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Shear Ductility of Reinforced Lightweight Concrete Beams of Normal Strength and High Strength Concrete

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Abstract

An experimental investigation was conducted to study the ductility of shear-predominant (shear-critical) reinforced lightweight concrete beams of normal as well as high strength concrete. A total of 15 shear-critical reinforced concrete beams without and with shear (web) reinforcement were tested, in a 'stiff' testing facility, and complete load-mid span deflection curves including the post-peak portion were obtained. The experimental variables were the concrete compressive strength, shear span-depth ratio and the amount of shear reinforcement. Concrete strength (f'_c) was varied between 30·5 MPa (4430 psi) and 89·3 MPa (12 950 psi). The shear span-depth ratio (a/d) was varied between 1 and 4 and the shear reinforcement ratio (γ_W) was varied between 0 and 0·784%.

For the range of variables tested, the results indicate that for beams with or without shear reinforcement, the shear ductility index (μ) decreases with an increase in the concrete strength. The effect is more pronounced for beams with a/d of 3 as compared to beams with a/d of 1. Normal strength concerete beams with a/d of 3 exhibited a near plastic post-peak response, when the shear reinforcement provided was about five times that of the minimum amount required by Section 11.1.2.1 of the ACI 318-89 Code. Increasing the shear reinforcement ratio (γ_W) up to 0·51% has an insignificant effect on the shear ductility index of beams with a/d of 1. However, for beams with a/d of 2 and 3, the shear ductility index increases. For beams with a/d of 3, increasing the shear reinforcement ratio from 0·51 to 0·65% increases the shear ductility index by 25%. Further increase in the shear reinforcement ratio does not increase the shear ductility index.

Keywords: Beams (supports), reinforced concrete, lightweight concrete, high strength concrete, shear ductility, shear strength, shear span-depth ratio, web reinforcement.

NOTATION

a	Shear span, distance between a concentrated load and the face of the support
b	Width of the beam
d	Effective depth of the beam
f'_c	Compressive strength of concrete
a/d	Shear span-depth ratio
P_{cr}	Diagonal cracking load, i.e. load when the diagonal tension crack crosses the mid-height of the beam
P_{max}	$= P_u =$ maximum load or peak load
Δ_0	Deflection corresponding to maximum load
A_s	Area of tensile reinforcement
A'_s	Area of compressive reinforcement
A_W	Area of web reinforcement, per stirrup
ρ	Tensile reinforcement ratio, $\rho = A_s/bd$
ρ'	Compressive reinforcement ratio, $\rho' = A_s/bd$
ρ_b	Reinforcement ratio producing a balanced strain condition
γ_W	Web reinforcement ratio, $\gamma_W = A_W/bs$

μ_1 Shear ductility, as defined by the ratio of the area of the load–deflection response up to $0{\cdot}75P_{max}$ in the descending portion, to the area up to P_{max}

μ_2 Shear ductility as defined by the ratio of the area of the load–deflection response up to $3\Delta_0$ to the area up to Δ_0. The deflection corresponding to P_{max} is Δ_0

INTRODUCTION

Concretes of higher strengths and lighter weights are very desirable for a variety of applications including offshore and marine structures, slabs and joists in high rise buildings and bridge decks in highway bridge structures. Research information is urgently needed for high strength lightweight aggregate (LWA) concretes.[1,2] LWA concretes with 28 day cylinder strength in excess of 28 MPa (4000 psi) are considered high strength concentrates. In design, a structural engineer must not only provide adequate strength but should also insure that the member exhibits adequate ductility under overload conditions.

The deformability of reinforced concrete flexural members depends on a number of factors, including the tensile reinforcement ratio–balanced reinforcement ratio (ρ/ρ_b), the amount of longitudinal compressive reinforcement, the amount of lateral tie steel and the strength of concrete.[3,4] Information regarding the deflection ductility of high strength flexural members utilizing normal weight aggregate (NWA) and LWA concretes has been developed in a number of studies.[5-9] Although adequate flexural ductility is essential for structures in high seismicity regions, many serious problems relating to the behavior of reinforced concrete structures under severe seismic action can be traced to the poor characteristics of reinforced concrete when subjected to shear.

A number of studies have generated very useful information on the strength and deformation characteristics of shear-predominant (shear-critical) reinforced concrete members of NWA concrete as well as LWA concrete[10-20] and LWA concretes with steel fibers. However, these studies are limited to the diagonal cracking and the maximum load stage and there is no information available regarding the post-peak deformation behavior of shear-critical reinforced concrete members. Knowledge of the post-peak deformation characteristics of shear-critical reinforced concrete members, of lightweight normal strength as well as high strength concrete, is very desirable to better understand the contribution of the shear (web) reinforcement and the failure mechanisms in situations such as under seismic conditions, where higher ductility demands are placed on reinforced concrete members.

It is well recognized that the diagonal tension (shear) failure of concrete is sudden and brittle in nature. For example, the diagonal tension failure of shear critical concrete beams reinforced only with flexural (tensile) reinforcement is brittle and fails with little or no warming. It has been pointed out by a number of investigators[22-25] that testing methodology influences the mode of failure and the post-peak behavior of concrete. For example, the failure of concrete under compressive stresses changes from an uncontrolled brittle-type failure when tested under increasing load conditions to a controlled-type, relatively ductile failure when tested under deformation controlled conditions.

