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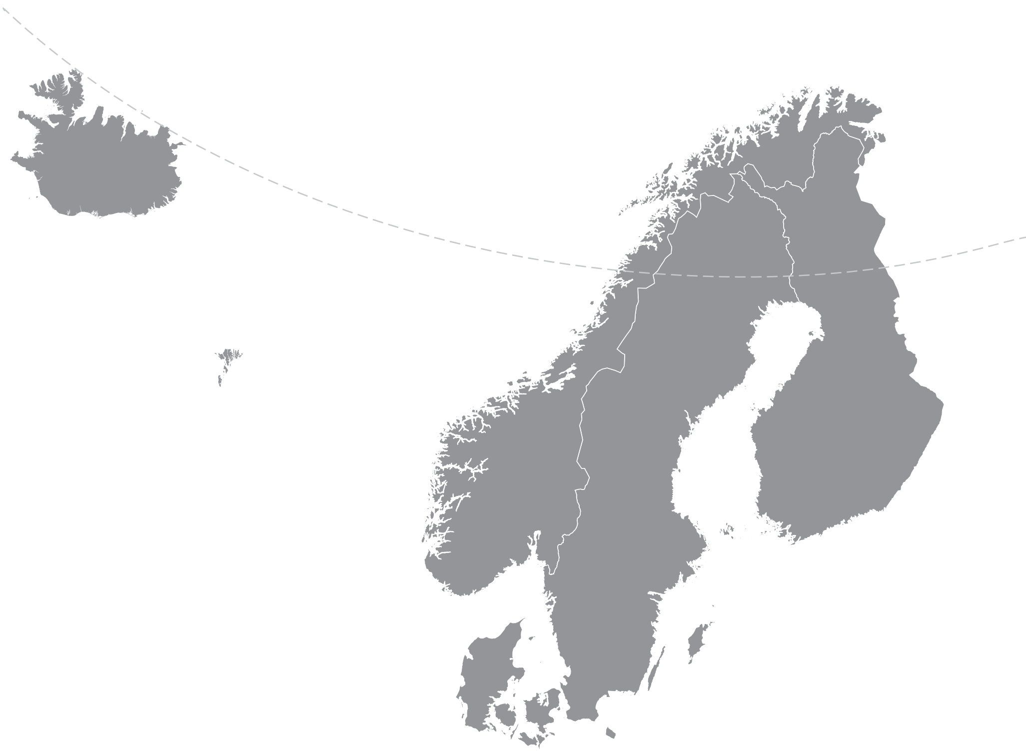

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Behaviour and Capacity of Lightweight Aggregate Concrete Beams with and without Shear Reinforcement

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ABSTRACT
The main disadvantages of lightweight aggregate compared with normal weight concrete are its brittleness at the material level in compression and uncontrolled crack propagation. This experimental investigation consists of five beams with lightweight concrete with Stalite as aggregate. Main goals were to investigate behaviour and capacity of the beams with and without shear reinforcement subjected to four-point bending test and compare those results with previous experimental work. The main test parameters were the shear span ratio (*a/d*) and amount of the shear reinforcement. Existing standards underestimate shear capacity because they do not differ between shear span (a). Tested beams were more ductile than expected, and cracking was similar as for normal weight concrete beams. According to this experimental investigation, the shear capacity in beams without shear reinforcement should be based on inclined cracking loads.

Keywords: Lightweight aggregate concrete, testing, shear reinforcement, bending, ductility.

1. INTRODUCTION

This investigation is part of the ongoing research programme, "Durable advanced concrete structures (DACS)". One part of this programme is to investigate the structural behaviour of lightweight aggregate concretes (LWAC), i.e. concretes with an oven-dry density below 2000 kg/m^3. A general characteristic of LWAC is its very high degree of brittleness at the material level and especially in compression, which results in sensitivity to stress concentrations and rapid crack/fracture development. This influences the behaviour of concrete where its tensile strength is important, as for instance with its shear and bond strength. To investigate the behaviour of LWAC, beams with and without shear reinforcement were subjected to a four-point bending test. The main test parameters were the shear span length to effective height ratio (*a/d*) and amount of shear reinforcement. For all beams, the shear loads at diagonal cracking and at failure were plotted as a function of the *a/d* ratio and compared with previous experimental work. For comparison, tested beams were of the same size and with the same area of compression and tension reinforcement as in earlier shear tests on other normal density (ND) and lightweight aggregate (LWA) concrete beams [1,2]. In addition small specimens were used to find the compressive strength [3, 4], splitting tensile strength [5], Young's modulus [6, 7], and fracture energy [8].

To produce the concrete, a lightweight aggregate Stalite was used to achieve an oven-dry density of about 1850 kg/m^3 and a compressive strength of about 65 MPa. Agreggate Stalite is the argillite slate, laminated, fine-grained siltstone of clastic rock. The foothills region of North Carolina is the only place where slate is exhausted as raw material to produce Stalite. The bulk density ranges from 720-1120 kg/m^3 for both coarse and fine aggregate and the hardness of the material is equivalent to that of the quartz [9, 10].

