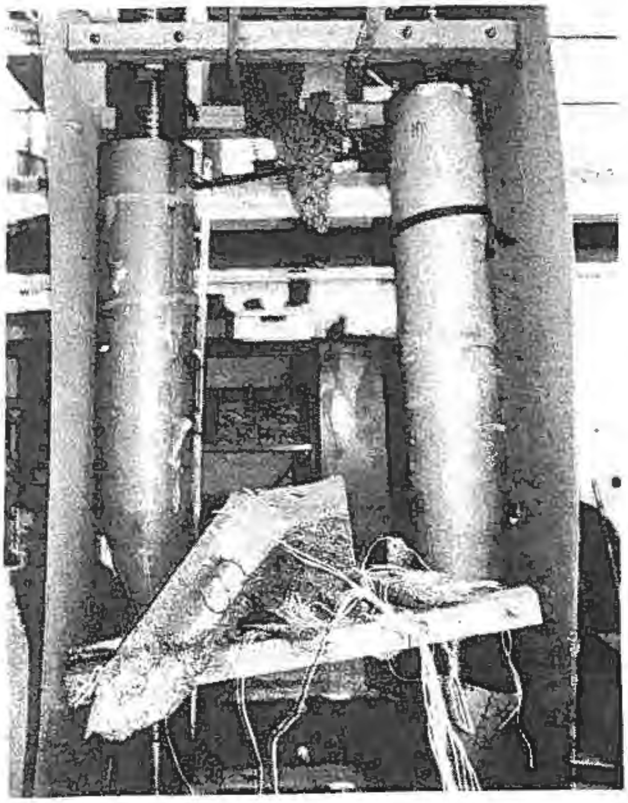


Fig. 11. Column S1, unreinforced.

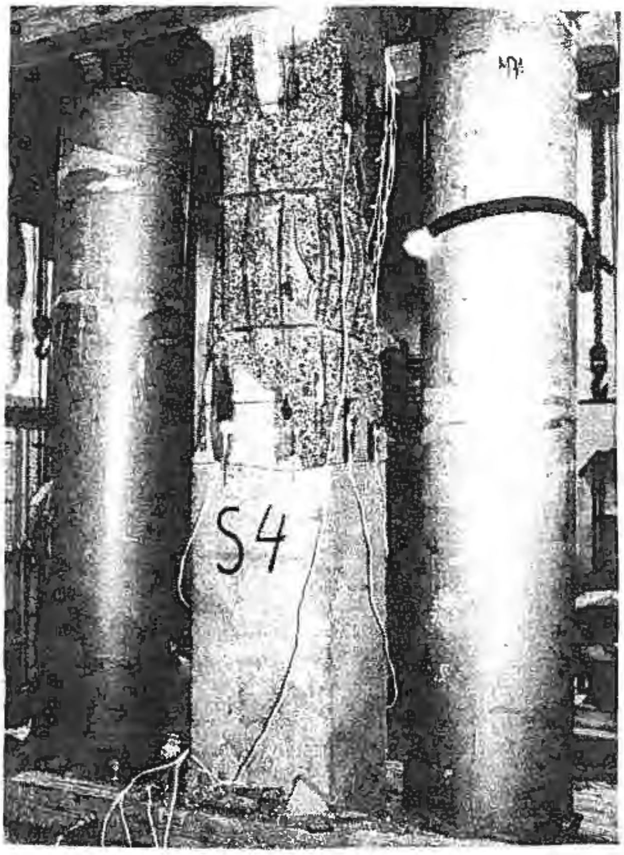


Fig. 12. Column S4, 12Ø16 Ks40.

The figure also compares the results with previous findings /2/ and /3/ on ND-concrete of different grades.

It is noticed that the LWA-columns generally give a higher degree of safety, P_{obs}/P_{NS3473}. The values for LWA-concrete concerning different reinforcement ratios and grades fit into the same pattern that was found in /2/ and /3/. That is, the degree of safety is inverse proportional with reinforcement ratio and grade of reinforcement. Even though the pattern is recognized, the figure shows that the reinforced LWA-columns gave a degree of security approximately 40% above what one could expect.