An energy absorbing 'stiff' testing facility was developed to investigate the shear ductility of reinforced concrete beams, and the details of the facility are described later and also in Ref. 26. In this paper, the results of an experimental investigation of shear-critical reinforced lightweight concrete beams using the 'stiff' testing facility are described. The results indicate that shear critical beams, when tested in an energy absorbing 'stiff' testing facility, exhibit a stable post-peak portion of the load versus mid-span deflection curve. The post-peak deformation characteristics can be quantified by the shear ductility index, which is defined later in the paper. For the range of variables tested, the results indicate that shear (web) reinforcement improves the shear ductility index of shear critical reinforced concrete beams of normal as well as high strength lightweight concrete and that shear ductility index decreases with increasing concrete strength.

RESEARCH SIGNIFICANCE

Knowledge of the post-peak deformation characteristics of shear-critical reinforced concrete members of lightweight normal strength, as well as high strength concrete, is very desirable to better understand the contribution of the shear (web) reinforcement and the failure mechanisms in situations such as under seismic conditions, where higher ductility demands are placed on reinforced concrete members. In this paper, complete and stable load versus mid-span deflection curves were obtained. The post-peak deformation characteristics can be quantified by the shear ductility index, which is defined later in the paper. For the range of variables tested, the results indicate that shear (web) reinforcement improves the shear ductility index of shear critical reinforced concrete beams of normal as well as high strength lightweight concrete and that shear ductility index decreases with increasing concrete strength.

EXPERIMENTAL PROGRAM

A total of 15 shear-critical reinforced LWA concrete beams with normal strength as well as high strength concrete were tested. The test variables were the concrete strength (f'_c), the shear span–depth ratio (a/d) and the amount of the shear reinforcement (γ_W). The concrete strength was varied between 30·5 MPa (4430 psi) and 89·3 MPa (12 950 psi). The shear span–depth ratio was varied between 0 and 0·784%. The summary of the test program is given in Table 1.

Materials

The mixture proportions for normal and high strength concrete are presented in Table 2(a) and 2(b). Type 1 Portland cement was used with natural sand having a fineness modulus of 2·62. The coarse, lightweight aggregate used was expanded slate with a maximum size of 12·5 mm (0·5 in). The workability of the mix was improved by using a napthalene based high-range water-reducing admixture (superplasticizer).

Grade 60 steel was used for all longitudinal (flexural) reinforcement. The stirrups used were #2 smooth bars of grade 40. Note that although the ACI 318-89 Code does not permit the use of smooth bars nor #2 bars, they were used considering the relatively small cross-sectional size of the test beams. The average yield strength of three coupon specimens tested in tension was 421 MPa (61 ksi) for the longitudinal reinforcement and 324 MPa (47 ksi) for the stirrup reinforcement.

For beams with shear reinforcement, strain gages were mounted on selected stirrups in the shear span to monitor the strains during the testing. All the gages had a resistance of 120 Ω and

Table 1. Test program for reinforced lightweight concrete beams with and without shear reinforcement

Beam number No.	*Width of beams (mm)*	*Effective depth (mm)*	*a/d*	*Strength of concrete at testing (MPa)*	*Age at testing (days)*	*Flexural tensile steel A_s (mm²)*	*ρ (%)*	*ρ/ρ_b*	*Flexural comp. steel A'_s (mm²)*	*ρ' (%)*	*ρ'/ρ_b*	*Spacing of #2 stirrups (mm)*	*Shear reinf. ratio, ρ_w (%)*
LNN-1†	127	216	1	33·79	143	258 (2#4)	0·94	0·29	0	0	0		0
LNN-2	127	216	2	44·83	136	258 (2#4)	0·94	0·21	0	0	0		0
LNN-3	127	216	3	40·34	150	258 (2#4)	0·94	0·24	0	0	0		0
LNW-1	127	216	1	30·55	151	567 (2#6)	2·07	0·70	258 (2#4)	0·94	0·31	101	0·49
LNW-2	127	216	2	38·97	133	567 (2#6)	2·07	0·55	258 (2#4)	0·94	0·31	101	0·49
LNW-3	127	216	3	44·62	142	567 (2#6)	2·07	0·48	258 (2#4)	0·94	0·31	101	0·49
LHN-1	127	216	1	86·90	107	567 (2#6)	2·07	0·30	0	0	0		0
LHN-2	127	216	2	85·45	76	567 (2#6)	2·07	0·31	0	0	0		0
LHN-3	127	216	3	89·17	103	567 (2#6)	2·07	0·29	0	0	0		0
LHW-1	127	198	1	82·34	85	1135 (4#6)	4·54	0·70	258 (2#4)	1·03	0·23	99	0·51
LHW-2‡	127	198	2	85·79	82	1135 (4#6)	4·54	0·67	258 (2#4)	1·03	0·29	99	0·51
LHW-3	127	198	3	89·31	76	1135 (4#6)	4·54	0·64	258 (2#4)	1·03	0·23	99	0·51
LHW-3a	127	198	3	88·21	85	1135 (4#6)	4·54	0·65	258 (2#4)	1·03	0·23	76	0·65
LHW-3b	127	198	3	86·97	72	1135 (4#6)	4·54	0·66	258 (2#4)	1·03	0·23	63	0·78
LHW-4	127	198	4	82·97	66	1135 (4#6)	4·54	0·69	258 (2#4)	1·03	0·23	99	0·51

†LNN-1.
- shear span to depth (a/d) ratio
- no web reinforcement
- normal strength
- lightweight concrete

‡LHW-2.
- shear span to depth (a/d) ratio
- web reinforcement
- high strength
- lightweight concrete

an active grid length of 6·25 mm (0·25 in). After mounting, the strain gages were covered with a protective coating to prevent damage during and after casting.