2. EXPERIMENTAL TEST PROGRAM AND RESULTS

2.1 Test specimens

Five reinforced LWA concrete beams with and without shear reinforcement were tested in a four point bending test. The loading system was designed to produce a constant moment in the middle part of the beam. The cross section (*bxh*) of the beams was 150x250 mm and the length 2900 mm. The main test parameters that were warried in this test were the shear span length to effective height ratio (*a/d*) and amount of shear reinforcement. In the ratio *a/d*, *a* is the shear span (the length between loading point and support) and *d* the effective height of the cross section (the distance from the top surface to the centre of the tensile reinforcement), which in this case is 219 mm. Two pairs of beams without shear reinforcement each with shear span ratio *a/d*=2.3 and *a/d*=4.0 were tested. In addition one beam, which contained shear reinforcement distributed between the support and loading point and with shear ratio *a/d* =3.43 was tested. An overview of the test programme is shown in Table 1.

Table 1-The main test parameters

Beam	a [mm]	d [mm]	a/d [-]	A_c [cm^2]	A_t [cm^2]	s [mm]	$f_{lc,cyl}$ [MPa]	A_t / (bxd) [%]
1	504	219	2.3	1.57	6.03	-	67.5	1.83
2	504	219	2.3	1.57	6.03	-	67.5	1.83
3	876	219	4	1.57	6.03	-	67.5	1.83
4	876	219	4	1.57	6.03	-	67.5	1.83
5	750	219	3.43	1.57	6.28	100	67.5	1.91

Where a is shear span; d is effective hight; a/d is shear span ratio; A_c – area of compresive reinforcement; A_t – area of tensile reinforcement; s is stirrup spaceing; $f_{lc,cyl}$-compressive cylinder strength.

All the beams had three ø8 mm stirrups in each anchorage zone (behind the supports). Beams without shear reinforcement had just two stirrups in the constant moment region below applied forces, while there were no stirrups in the shear spans. One of the tested beams contained shear reinforcement distributed along the shear span. Reinforcement on the tension side consisted of three ø16 mm bars in beams with shear span ratio *a/d* =2.3 and 4.0 and beam with *a/d* =3.43 had two ø20 mm bars. As compressive reinforcement, two ø10 mm bars were used in all the beams. Longitudinal and cross section details of the beam specimens are shown in Figure 1.

All the beams and small samples, cubes and cylinders, were cast from the same concrete batch. The beams were demoulded 24 hours after casting and further cured in the laboratory under wet burlaps covered with a plastic sheet. Two days before the testing beams were taken out and

prepared for instrumentation. Finally, the beams were painted white for easier detection of cracks.
Small samples, which were cast in order to identify the mechanical properties of the LWAC, included 12 cubes (with dimensions 100x100x100 mm), 15 cylinders (ø100x200mm) and 3 small beams (100x100x1200mm). From mentioned samples, the authors derived the stress-strain diagram, compressive strength for cube and cylinder, tensile strength, Young's modulus of elasticity and fracture energy. All small specimens were demolded after 24 hours and kept in water until testing day. Compression test on cubes and cylinders were carried out in the start, middle and last day of beam testing.

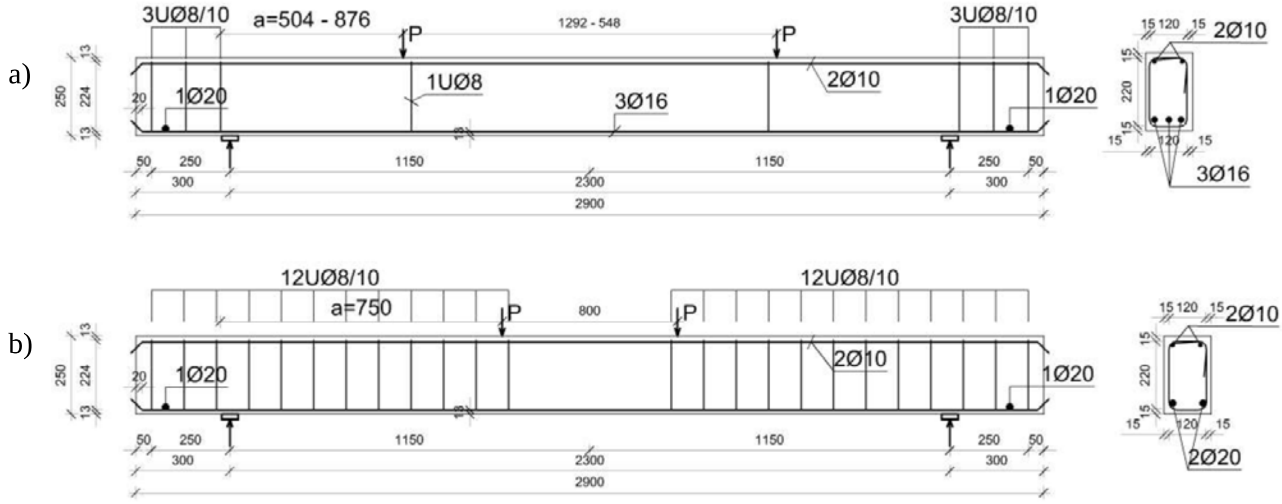


Figure 1 – Reinforcement and cross section details for the beams a) without and b) with shear reinforcement