3.3.2 Failure strains

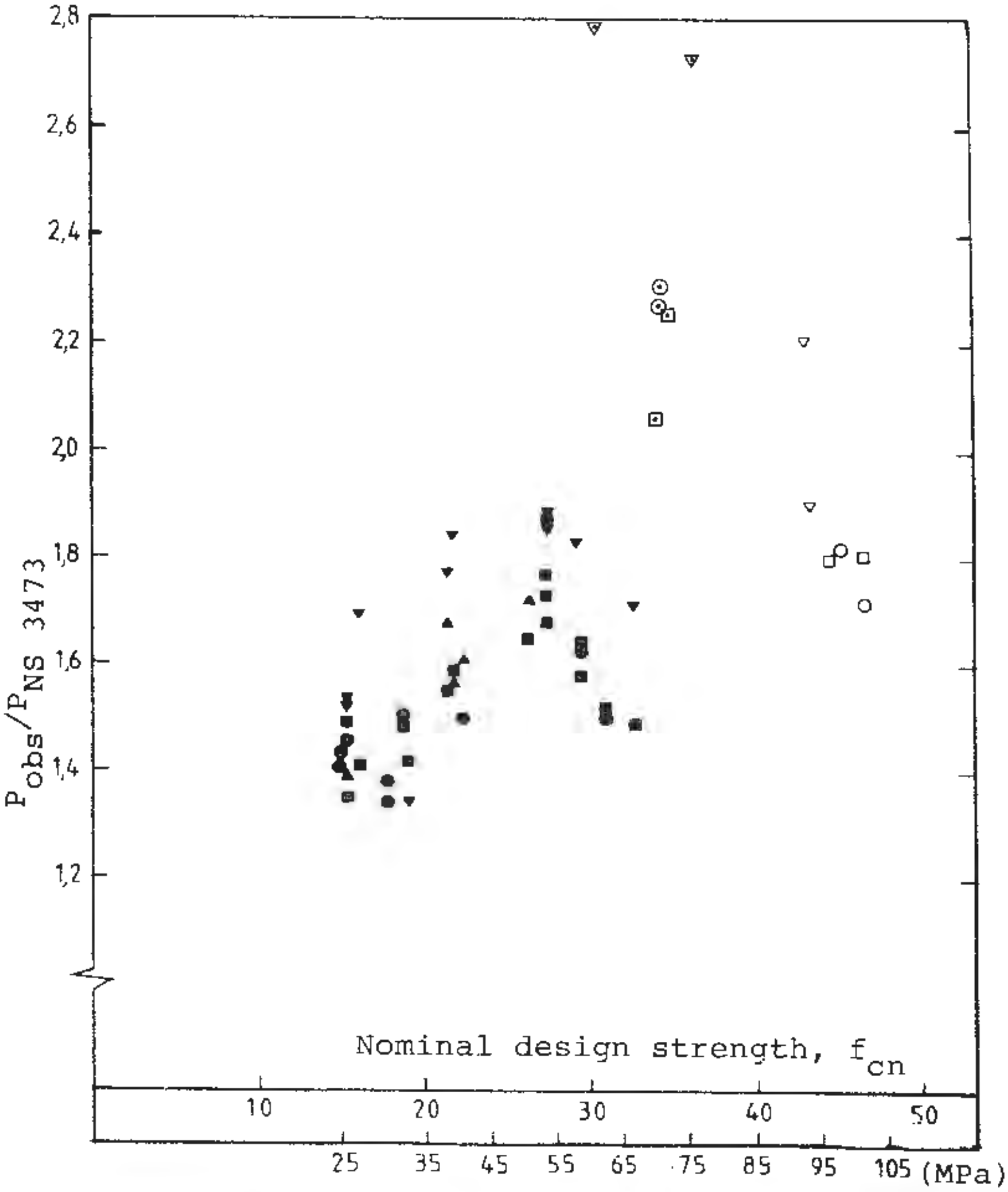
The LWA-concrete modulus of elasticity had a low value compared with its strength, and hence, because of the fairly linear stress/strain diagram high failure strains ≈ 2.95 o/oo. Thus one found good compatibility between concrete and reinforcement. That is, the high obtainable strains in the concrete before failure made it possible to get "full" utilization of the reinforcement strength. This also was the case for Ks60, which has a yield strain equal to 2.86 o/oo.

Therefore, the test results confirmed the validity of the principle of superposition. Prediction of the total bearing strength can be done by assuming that the concrete's and reinforcement's contributions can be added, or superimposed.

3.3.3 Strength, dependency on size

It is a well known fact that the bearing strength is dependent on size, the larger size, the less ultimate or failure stress.

As seen in Table 7 the cylinder/cube strength relationship for the LWA-concrete tested was found to be approximately 0.90, while /2/ and /3/ for ND-concrete gave 0.76 - 0.80. The same tendency is shown between columns and cubes, that is, a relatively less size dependent failure strength for the LWA-concrete.



Reinforcement	ND-conc /2/	ND-conc /3/	LWA-conc
Unreinf	▼	▽	▽ (with dot)
Ks40	●	○	⊙
Ks50	▲		
Ks60	■	□	⊡

Fig. 13. Relationship between failure loads and loads calculated in accordance with NS3473.

Table 7. Size dependency.

	f_{cyl} / f_{cube}	f_u^* / f_{cube}
LWA-concrete	0.90	0.83
ND-conc /1/	0.76	0.62
ND-conc /2/	0.80	0.63

* f_u = ultimate stress columns

Fig. 14 is an attempt of visualizing the size dependency for LWA-concrete compared with /1/ and /2/.

The different sizes of test specimens, cube, cylinder and column are represented along the abscissa with its equivalent diameter. The equivalent diameter is, for cylinders equal with its real diameter, for cubes and columns the diameter that appears when converting a quadratic to a circular cross-section keeping the sectional area constant.

For a structural element with the size of the tested columns a difference in "utilized" strength of about 20% is noticed.

Ref. /4/ points out that loss in strength in addition to being dependent on size also relies on Young modulus, tension strength and energy of fracture. Apart from this it may be possible that the aggregates somewhat particular shape has a favourable effect on the failure strength.