Table 2. Mixture proportions

(a) Lightweight normal strength concrete

Material	*Quantity*
Type 1 cement	344·40 kg m^{-3}
Sand (Lillington, NC)	759·29 kg m^{-3}
Coarse lightweight aggregate	516·28 kg m^{-3}
Retarder (PSI-400R)	441·43 ml m^{-3}
AEA† (Daravair-R)	117·60 ml m^{-3}
Water	230·19 kg m^{-3}

†Air entraining agent.

(b) Lightweight high strength concrete

Material	*Quantity*
Type 1 cement	509·88 kg m^{-3}
Sand (Lillington, NC)	889·36kg m^{-3}
Coarse lightweight aggregate	430·10 kg m^{-3}
Silica fume	88·7 kg m^{-3}
HRWR† (Naphthalene based)	3 118·3 ml m^{-3}
AEA‡ (Daravair-R)	161·7 ml m^{-3}
Water	135·52 kg m^{-3}

†High-range water reducing agent (naphthalene).
‡Air entraining agent.

Conversion: 1 lb/cf = 16·03 kg m^{-3}; 1 ml/cf = 35·31 ml m^{-3}.

Casting and curing

The forms were made of 19 mm (0·75 in) thick plywood and were re-used. The concrete was placed in two layers in the beam and was internally vibrated. Along with each batch, six 102 × 204 mm (4 × 8 in) cylindrical specimens, referred to hereafter as 'control' cylinders, were also cast. Immediately after casting, the beams and the 'control' cylinders were covered with a polyethylene sheet to avoid escape of moisture. Twenty-four hours after casting, the beams and their respective 'control' cylinders were stripped. The beams and their 'control' cylinders were moved for curing to a moist room with a temperature of 70–75°F and 100% relative humidity.

Specimen details and testing

The summary of the testing program and the specimen details are presented in Table 1 and Fig. 1. All the beams were 127 mm (5 in) wide and 254 mm (10 in) deep.

The 'control' cylinders were tested periodically to ascertain if the concrete had reached the desired strength. One day before testing, the beams and their respective 'control' cylinders were taken out of the moisture room and allowed to dry. On the day of the beam test, three of the 'control' cylinders were capped and tested in compression to determine the strength of the concrete.

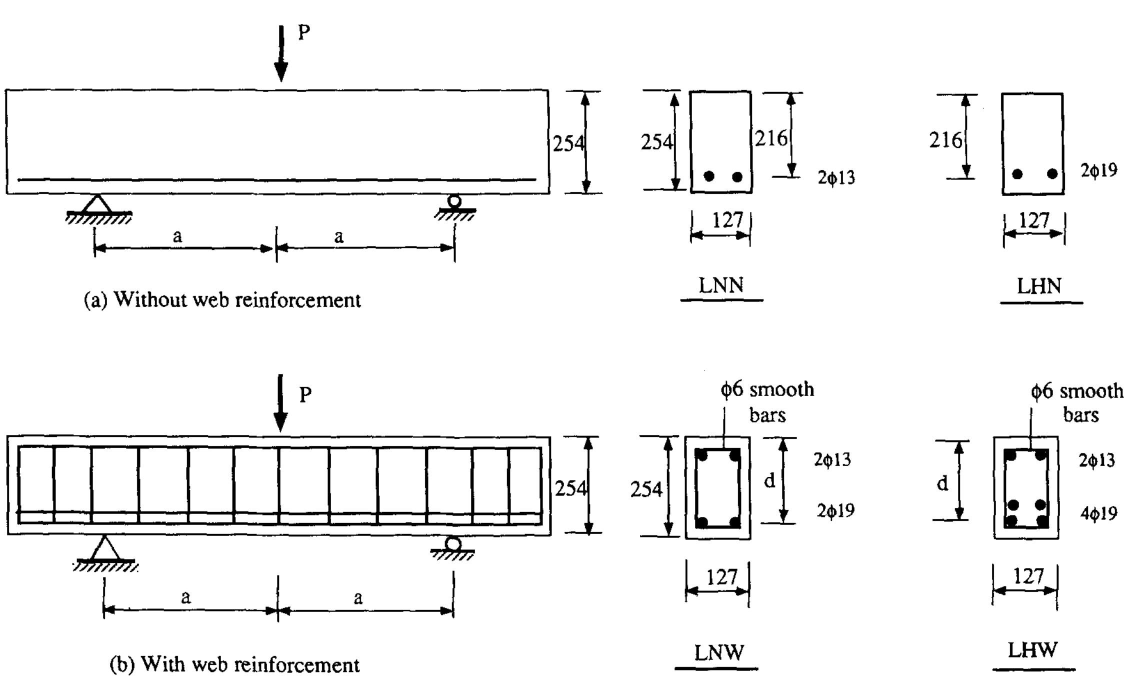


Fig. 1. Specimen details for lightweight normal and high strength concrete beams (all dimensions are in mm).

In order to obtain a stable post-peak behavior, it is essential to absorb the energy which is released from the loading machine during unloading of the test specimen that occurs due to the rapid propagation of the diagonal tension crack after the peak load. An energy absorbing 'stiff' testing facility was developed for obtaining a stable and controlled diagonal tension failure of shear-critical reinforced concrete beams.