2.2 Material and mix properties

The concrete mixture was prepared from one batch. The lightweight aggregate was the ½" fraction from Stalite [11]. The moisture content and the absorbed water in the Stalite were measured, which was necessary input when designing the concrete mix [12, 13]. The moisture content was 8.2%, and the absorption after 24 hour and 100 hours was 6 % and 8.5%, respectively. Table 2 gives the concrete mixture.

Table 2 – Concrete mixture for LWAC 65

Constituent	Weight [kg/m^3]
Cement (Norcem Anlegg)	398.21
Silica fume (Elkem Microsilica)	19.62
Water (free+absorbed 24 hour)	93.87+40.12=134
Sand (Årdal (NSBR) 0/8 mm)	745.56
Aggregate (Stalite 1/2")	618.79
Superplasticiser (Sika ViscoCrete RMC-420)	3.20

The mixing was done using a 0.8 m^3 laboratory mixer. First cement, silica fume, Stalite and sand were mixed for approximately 2 min. Water and superplasticiser were continuously added and adjusted during mixing, until the desired workability of the concrete was achieved.

Characteristics of the fresh concrete were: density 1990 kg/m^3, air content 2.6 % [14] and slump 140 mm [15]. The reinforcement was of the type B500NC [16]. The yielding stress of the reinforcement assumed in calculation is approximately 560 MPa.

2.3 Test Setup and procedure

The load was applied with a mechanical screw jack and was transferred to the test beam through a steel spreader beam, which was supported, on two steel rollers covering the entire width of the beam. The loading point has free rotation transversal to the beam. Between jacks and the beam surface, it was used 50 mm wide steel plates and a 15 mm thick fibreboard with the same width. The supports were both free for rotation and displacement in the longitudinal direction. At the supports, only steel plate was between the support and the beam. The supports were placed 300 mm from the beam-ends. To avoid anchorage problems, short ø20 mm reinforcement bars were welded to the tensile reinforcement in this region. The load was measured using electrical load cell under the screw jack of maximum capacity 1000 kN. Instrumentation set-up differs between beams with and without shear reinforcement. Figure 2 shows the layout of the test set-up for the beams with and without shear reinforcement and view from the laboratory.