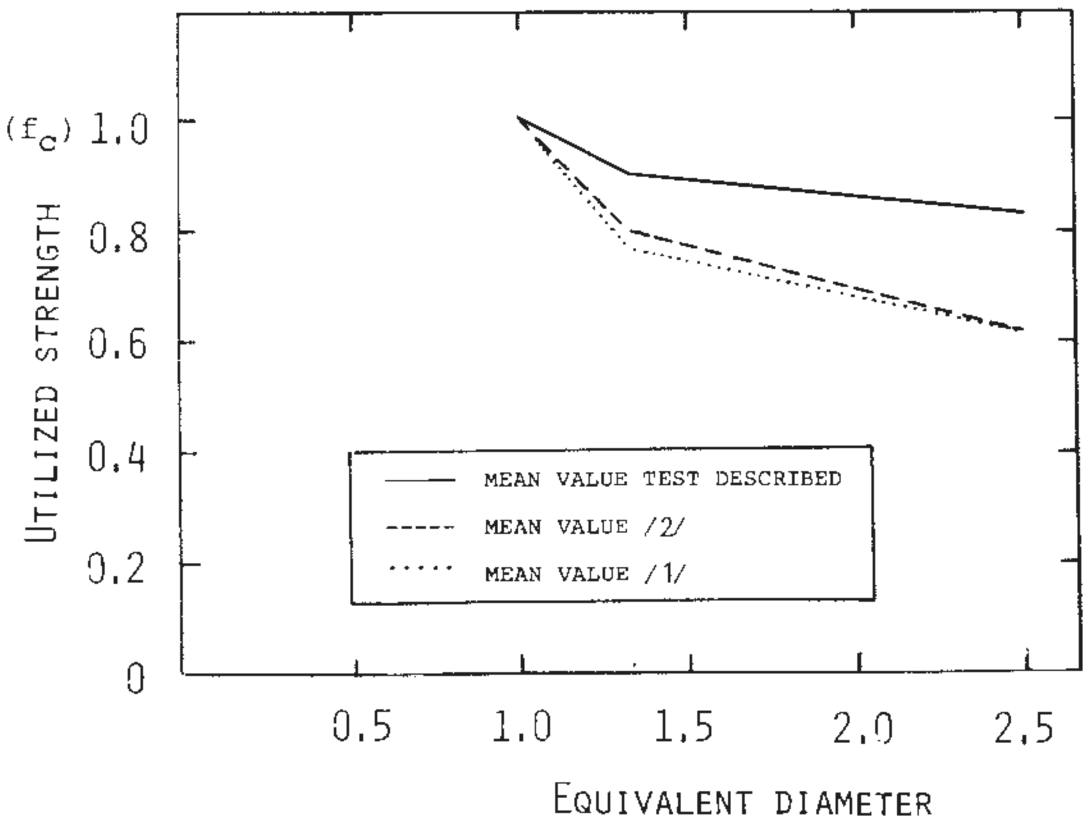


Fig. 14. Strength, dependency on size.

3.3.4 Conclusions

The results showed a concrete bearing strength in columns greater than what was found in previous tests /2/ and /3/ on ND-concrete. The load bearing capacity did far exceed the values calculated according to NS3473, especially regarding the unreinforced columns.

The LWA-concrete's low value regarding modulus of elasticity combined with its high strength, leads to a high failure strain. This concerns the utilization of the reinforcement's strength and is a favourable property.

The concrete failure strain was greater than the theoretical yield strain both for Ks40 and Ks60. Hence the measured strains and corresponding loads showed that superposition of the concrete's and reinforcement's contribution to the load bearing capacity may be done for both grades. Although it was noticed that Ks40 because of its lower yield strain gave a more ductile effect on high loads.

The results indicate a particular degree of "utilizable" strength, especially if related to cube strength, but also compared to cylinder strength. The bearing strength of columns was approximately 20% higher than what was found in previous tests /2/ and /3/ on ND-concrete, when related to cube strength.

In large structures exposed to high loads of compression, in cases where low density is favourable, this represents a corresponding decrease in weight. That is if one neglects possible non-linear problems.

Anyhow, the present results confirm that NS3473 may be used for calculating the bearing capacity of columns exposed to compressive loads. Though it is noticed that a higher rate of utility seems possible. To give practical recommendations for this statement calls for more work being done to elaborate the understanding of the fracture mechanism. The great bearing strength discovered may be due to properties as failure energy, tensile strength, the aggregates somewhat particular shape and the relationship between the matrix and the aggregates modulus of elasticity.

4. FATIGUE TESTS

4.1 Test specimens and test arrangements

The fatigue tests were carried out in order to identify special problems in fatigue material behaviour and failure mechanism of high strength LWA-concrete at repeated loading. Totally 26 cylinders (Ø 100x250 mm) were subjected to fatigue loading at different load levels. In addition, cylinders and cubes were tested statically during the fatigue testing programme in order to serve as static reference parameters. All the cylinders were drilled out from a LWA-concrete plate and the cubes were casted in standard forms (100x100x100 mm).

The test specimens were water-cured for 28 days at a temperature of 20°C after casting. Then they were placed in a room with constant humidity of about 70% and a temperature of 20°C. The fatigue testing period was 48-74 days after casting.

15 of the cylinders which were subjected to fatigue loading, were instrumented with 2 strain gauges (PL120) in the longitudinal direction (ε_L) and 2 strain gauges (PL60) in the transverse direction (ε_T) as shown in Fig. 16.

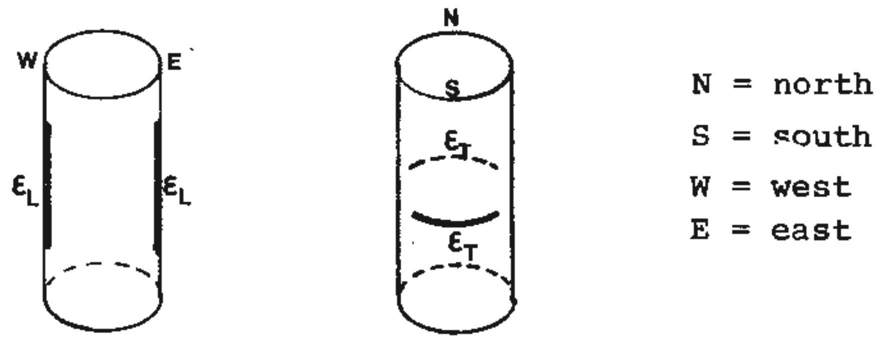


Fig. 15. Cylinders with fixed strain gauges.

The strain registrations during the fatigue tests were carried out by a Brüel & Kjær amplifier and printed onto paper. A schematic layout of the loading and registration equipment is shown in Fig. 16. All the fatigue tests were carried out with sinusoidal loading.