To develop an energy absorbing 'stiff' for testing shear-critical reinforced concrete members, stiffening elements were added to an existing testing facility; rigid supports with a stiff cross steel beam were employed to act as stiffener and energy absorbing element in the test facility. The test set-up with rigid supports of an ultra high strength concerete and cross steel beam (W12 × 35) is shown in Fig. 2. Both the cross steel beam and the concrete test specimen are subjected to a single load at mid-span. The span of the cross steel beam is fixed to be 610 mm (24 in). The steel cross beam and the test specimen share the total load (P_t) from the loading jack. The stiffness, the maximum elastic load limit and the corresponding deflection of the cross steel beam are $K_S = 5210$ kN mm^{-1}, $P_{se} = 845$ kN and $\delta_{max} = 0{\cdot}16$ mm. Note that the maximum load capacity of the testing facility limits the stiffness of the cross steel beam. The ultra high strength rigid concrete supports were fabricated with concrete having a 28 day cylinder strength of 124 MPa. The cross steel beam and the concrete test specimen act as parallel springs, since the supports are rigid (Fig. 2). The deflection compatibility at Point A in the test set-up requires that mid-span deflection of the concrete test beam is the same as that of the cross steel at the mid-span (Fig. 2(b)). This can be written as $\delta_c = \delta_{cb}$. From this deflection compatibility condition and by assuming that the testing system is to be designed for a maximum deflection capability of 2·54 mm (1 in), the estimated elastic load (P_s) to be experienced by the steel cross beam can be computed. Note that the maximum allowable deflection of the cross steel beam has to be within the elastic range of the cross steel beam and limits the deflection up to which the concrete test beam can be tested. Therefore, in order to test the concrete test beam up to larger deflections in the post-peak region, the hydraulic jacks in the facility (Fig. 2) must be used to reduce the net deflection of the steel cross beam so as to accommodate the deflection increment in the subsequent increase of the total load (P_t). Since the stiffness of the cross steel beam is much greater than that of the concrete test beam ($K_s = 5210$ kN mm$^{-1} > 2\ K_c$), the testing facility is stable. Hydraulic jacks between the ultra high strength concrete rigid suports and the cross steel beam were used to initiate the contact of the rigid supports with the cross steel beam. Once this is done, then further increase in the total load (P_t) is shared by the concrete test specimen and the steel cross beam supported by rigid supports. This operation of load sharing between the concrete test specimen and the steel cross beam can be done at any stage in the loading prior to reaching the peak load of the concrete test specimen. After the peak load of the specimen, the energy release of the test specimen is absorbed by the cross steel beam supported by the rigid supports. After the peak load, to increase the mid-span deflection of the concrete test specimen, the total load (P_t) is

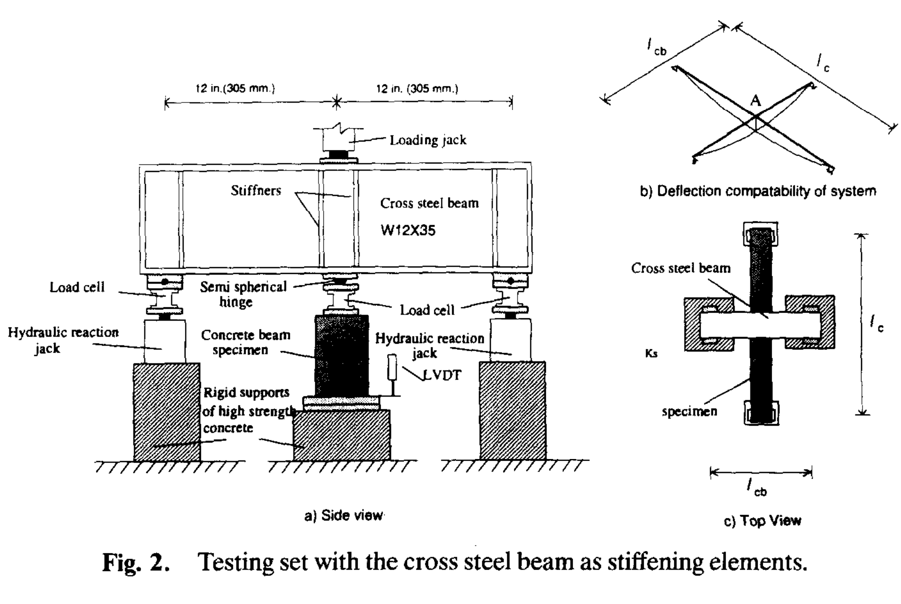


Fig. 2. Testing set with the cross steel beam as stiffening elements.

increased which results in increased loads experienced by the two hydraulic jacks at the end supports of the cross steel beam, but the load carried by the concrete test beam (P_c) decreases. Schematic load-deflection curves for the test specimen and the cross steel beam are shown in Fig. 3.