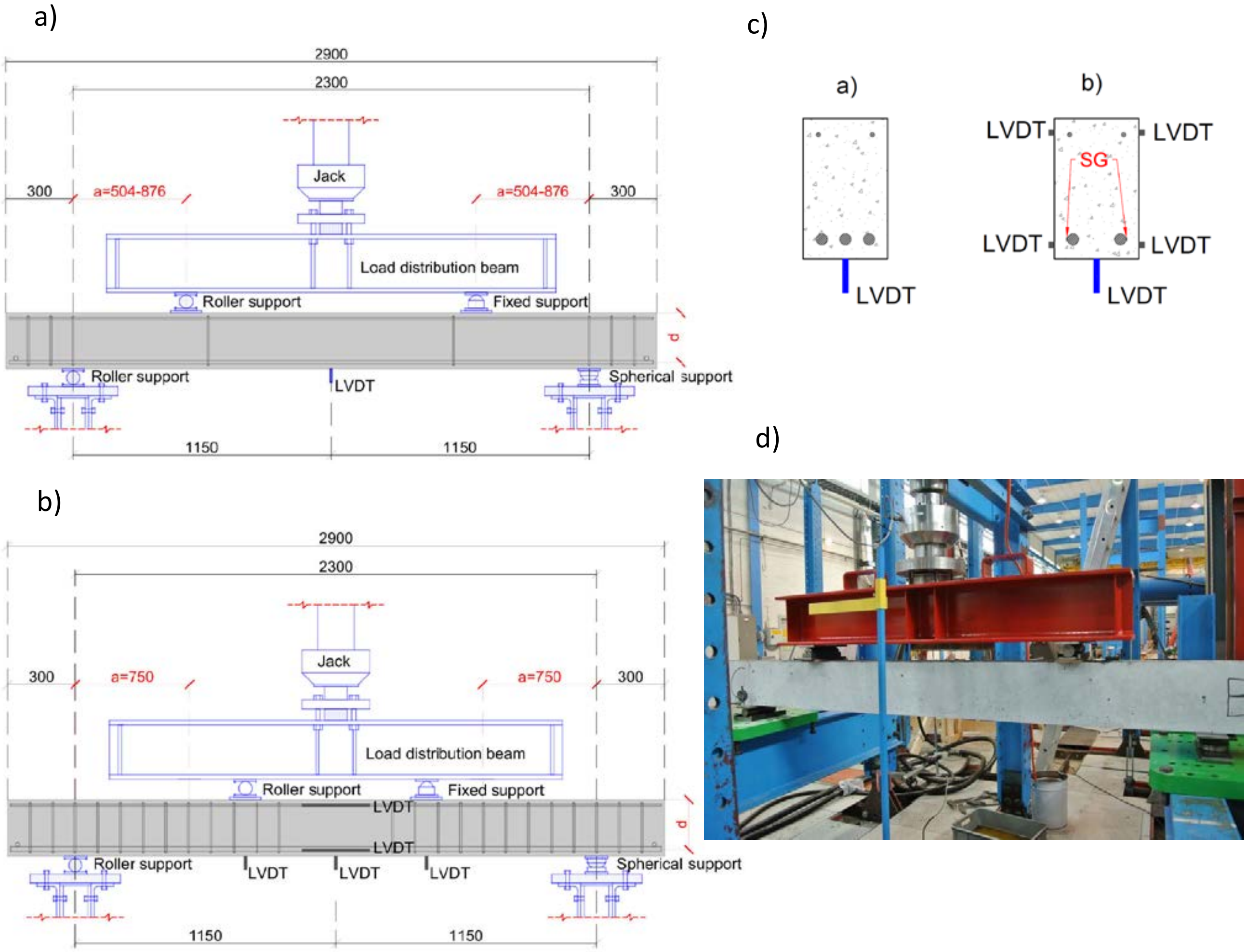


Figure 2 – Test set-up off beams a) without shear reinforcement; b) with shear reinforcement; c) detail middle cross section for beam without and with shear reinforcement; d) view from the laboratory.

For observation of beam with shear reinforcement, more instrumentation was used. Linear Variable Displacement Transducers (LVDT) measured the deflections in the middle of the beam span in all the beams. In beam with shear reinforcement additional LVDT were used for measuring deformations under the loading points and over the middle cross section. Length of LVDT for observation of middle section was 200 mm. In addition, in this beam strain gauges (SG, type FLA-6-11-5L with gauge resistance of 119,5±0.5Ω) were inserted on the tensile reinforcement inside of the middle cross section. All measuring devices (LVDTs and SGs) together with the load cell were connected to HBM eight channel spider to record the data. From here, data were sent to computer using a specific software program, where they are processed and stored in a text file.

The load was applied stepwise in increment of 20 kN for the beams without shear reinforcement until failure. For the beam 5, with shear reinforcement, an increment of 40 kN was used since calculated capacity was doubled. The first tested beam was beam 1, one of two beams with shear span ratio 2.3. This beam was tested with loading steps of 10 kN. Since there was too many steps and testing takes, more than 2 hours loading steps were increased on 20 kN for the next beam with 2.3 ratio and for the two beams with 4.0 ratio. This is displacement control test with loading rate 0.5 mm/minute for the beams without shear reinforcement and 1 mm/minute for beam with shear reinforcement. The deflection measurements were carried out as a control. At each step, deflections in the middle section and under the loading points were measured. The loading time for each step takes about 10 minutes, from what 5 minutes was break and mainly used to draw the crack progression with dark pen. The output data were recorded by the data acquisition system. Pictures were taken after each step and failure.

3. EXPERIMENTAL RESULTS

3.1 Results for small specimens

Concrete class measured from small samples was LC65 and that actually represents high strength lightweight concrete. The compressive failures of cubes and cylinders were very explosive which is typical for high strength and lightweight concrete.

Small specimens were tested after 28 days for determination of compressive strength, 29 days for tensile strength and Young's modulus. Small beams for fracture energy were tested after 36 days. A brief summary of the small scale test results is given in Table 3.

Table 3 – Mechanical properties for LWAC

Saturated density	ρ_{cs} = 1980 kg/m3
Oven dry density	ρ_{cv} = 1850 kg/m ^{3}
Compression cube after 7 days	$f_{lcm,7}$ = 57,3 N/mm^2
Compression cube after 28 days	$f_{lcm,28}$ = 73,8 N/mm^2
Compression cylinder	f_{lcm} = 67,5 N/mm^2
Tensile strength	f_{lctm} = 4,05 N/mm^2
Modulus of elasticity	E_{lcm} = 24175 N/mm^2
Fracture energy	G_F = 76,7 Nm/m^2