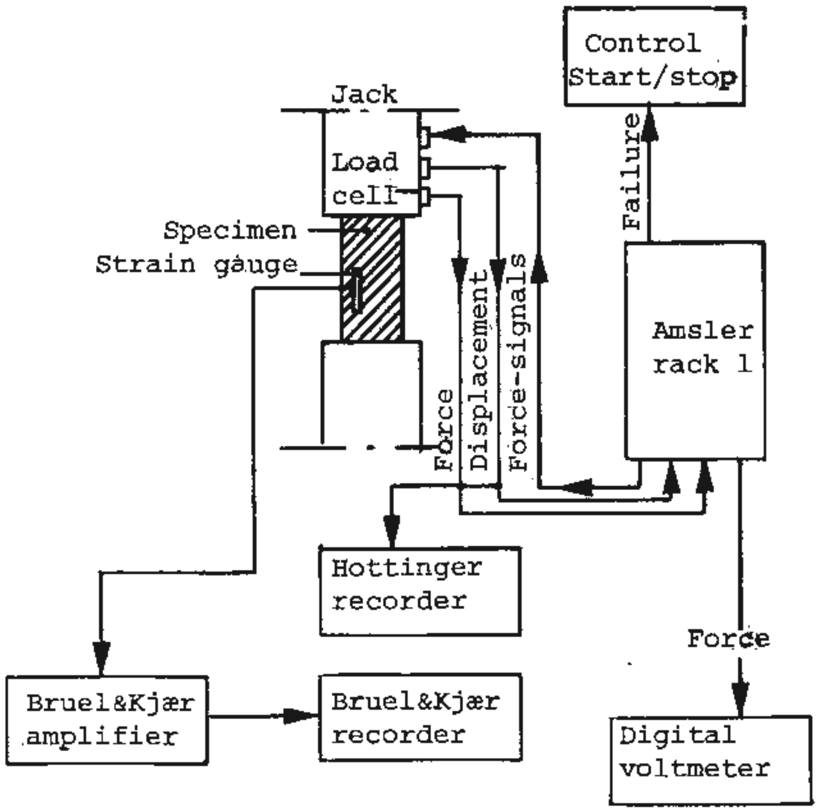


Fig. 16. Schematic layout of the loading and registration equipment.

4.2 Test programme

The test programme consisted of static and dynamic tests. The test programme for the fatigue tests consisted of 5 series as shown in Table 8.

Table 8. Fatigue test programme for cylinders (Ø 100x250 mm).

TEST SERIES	NUMBER OF SPECIMENS	LOAD LEVEL S_{max} / S_{min}
D 1	5	0.80/0.05
D 2	6	0.60-0.65/0.05
D 3	5	0.675/0.05
D 4	5	0.80/0.05, 0.675/0.05
D 5	5	0.675/0.05, 0.80/0.05

Test series D1-D3 were subjected to fatigue loading at one load level mainly in order to get a S-N curve and a strain development for high strength LWA-concrete. Test series D4-D5 were subjected to fatigue loading at two load levels in order to test the validity of Miner-Palmgrens sum. In all series the strain developments were registered.

4.3 Test results

Static test results for cubes and cylinders which were used as reference parameters for the fatigue tests, are shown in Table 9. The table shows that the cylinder strength was higher than the cube strength. Probably since the cylinders were drilled out

Table 9. Static capacities (compression).

FOR TEST SERIES	CUBE 100x100x100 mm	CYLINDER Ø 130x300 mm	CYLINDER Ø 100x250 mm
28 days	66.6	69.0	69.7
D 1	70.4	-	73.7
D 2	69.7	72.8	73.7
D 3	72.7	-	74.4
D 4	69.7	72.8	73.7
D 5	69.7	72.8	73.7
	MPa	MPa	MPa

from a well vibrated plate. The modulus of elasticity (E_c), secant module between 0 and 40% of the static failure load, was 25 100 MPa.

The test results for series D1-D3 are presented in Figs. 17, 18, 19 and in Table 10. They may be summarized as:

- The mean fatigue life ($\bar{N}$) for the cylinders in series D1, D2 and D3 were 587, 269 821 and 17 772 cycles (Table 9).
- The strain developments ($\bar{\varepsilon}_L$, $\bar{\varepsilon}_T$) were relatively constant during the test periods. This shows that the effects of dynamic creep are small (Figs. 17, 18).
- The longitudinal strain ($\bar{\varepsilon}_L$) was $\sim$2.5 o/oo before fatigue failure in series D2 and D3 and $\sim$3.0 o/oo in series D1. This indicates that the "failure strain" decreases with lower load level for LWA-concrete. For static tests the "failure strain" was $\sim$3.5 o/oo as for ND-concrete (Figs. 17, 18).
- The fatigue failure of LWA-concrete was of brittle nature and the cracks propagated through the aggregate (Fig. 19).
- During the tests of the instrumented cylinders the load was stopped for strain registrations at different number of cycles. It was observed that the fatigue life for these cylinders was considerably less than for the non-instrumented cylinders (Table 17).
- The stress-strain curves at different number of cycles showed that the hysteresis effects were small, even during the first cycles.

Table 10. Fatigue life for the cylinders in series D1-D3.

TEST SERIES	TEST SPECIMEN	LOAD LEVEL	FREQUENCY (Hz)	INSTRUMENTED	FATIGUE LIFE -N	MEAN FATIGUE LIFE -$\bar{N}$
D 1	D1-1	0.80/0.05	0.5	yes	306	587
	D1-2		0.5	yes	445	
	D1-3		0.33	yes	330	
	D1-4		0.5	no	863	
	D1-5		0.5	no	989	
D 2	D2-1	0.65/0.05	2.9	yes	83 874	269 821
	D2-2	0.65/0.05		yes	163 213	
	D2-3	0.65/0.05		no	98 348	
	D2-4	0.625/0.05		no	2 436 602 *	
	D2-5	0.60/0.05		no	>2 000 000 *	
	D2-6	0.65/0.05		yes	733 849	
D 3	D3-1	0.675/0.05	1.5-2.5	yes	13 754	17 772
	D3-2		1.5-2.5	yes	11 377	
	D3-3		1.5-2.0	yes	13 603	
	D3-4		2.0-2.5	no	21 541	
	D3-5		2.0-2.5	no	28 585	