The reinforced concrete beams were loaded with a central point load in the 'stiff' testing facility with 1780 N (400 kips) capacity (Fig. 2). The total load on the test facility was monotonically increased. The tests were conducted by increasing the total load (P_t) from the loading jack at each stage of loading. The load was applied at a constant deflection rate throughout the entire test of each beam. The reinforced concrete test beams experienced nearly an equal increment of deflection in the pre-peak region as well as in the post-peak region of the load-mid-span deflection response. The deflection compatibility at Point A in the test set-up requires that the mid-span deflection of the reinforced concrete test beam be the same as that of the cross steel beam at the mid-span (see Fig. 2(b)). From this deflection compatibility condition, and noting that the maximum allowable deflection of the steel cross beam has to be within the elastic range of the cross steel beam, it can be seen that the deflection up to which the reinforced concrete test beam can be tested is limited, if the steel cross beam shares the portion of the total load from the start of the test. Therefore, the reinforced concrete beams were loaded up to about 60-70% of the anticipated maximum load, before the hydraulic reaction jacks were used to initiate the contact of the rigid supports with the cross steel beam. Once this was done, then further increase in the total load (P_t) from the loading jack was shared by the test specimen and the cross steel beam supported by the rigid supports. The hydraulic jacks were also used in the post-peak region of the load-mid-span deflection curve, in order to reduce the net deflection on the steel cross beam so as to accommodate the deflection increment in subsequent increments of the total load (P_t).

Three load cells were used for monitoring the load on the test beams and the load experienced by the hydraulic reaction jacks supported on the rigid supports. A linear voltage differential transducer (LVDT) and strain gages were used to monitor the vertical deflection at mid-span and the strains in the stirrups. The outputs from the load cells, LVDT and the strain gages were continuously recorded by use of a personal computer and an OPTIM Megadec 100 data acquisition system. During the tests, the cracking pattern was also monitored. The crack pattern and the results of the stirrup strains are described in Ref.27

RESULTS OF EXPERIMENTAL INVESTIGATION

The test results are presented in Tables 3-5. The results in Table 3 include the observed values for diagonal cracking load, the maximum (ultimate) load and the type of failure observed. Information on the shear ductility and the effectiveness of shear (web) reinforcement is presented in Tables 4 and 5. The experimental results are presented in two categories; the load-deflection behavior and the shear ductility.

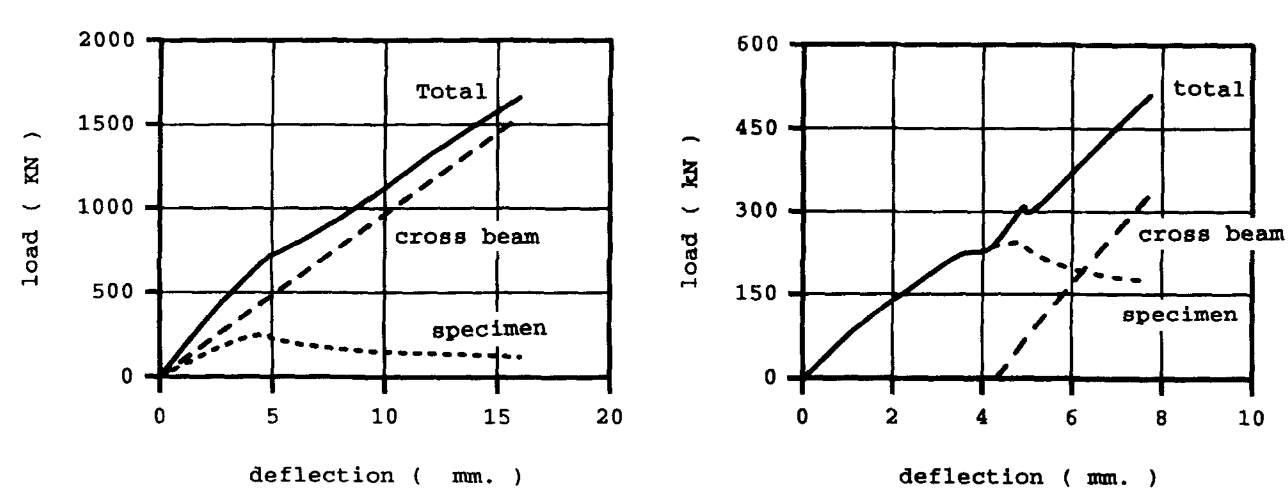


Fig. 3. Schematic load-deflection curves for the test specimen and the steel cross beam. (a) Reaction jacks are activated at the start of the test. (b) Reaction jacks are activated near the maximum load capacity of the specimen.

Table 3. Diagonal cracking and ultimate loads for reinforced lightweight normal and high strength concrete beams

Beam No.	*a/d*	f'_c† *(MPa)*	f'_c‡ *(MPa)*	*Crack load* P_{cr}§ *(kN)*	*Maximum (ultimate) load* P_u *(kN)*	*Failure type*
LNN-1	1	33·79	32·14	80·01	211·72	Diagonal compression
LNN-2	2	44·83	42·62	42·43	66·75	Shear-compression
LNN-3	3	40·34	38·34	31·50	45·30	Diagonal tension
LNW-1	1	30·55	29·03	82·41	299·83	Diagonal compression
LNW-2	2	38·97	37·03	53·26	168·38	Shear-compression
LNW-3	3	44·62	42·41	48·90	126·60	Shear-compression
LHN-1	1	86·90	82·55	190·80	377·17	Diagonal compression
LHN-2	2	85·45	81·17	65·28	170·87	Shear compression
LHN-3	3	89·17	84·69	52·69	86·82	Diagonal tension
LHW-1	1	82·34	78·1	218·10	554·95	Diagonal compression
LHW-2	2	85·79	81·52	97·50	270·95	Shear-compression
LHW-3	3	89·31	84·83	70·49	184·93	Shear-compression
LHW-3a	3	88·21	83·79	76·58	214·13	Shear-compression
LHW-3b	3	86·97	82·62	90·55	241·45	Shear-compression
LHW-4	4	82·97	8·83	88·15	189·70	Shear-compression

†From 101 × 202 mm cylinder at the test age.
‡Using equivalent 152 × 303 mm cylinder strength taken as 95% of 101 × 202 mm cylinder strength.[8]
§Load at which the diagonal tension crack crosses the mid-height of the beam.