3.2 Capacity of the beams

Table 4 shows the results of the tested beams. In the table, the following forces are plotted: the forces when the first bending crack occurred (P_{fcr}), force of diagonal cracking (P_{cr}) and failure force (P_u). The shear capacity ($P_{calc}=V_{Rd,c}$) for both pairs of the beams without shear reinforcement was calculated according to Eurocode 2 [17, 18, 19] and Norwegian standard NS 3473 [20, 21] and those values were 42.8 kN and 44.6 kN, respectively. Calculation of shear capacity according to both standards was influenced by external load but position of the load was not taken into account, which resulted with the same capacity for the beams with shear span ratio 2.3 and 4. Calculated shear capacity for all the tested beams plotted in the table 4 was according to Eurocode 2 [17, 18, 19]. Shear capacity according to Eurocode 2 for the beam 5 which contain shear reinforcement was almost doubled and that value is 92.85 kN. Shear capacity for beam with ratio 2.3 and 4 was also calculated according to Norwegian standard NS 3473 and that value was plotted in table 5. Reason for this is that all previous results for comparison were calculated by using the same standard NS 3473. Tensile strength used in this calculation is obtained from small scale testing [5] and later interpolated according to NS 3473. For lightweight concrete the values are multiplied by a reduction factor ($0,30+0,70\rho/\rho_1$), where ρ is the dry density of the lightweight concrete and $\rho_1=2400$ kg/m^3. Hence, with a dry density of 1850 kg/m^3 the reduction factor was 0,839.

The beams were tested 30 days (beam 1_ with shear ratio 2.3), 31 days (2_2.3), 34 days (3_4.0 and 4_4.0) and 35 days (5_3.43) after casting.

Table 4-Test parameters and results of first cracking, diagonal cracking, failure loads and calculated shear capacity in accordance with EC2[17,18]

Beam	a [mm]	d [mm]	a/d	P_{fcr} [kN]	P_{cr} [kN]	P_u [kN]	P_{calc} [kN]	P_{cr} / P_{calc}	P_u / P_{calc}
1	504	219	2.3	25	45	92.3	42.8	1.05	2.15
2	504	219	2.3	22.5	44.5	127.2	42.8	1.04	2.97
3	876	219	4	21	36.8	44.4	42.8	0.86	1.04
4	876	219	4	21	33	62	42.8	0.77	1.44
5	750	219	3.43	25	40	91.4	92.85	0.43	0.98

In all the beams first cracking started for approximately, same load level of 21-25 kN. Formation of the first shear diagonal cracks depend and differs from the shear span ratio. So in the beams with higher shear span ratio shear cracks formed for lower load level, while by reducing shear span (a), distance between loading point and support , shear capacity increased.

3.3 Load-deformation relationship

The load–deformation relationship was followed during the entire test. Figure 3 shows the load-deformation relationship for the centre point of the cross section at middle span for all the five beams. As expected, beams that do not contain shear reinforcement along the shear span and with low shear span ratio 2.3, had higher capacity. Beam 2, with shear span ratio 2.3 had even higher capacity than the beam 5 that contain shear reinforcement. The resemblance between two identical beams is quite good before failure. As expected, the response depends on the shear span ratio. With a low ratio a concrete strut forms making a direct load transfer of the point load to the support. The load and deformation can increase several times the shear cracking load.

However, the ultimate load is more unpredictable in this case. For a high shear span ratio, pure shear governs the failure mode. The failure loads for these two beams can be defined to be the same. Beam 4 with shear span ratio 4.0 was able to sustain a higher load after significant deformations, but it is not possible to rely on such deformations in a design situation. As expected the beam with shear reinforcement showed significant ductility, deflections are doubled for the same load level and before failure this beam were able to sustain even increase of loading. In addition, one of the beam with shear span ratio 2.3 showed very ductile behaviour. In general, all the tested beams are in the very good agreement considering the main test parameter shear span ratio.