* not included in $\bar{N}$

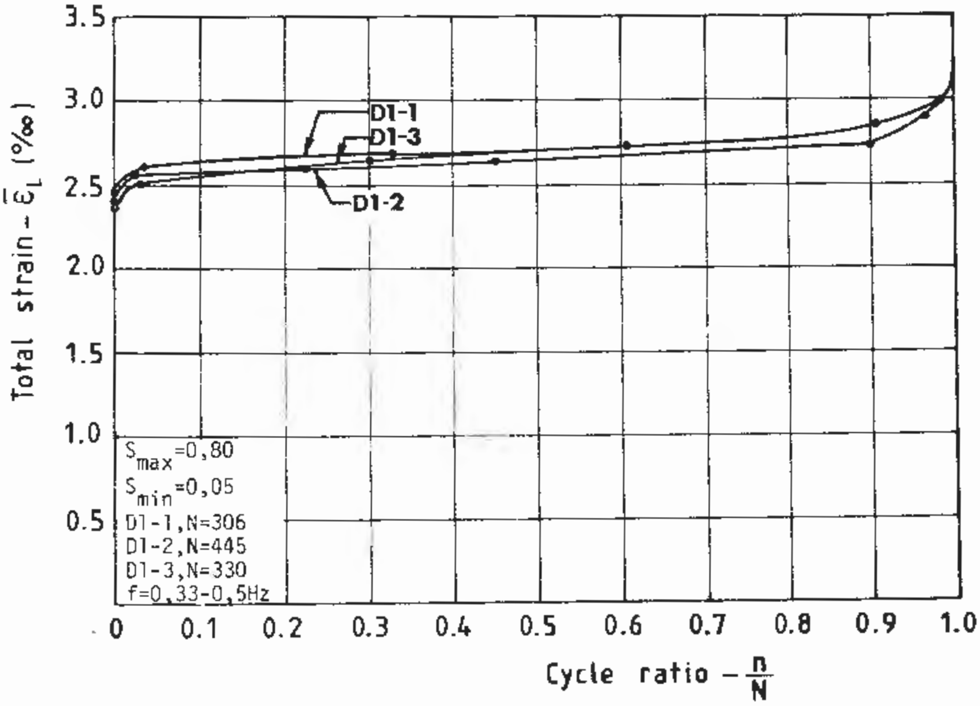


Fig. 17. Longitudinal strain development for the instrumented cylinders in series D1.

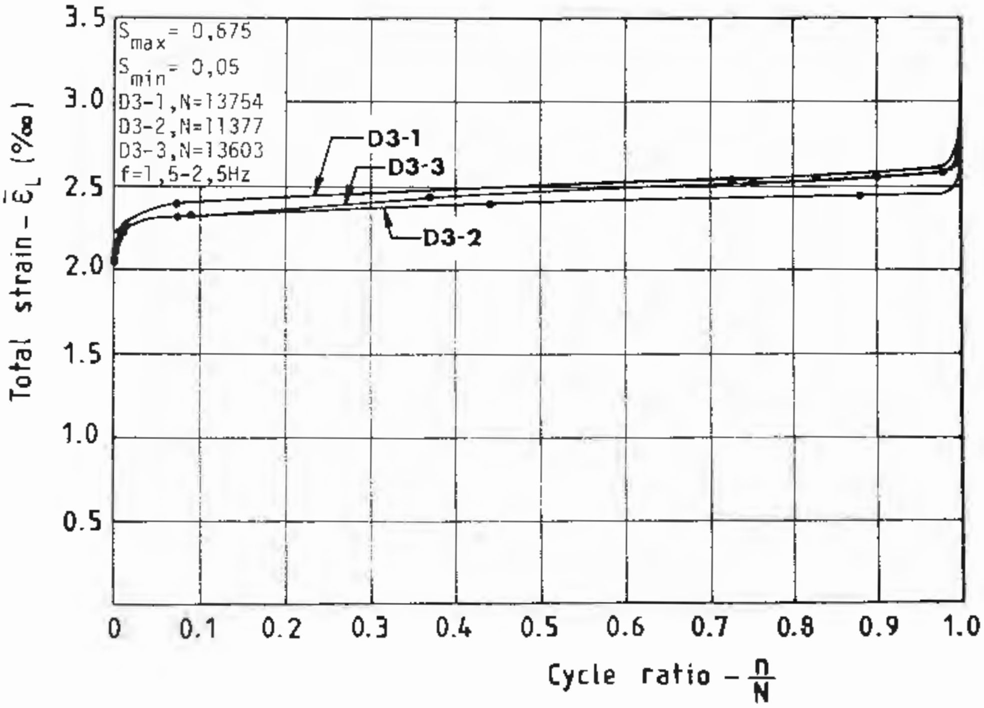


Fig. 18. Longitudinal strain development for the instrumented cylinders in series D3.

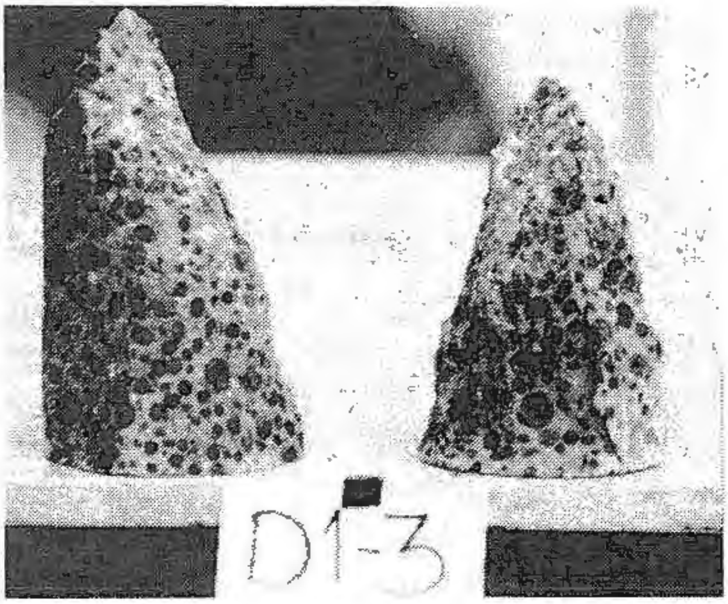


Fig. 19. Typical surfaces after fatigue failure.