Table 4. Shear ductility of reinforced lightweight normal and high strength concrete beams

Beam No.	*a/d*	f'_c *(MPa)*	ρ *(%)*	ρ/ρ_b	ρ'/ρ_b	γ_W	P_u *(%)*	Δ_0 *(kN)*	μ† *(mm)*
LNN-1	1	33·79	0·94	0·29	0	0	211·72	2·26	3·24
LNN-2	2	44·83	0·94	0·21	0	0	66·75	2·11	2·62
LNN-3	3	40·34	0·94	0·24	0	0	45·30	4·14	2·69
LNW-1	1	30·55	2·07	0·70	0·31	0·49	299·83	3·43	3·60
LNW-2	2	38·97	2·07	0·55	0·31	0·49	168·38	6·30	3·21
LNW-3	3	44·62	2·07	0·48	0·31	0·49	126·60	10·59	4·20
LHN-1	1	86·90	2·07	0·30	0	0	377·17	1·98	3·06
LHN-2	2	85·45	2·07	0·31	0	0	170·87	4·60	2·21
LHN-3	3	89·17	2·07	0·29	0	0	86·82	6·58	2·31
LHW-1	1	82·34	4·54	0·70	0·23	0·51	554·94	2·31	3·13
LHW-3	2	85·79	4·54	0·67	0·23	0·51	270·95	3·94	3·05
LHW-3	3	89·31	4·54	0·64	0·23	0·51	184·93	8·23	3·21
LHW-3a	3	88·21	4·54	0·65	0·23	0·65	214·13	9·65	4·01
LHW-3b	3	86·97	4·54	0·66	0·23	0·78	241·45	10·51	3·78
LHW-4	4	82·97	4·54	0·69	0·23	0·51	189·70	11·33	3·73

†μ = Ratio of the area of the load–deflection response up to $3\Delta_0$ to the area up to Δ_0. The deflection corresponding to P_{max} is Δ_0.

Load–deflection behavior

The load–mid-span deflection curves for all the 15 test beams of normal, as well as high strength, lightweight concrete with different shear span–depth ratio (a/d) and different shear reinforcement ratio (γ_W) are shown in Figs 4–6.

The effect of the presence of shear (web) reinforcement on load versus mid-span deflection for reinforced LWA normal strength concrete beams with shear span–depth ratio (a/d) varying from 1 to 3 is shown in Fig. 4. From this figure, it can be seen that for normal strength LWA concrete beams, as the shear span–depth ratio (a/d) increases from 1 to 3, the effectiveness of the shear reinforcement in improving the ultimate load capacity and the post-peak deformation characteristics increases. Note that the beams LNW-1, LNW-2 and LNW-3 had the same amount of shear reinforcement (γ_W = 0·49%). A similar tend is also observed for beams with LWA high strength concrete (Fig. 5). The beams LHW-1, LHW-2 and LHW-3 had the same amount of shear reinforcement (γ_W = 0·51%). The results (Fig. 4(c)) indicate that it is possible to achieve an elasto-plastic behavior for beams with normal strength LWA concrete when reinforced with

Table 5. Efficiency of shear reinforcement for reinforced lightweight normal and high strength concrete beams

Beam No.	*a/d*	f'_c *(MPa)*	ρ *(%)*	ρ/ρ_b	ρ'/ρ_b	γ_W *(%)*	†*Energy absorption ratio*
LNW-1	1	30·55	2·07	0·70	0·31	0·49	2·05
LNW-2	2	38·97	2·07	0·55	0·31	0·49	9·36
LNW-3	3	44·62	2·07	0·48	0·31	0·49	10·97
LHW-1	1	82·34	4·54	0·70	0·23	0·51	1·66
LHW-2	2	85·79	4·54	0·67	0·23	0·51	2·01
LHW-3	3	89·31	4·54	0·64	0·23	0·51	3·44
LHW-3a	3	88·21	4·54	0·65	0·23	0·65	5·26
LHW-3b	3	86·97	4·54	0·66	0·23	0·78	6·28

†Energy absorption ratio is the area of the load–deflection response up to $3\Delta_0$ for beams with web reinforcement to the area of the load–deflection response up to $3\Delta_0$ for beams without web reinforcement with the same shear span–depth ratio. Δ_0 is the deflection corresponding to P_{max} for the individual beam.

shear reinforcement when it is about five times the minimum recommended by Section 11.1.2.1 of the ACI 318-89 Code.[28]

The effect of increasing the shear reinforcement ratio (γ_W) on the load versus mid-span deflection response of shear-critical reinforced LWA high strength concrete beams is shown in Fig. 5(c). Beams LHW-3, LHW-3a and LHW-3b had shear reinforcement ratios of 0·51, 0·65 and 0·78%, respectively. It can be seen that increasing the shear reinforcement ratio increases the load capacity and decreases the slope of the load versus mid-span deflection curves in the post-peak region. The figure also shows that for the high strength LWA beams, it is not possible to achieve a plastic post-peak response, even when the shear reinforcement provided is about three