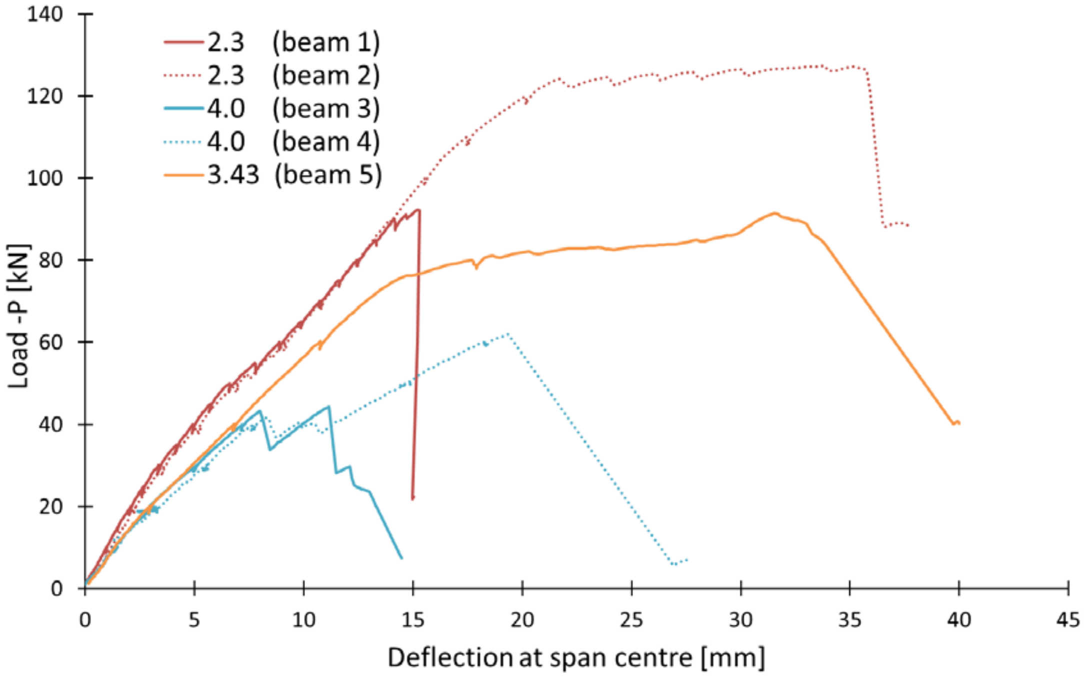


Figure 3 – Load deflection curves for all the tested beams

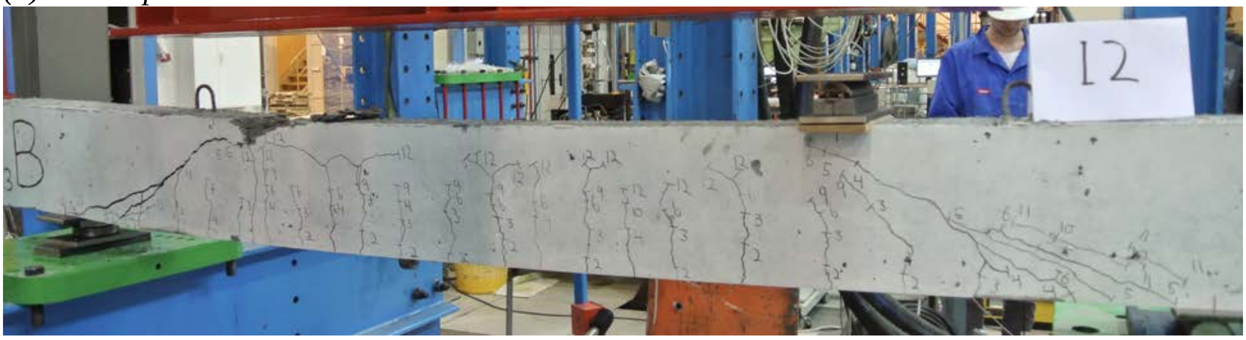
Results of strain distribution were observed and recorded over the middle cross section just in beam 5 with shear ratio 3.43. Since those results cannot be compared with first four tested beams 1-4. they are not plotted here.

4. DISCUSSION

4.1 Failure modes, cracking patterns and capacity of the beams

The first cracks in all beams started to develop in the constant moment region, between the loading points, on the tension side for load levels between 20 and 40 kN, depending on the *a/d* ratio. As load increased cracks formed along the entire length of the constant moment region and wider. When flexural bending cracks stopped to develop, suddenly, the diagonal shear cracks occurred, but the beams did not fail immediately. The beams with *a/d* ratio of 2.3 formed directly the diagonal inclined shear crack. These cracks appeared suddenly developing from the tension side of the beam towards the compression side near the point loads. For beams with ratio 2.3 diagonal cracks formed at higher load levels than for beams with ratio 4.0. For beams with ratio 4.0 the diagonal shear cracks propagated from one of the flexural bending cracks at the *(a) Shear span ratio 2.3 - beam 1*

(b) Shear span ratio 2.3 - beam 2



(c) Shear span ratio 4.0 - beam 3

(d) Shear span ratio 4.0 - beam 4

(d) Shear span ratio 3.43 – beam 5

Figure 4 – Final failure state for beams (a) 2.3-1; (b) 2.3-2; (c) 4.0-3; (d) 4.0-4 and (e) 3.43-5

tension side. For beams with shear reinforcement and shear span ratio 3.43 cracking started from the tension side. Location of cracks depends from reinforcement layout. Flexural bending cracks start to form between stirrups. Later diagonal shear cracks propagated from flexural bending cracks. For a certain load level propagation of diagonal cracks was stopped due to shear reinforcement while propagation of bending cracks were continued. At the end, bending cracks led to failure in beam with shear reinforcement, while in beams without shear reinforcement diagonal shear cracks lead to failure. Figure 4 showed the final failure state of all the tested beams.

In general, beams with lowest shear span ratio had higher capacity, see Table 4. Formation of cracks for beams with ratios 4.0 and 3.43 was equal in both shear spans. For beams with ratio 4.0 failure happened suddenly in one of the shear span, while beam with ratio 3.43 failed in compression between the loading points. In the beams with ratio 2.3 shear cracking was more non-symmetrical, with more and larger cracks at one end of the beam, and beams failed at the shear span.