The test results for series D4-D5 are presented in Figs. 20-21 and in Tables 11-12. They may be summarized as:

- The mean Miner-Palmgrens sum ($\bar{M}$) for the cylinders in series D4 was 0.67. For all the tests Miner-Palmgrens sum were less than 1.0. This indicates that a higher load level before a lower load level will give Miner-Palmgrens sum less than 1.0 (Table 10).
- The mean Miner-Palmgrens sum ($\bar{M}$) for the cylinders in series D5 was 2.19. 4 og 5 tests gave Miner-Palmgrens sum greater than 1.0. This indicates that a lower load level before a higher load level will give Miner-Palmgrens sum greater than 1.0 (Table 11).
- The strain developments for the tests in series D4-D5 show the same tendency as for the tests in series D1-D3 (Figs. 20-21).
- The fatigue failure was of brittle nature and the cracks propogated through the aggregate.

Table 11. Miner-Palmgrens sum with fatigue failure of the cylinders in series D4.

CYLINDER	INSTRU-MENTED	LOAD LEVEL 1 (0,80/0,05)			LOAD LEVEL 2 (0,675/0,05)			$M = M_1 + M_2$
		f(Hz)	n_1	$M_1 = \frac{n_1}{N_1}$	$f(H_z)$	n_2	$M_2 = \frac{n_2}{N_2}$	
D4-1	yes	0,5	150	0,26	0,75	0	0	0,26
D4-2	yes	0,5	150	0,26	0,75	4560	0,26	0,52
D4-3	yes	0,5	150	0,26	0,75	11792	0,66	0,92
D4-4	no	0,5	150	0,26	0,75	10393	0,59	0,85
D4-5	no	0,5	150	0,26	0,75	9577	0,54	0,80
								$\bar{M} = 0,67$

Table 12. Miner-Palmgrens sum with fatigue failure of the cylinders in series D5.

CYLINDER	INSTRU-MENTED	LOAD LEVEL 1 (0,675/0,05)			LOAD LEVEL 2 (0,80/0,05)			$M = M_1 + M_2$
		f(Hz)	n_1	$M_1 = \frac{n_1}{N_1}$	f(Hz)	n_2	$M_2 = \frac{n_2}{N_2}$	
D5-1	yes	0,75	4500	0,25	0,50	1114	1,90	2,15
D5-2	yes	0,75	4500	0,25	0,50	291	0,50	0,75
D5-3	yes	0,75	4500	0,25	0,50	1158	1,97	1,22
D5-4	no	0,75	4500	0,25	0,50	2330	3,97	4.22
D5-5	no	0,75	4500	0,25	0,50	1393	2,37	2,62
								$\overline{M} = 2,19$

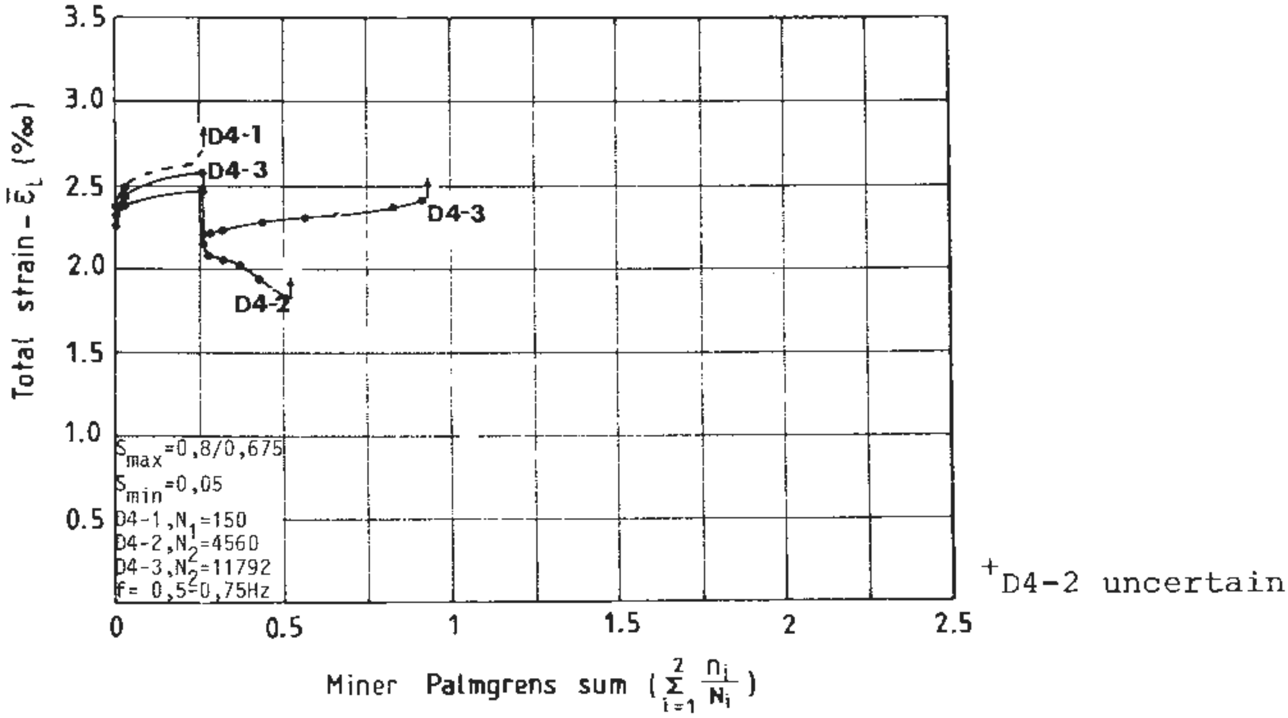


Fig. 20. Longitudinal strain development for the instrumented cylinders in series D4.