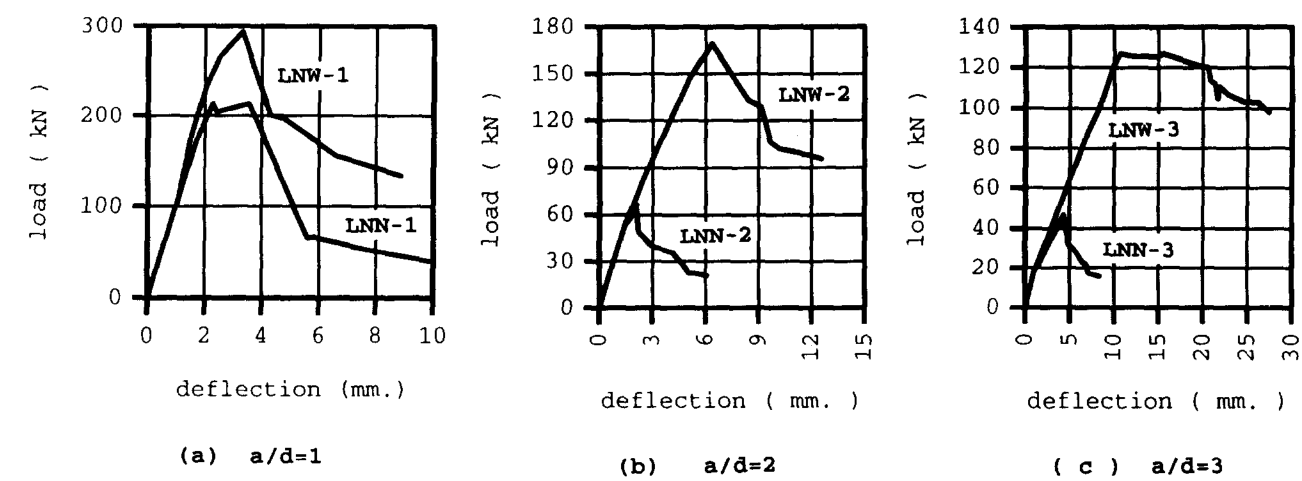


Fig. 4. Effect of web reinforcement on load–mid-span deflection of lightweight normal strength concrete beams.

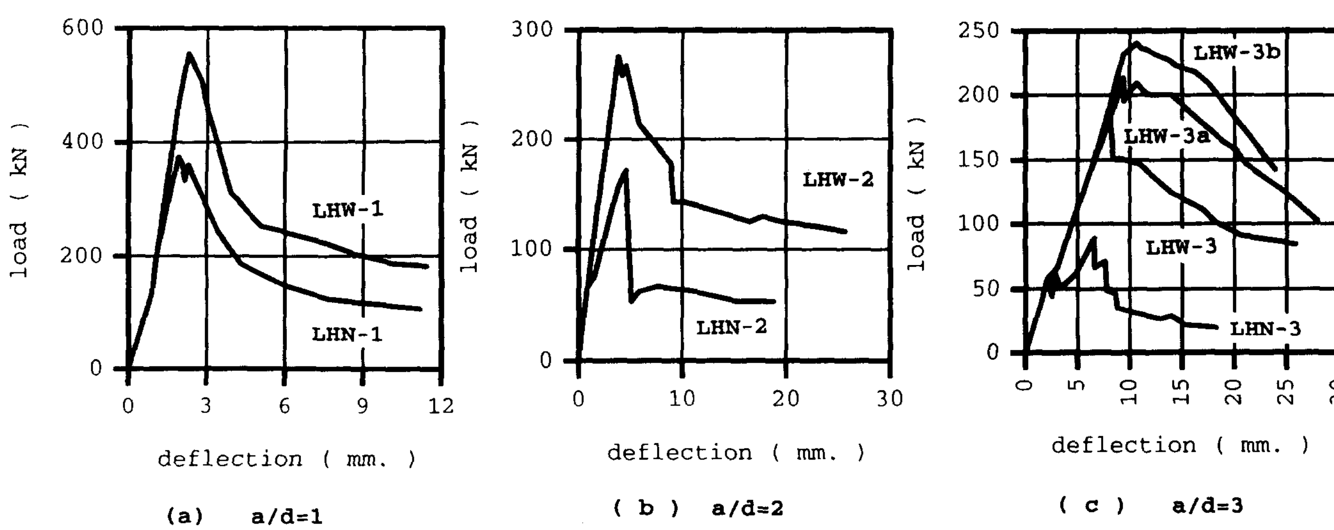


Fig. 5. Effect of web reinforcement on load–mid-span deflection of lightweight high strength concrete beams.