Failure that happened between support and the loading point, shear zone, is defined as shear tension failure or shear compression failure depending which zone cracked [22, 23]. Here we can notice that beams with ratio 2.3 in the final failure phase cracked always under the loading points so they had shear compression failure. In beams with ratio 4.0 development of cracks on a tension shear zone were wide, cracks followed tensile reinforcement, and they even continued in anchorage area, so they had shear tension failure.

Behavior of beam with ratio 3.43 in beginning matches well with beams with ratio 4.0. However, due to shear reinforcement this beam had approximately double jump in capacity and at the end cracking of compression zone between loading point happened. This type of failure can be defined as bending compression failure [22, 23].

In general, cracking and failure mechanism for beams without web shear reinforcement, which is typical for NWC, is that crack will appear in the shear span by increasing the load. Owing to the presence of the shear stresses, they bend towards the axis of the beam. Other secondary cracks due to stress redistribution may also appear. The development of the diagonal cracks stops at a certain load level, while the crack propagating into the compression zone. The beam either collapses simultaneously with the appearance of this diagonal crack or continue to sustain higher load until crushing the concrete in the compression zone. The term diagonal crack load is in this work defined as the load when the formation of the specific shear crack happened, which later develops into shear failure. The load for which the testing beam collapses is the ultimate load or load carrying capacity [24, 25, 26].

Due to the brittle nature for LWAC it is typical that diagonal cracking load is equal to ultimate load. However, after formation of diagonal cracks in this experiment beams were able to carry an increase the loading [26, 27]. The cracks propagated almost horizontally along the tensile reinforcement and diagonally into the compression zone. In some cases the cracks even passed the loading point and into the constant moment region. In the final stage, the shear cracks opened widely together with sliding along of the diagonal cracks and resulted in crushing of the concrete close to the loading point, see Figure 4. In the beam that contained shear reinforcement formation of cracks have determined by reinforcement layout and for the certain load level development of shear cracks is limited and stopped with shear reinforcement. This resulted in significant increase in capacity and contribute to ductile behavior of the beam. Even this as all the tested beams are from lightweight aggregate concrete, this beam was able to withstand increase of loading in final failure phase.

4.2 Comparison with previous experimental work

Similar tests have earlier been carried out for higher strength concrete classes of normal density and lightweight aggregate concretes. Table 5 shows a comparison of these tests with the present investigation tests with a/d = 2.3 and with a/d = 4.0 for the beams without shear reinforcement. Beam with shear span ratio 3.43 is out of the comparison since this beam contained shear reinforcement. In all previous investigation the tests conditions were the same, including the rig, the cross section and the amount and distribution of reinforcement [1,2]. Shear capacity for all the previously tested beams were calculated according to Norwegian standard NS 3473 [20, 21]. Because of the comparison shear capacity for the beams with shear span ratio 2.3 and 4 was also calculated according to the same standard and that value was 44.6 kN. Again all the beams without shear reinforcement have the same capacity since standard in calculation of the shear capacity do not include the load position.

The concrete types compared with previous investigations are ND65, ND95, LWA75, LWA40 and LWAC_Leca. The ND65 and ND95 concretes had normal density aggregates from Årdal with maximum aggregate size 16 mm. The mean cylinder strengths of these concretes were 54 and 78 MPa and the dry density was between 2300 and 2350 kg/m^3.

The LWA75 concrete had natural sand (0-4 mm) and Liapor 8 (lightweight aggregate [28, 29]) in the coarse fraction from 4 to16 mm. The mean cylinder strength of this concrete was 58 MPa and the dry density about 1900 kg/m^3. The LWA40 had natural sand (0-4 mm) and Leca 700 aggregate (4-16 mm), while LWAC_Leca also had Leca but several fractions (Leca sand (0-4 mm, crushed), Leca sand (2-4 mm, round), Leca 7 (4-8 mm) and Leca 7 (8-12 mm)). The mean cylinder strengths of these concretes were 37 and 42,7 MPa, and the dry densities about 1600 and 1320 kg/m^3, respectively [1]. For all of these concretes capacity was calculated according to NS3473 (P_{calc}).

The results of main interest for this comparison is the ratio between the observed shear diagonal cracking load and calculated capacity and also ratio between obtained failure load and capacity.