4.4 Discussion and conclusions

The fatigue tests of the LWA-concrete cylinders, $f_{cyl} \sim 73$ MPa, were carried out almost identical to the tests of equal ND-concrete cylinders, $f_{cyl} \sim 45$ MPa, reported in /7/. Comparisons between these tests therefore gave a basis to evaluate the properties of LWA-concrete subjected to fatigue loading.

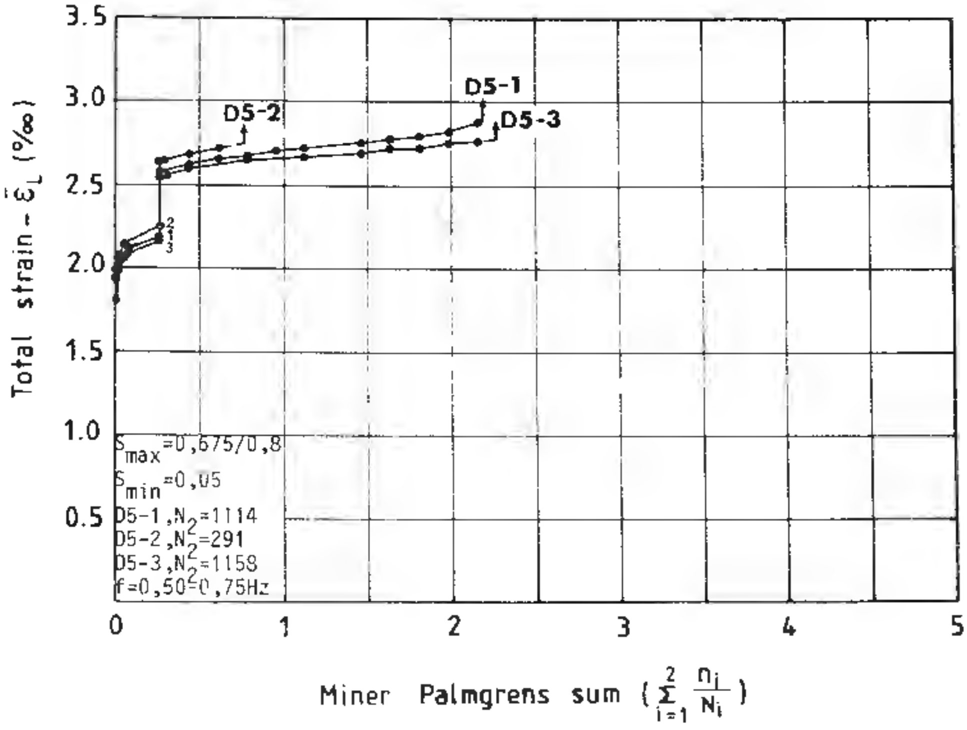


Fig. 21. Longitudinal strain development for the instrumented cylinders in series D5.

In Fig. 22 the fatigue life for the cylinders in series D1-D3 are plotted together with a S-N curve (eq. 4.1) based on the test results in /7/ (50% probability for failure, f = 5 Hz).

$$\log_{10}N = 1.841\ (S_{max})^{-3.033}\ ,\quad S_{min} = 0.05 \qquad (4.1)$$

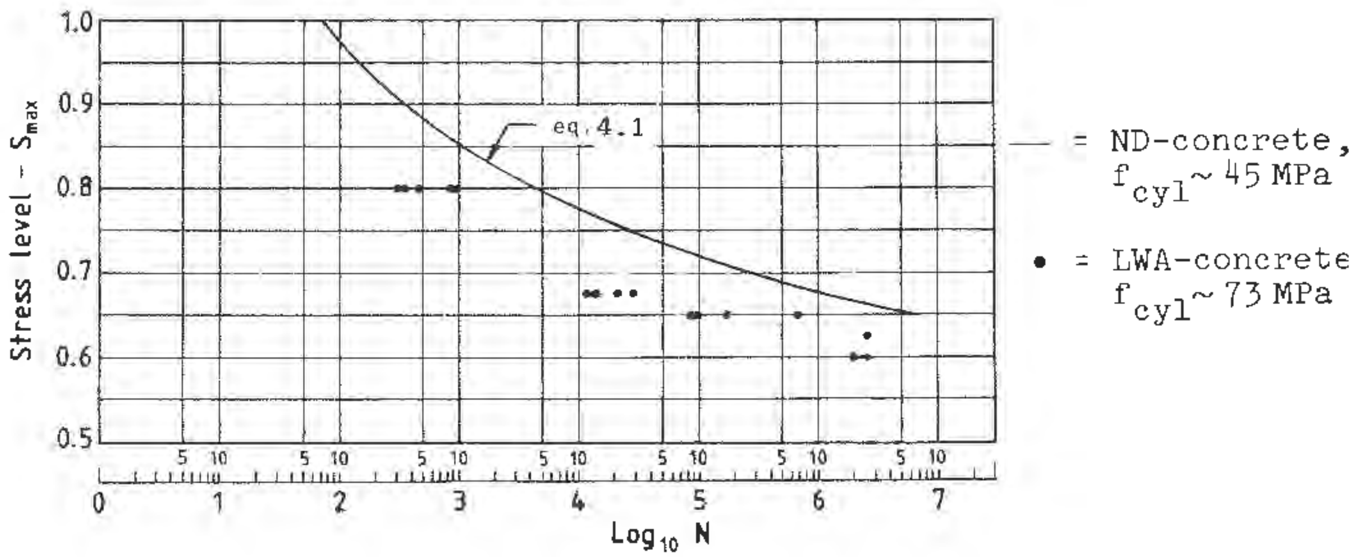


Fig. 22. Fatigue life of LWA-concrete and ND-concrete based on cylinder tests (Ø 100x250 mm).

The results in Fig. 22 show that the scatter in the fatigue life of LWA-concrete is relatively small.

The strength of the LWA-concrete was approximately 62% higher than the strength of the ND-concrete that was used for comparison in Fig. 22. For a more relevant comparison of the fatigue properties of LWA-concrete and ND-concrete it is necessary to test these types of concrete with the same cylinder strength.

Because of the considerable difference in cylinder strength between the LWA- and the ND-concrete the stress variation in MPa between σ_{max} and σ_{min} is greater for the LWA- than for the ND-concrete. This also might influence the results in Fig. 22.

It also should be noted that the fatigue properties for submerged LWA-concrete and LWA-concrete exposed to external water pressure cannot be deduced from the test results presented here.

Series D4 and D5 were subjected to fatigue loading at the same two load levels, but in opposite order. Miner-Palmgrens sum shows that a higher load level before a lower load level will give a sum less than 1.0. For the opposite load situation the sum will be greater than 1.0. Similar tests were performed for ND-concrete in /7/. These tests showed the same tendency as for LWA-concrete.

Typical strain development curves for LWA-concrete and ND-concrete cylinders are shown in Fig. 23. It is obvious that the

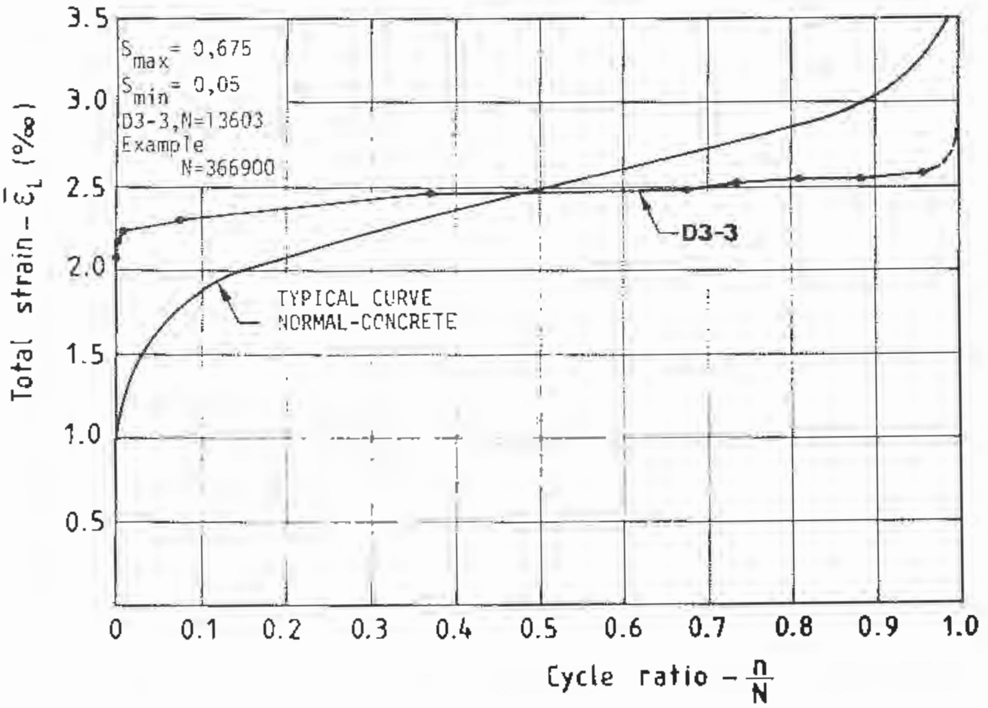


Fig. 23. Typical longitudinal strain developments for LWA-concrete and ND-concrete based on cylinder tests.

strains for ND-concrete increase considerably during the fatigue loading period, especially during the first and last cycles. For LWA-concrete the strain are relatively constant until failure. The test results of the cylinders in series D1-D3 show that the "failure strain" for LWA-concrete decreases with lower load levels. For ND-concrete the "failure strain" is ~3.5 o/oo for static and dynamic tests.

During the tests of LWA-concrete it was observed that the cracks propogated through the aggregate. The fatigue failure for LWA-concrete was also more of brittle nature than for ND-concrete.

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REFERENCES

/1/ Hoff, A. and Høiseth, K. "Høyfast betong. Delrapport 2. Søyler under sentrisk belastning" (High strength concrete. Columns exposed to centric loads). SINTEF-FCB report STF65 A83049, Cement and Concrete Research Institute, Trondheim, August 1983

/2/ Hoff, A. "Bruk av høyverdig kamstål (Ks60) i betongkonstrusjoner. Søyleforsøk" (High-grade reinforcement (Ks60) in concrete structures. Testing of columns). SINTEF-FCB report STF65 A79053, Cement and Concrete Research Institute, Trondheim, December 1979

/3/ Høiseth, K. "High strength lightweight aggregate concrete. Part 2: Testing of columns". SINTEF-FCB report STF65 F84028, Cement and Concrete Research Institute, Trondheim, May 1984

/4/ Modéer, M. "A fracture mechanics approach to failure analyses of concrete materials". TVBM-1001, Div. of Building Materials, University of Lund, Lund, Sweden, June 1979

/5/ Hoff, A. "High strength lightweight aggregate concrete. Part 1: Tests on reinforced beams". SINTEF-FCB report STF65 F84016, Cement and Concrete Research Institute, Trondheim, February 1984

/6/ Håverstad, T.A. "High strength lightweight aggregate concrete. Part 3: Fatigue of LWA-concrete cylinders". SINTEF-FCB report STF65 F84015, Cement and Concrete Research Institute, Trondheim, March 1984

/7/ Holmen, J.O. "Fatigue of concrete by constant and variable amplitude loading". SINTEF-FCB report STF65 A79058, Cement and Concrete Research Institute, Trondheim 1979

/8/ Leslie, K. et al: "Flexural Behaviour of High Strength Concrete Beams", ACI Journal, Sept. 1976

/9/ Bernhardt, C.J. "Dimensjonering ved prøving". Dr.ing. course, 1980

/10/ NS3473: Norwegian Code for design of concrete structures

/11/ CEB-FIP Model Code, 3rd Edition, 1978