Table 5 - Comparison of the shear strengths for beams with a/d = 2.3 and 4.0, shear capacity is calculated in accordance with NS3473[20]

Beam/Aggregate	*a/d*	$f_{lc,cyl}$ [MPa]	P_{cr} [kN]	P_u [kN]	P_{calc} [kN]	P_{cr} / P_{calc}	P_u / P_{calc}
1/Stalite	2.3	67.5	45	92.3	44.6	1.01	2.07
2/Stalite	2.3	67.5	44.5	127.2	44.6	1.00	2.85
ND65/Årdal	2.3	54	62.2	71.6	55.1	1.13	1.30
ND95/Årdal	2.3	78	66.7	103.5	57.3	1.16	1.81
LWA75/Liapor 8	2.3	58	47.1	126.1	52.0	0.91	2.43
LWA40/Leca	2.3	37	46.6	77.9	39.3	1.19	1.98
LWA_Leca_mix	2.3	42.7	34.3	102.9	42.1	0.81	2.44
3/Stalite	4	67.5	36.8	44.4	44.6	0.82	0.99
4/Stalite	4	67.5	35	62	44.6	0.78	1.39
LWA40/Leca	4	37	38.2	38.2	39.30	0.97	0.97
LWA_Leca_mix	4	42.7	29.4	44.1	42.10	0.70	1.05

where a/d is shear span ratio; $f_{lc,cyl}$-compressive cylinder strength; P_{cr} – load level for first shear crack; P_u – load level of maximum load; P_{calc}– calculated shear capacity acoording to Norwegian standard NS3473 [20].

For beams with a/d = 2.3, the ratio between shear diagonal cracking load and calculated load was larger or equal to 1.0 for all concretes except LWA75 and LWA_Leca_mix. This indicates that they are on the conservative side. For LWA75 and LWAC_Leca_mix, this ratio was less than 1.0, indicating a higher drop in shear strength than predicted by NS 3473. The ratio between ultimate load and calculated one for all LWA concretes was almost 2 or higher, which shows that LWA concretes in general can withstand more loading than predicted by standards. For normal density concretes, this ratio was lower. In general, it is obvious that shear capacity calculated by standards is significantly underestimated compared with experimental results.

It can be observed that the ultimate load capacity for the beams 1and 2 with shear span ratio 2.3 was slightly higher than for the beams with normal weight concretes ND65 and ND95. This result was expected because LWAC concrete with Stalite as aggregate have the same behaviour as normal weight concrete. Slightly higher ultimate load than for normal weight concrete might be result of stronger transition zone between the aggregate and the matrix, which result in cracking development around and trough lightweight aggregate.

The ND65, ND95 and LWA75 concretes were not tested with ratio *a/d* = 4.0. For beams with *a/d* = 4.0, the ratio between shear diagonal cracking load and calculated load for both concretes tested was below 1.0 – indicating that beams with *a/d* = 4.0 had a certain drop in capacity [1,12]. The ratio between ultimate load and calculated one for all the beams is around or higher than 1.0 – indicating that standards are applicable for larger shear span ratios. In general, calculation by standards matches well with experimental results for the beams with ratio 4.0. The same can be noted from the Table 4 for the beam 5 with ratio *a/d* = 3.43, where the ultimate capacity predicted by standard matches well with experimental one.

For tested LWA concrete with Stalite as aggregate, the diagonal cracking load was close to other lightweight aggregate concretes, while the failure load was higher, especially in the case of *a/d* = 4.0.

In comparison with normal density concretes ND65 and ND95, the diagonal cracking load for normal concretes was approximately 30% higher, while failure load for LWAC with Stalite was significantly higher.

Actually, from this experimental investigation from high importance is the fact that after shear diagonal crack were formed beams can withstand increase of load from 30 to 50 %, which is of great importance having in mind that here is tested lightweight aggregate concrete. In addition, by introducing the shear reinforcement in beams with very similar shear span ratio, 4.0 and 3.43, ultimate capacity will increase and nature of failure differ from shear failure to bending failure.

5. CONCLUSIONS

For all tested beams in this experiment, the shear stress at inclined cracking of the beams decreased with an increase in the shear span to effective height ratio (*a/d*). Cracking propagation in the tested beams showed that they were more ductile than expected, which should promote increased investigation and structural use of this type of LWAC. Beam with shear reinforcement showed significant ductility compared to other tested beams.

For beams with larger shear span ratio (3.43 and 4.0) calculations predicted by standards, match well with experimental results, while standards underestimate a lot ultimate capacity in beams with lower sheer span ratio (2.3). Calculation of the shear capacity according to existing standards do not take into account position of the load, shear span a. Standards just differ between the beams congaing or not shear reinforcement. According to this experimental investigation, the design strength for shear in beams and slabs without shear reinforcement should be based on inclined cracking loads.

Comparison with similar tests on other types of lightweight concretes and normal density concretes showed the same that the shear stress at inclined cracking of the beams decreased with an increase in shear span ratio (*a/d*). For concretes tested in this experiment and in previous investigation ratio between the load observed at diagonal cracking and the predicted strengths was in the same range. However, the ratio between observed load at failure and the strengths predicted was significantly higher for the lightweight concrete used in this investigation.

6. FURTHER RESEARCH

During this experimental work was observed that the load-carrying capacity of LWAC members is very similar to that of corresponding ND concrete members. The high strength-to-weight ratio of LWAC compared to ND concrete means that increased use of the material in structural applications would be both economical and environmentally friendly. In the continuation of this project, a better understanding of the ultimate behaviour of LWAC in compression and bending by varying different reinforcement detailing will be of main interest.

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