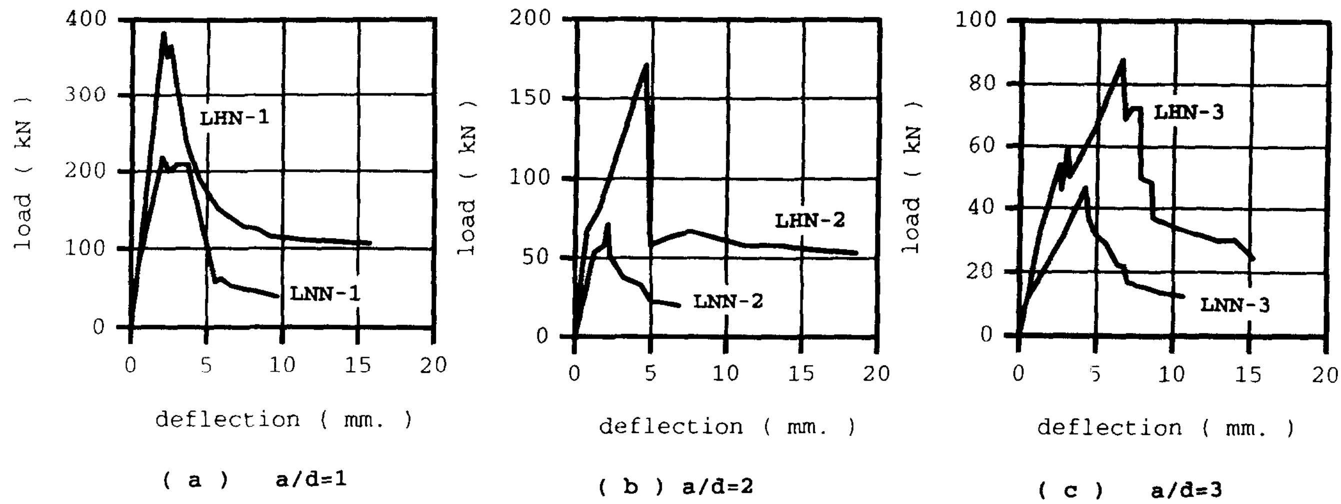


Fig. 6. Effect of concrete strength on load–mid-span deflection for lightweight concrete beams without web reinforcement.

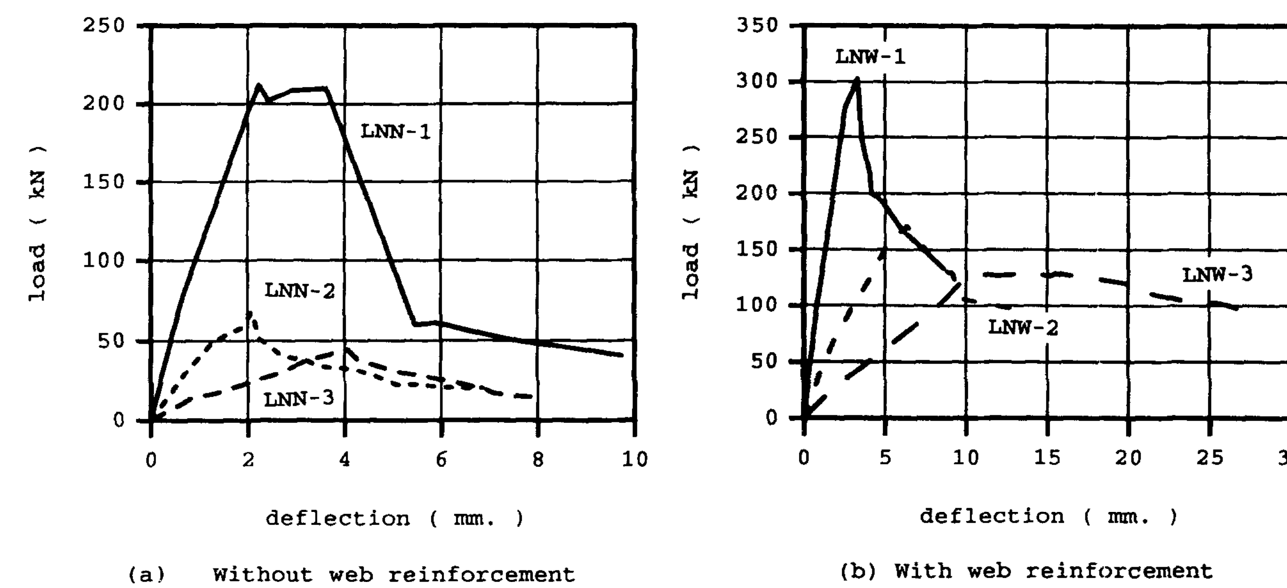


Fig. 7. Effect of shear span–depth ratio (a/d) for lightweight normal strength concrete beams.

times the minimum required by Section 11.1.2.1 of the ACI 318-89 Code.[28]

The effect of the concrete strength on load versus mid-span deflection for reinforced LWA concrete beams without web reinforcement is shown in Fig. 6. From this figure, it can be seen that the slopes of the descending portion of load–deflection curves for high strength LWA concrete beams are steeper than those for normal strength LWA concrete beams.

The effect of shear span–depth (a/d) ratio on load–mid-span deflection for reinforced lightweight normal strength and high strength concrete beams with and without web reinforcement is shown in Figs 7 and 8. It can be seen that for all the beams, the slope of the pre-peak region as well as the post-peak region is steeper for a/d of 1 as compared to beams with a/d of 2, 3 and 4.

Shear ductility

Member ductility can be broadly defined as 'the ability of the member to withstand load while incurring additional deformation beyond the maximum load stage'. This definition is qualitative and to quantify the deflection ductility of shear-critical reinforced concrete members, the shear ductility index (μ) was defined in this study as 'the ratio of the area of the load–deflection response up to $3\Delta_0$ to the area up to Δ_0' (Fig. 9). Note that, although shear ductility should be measured on

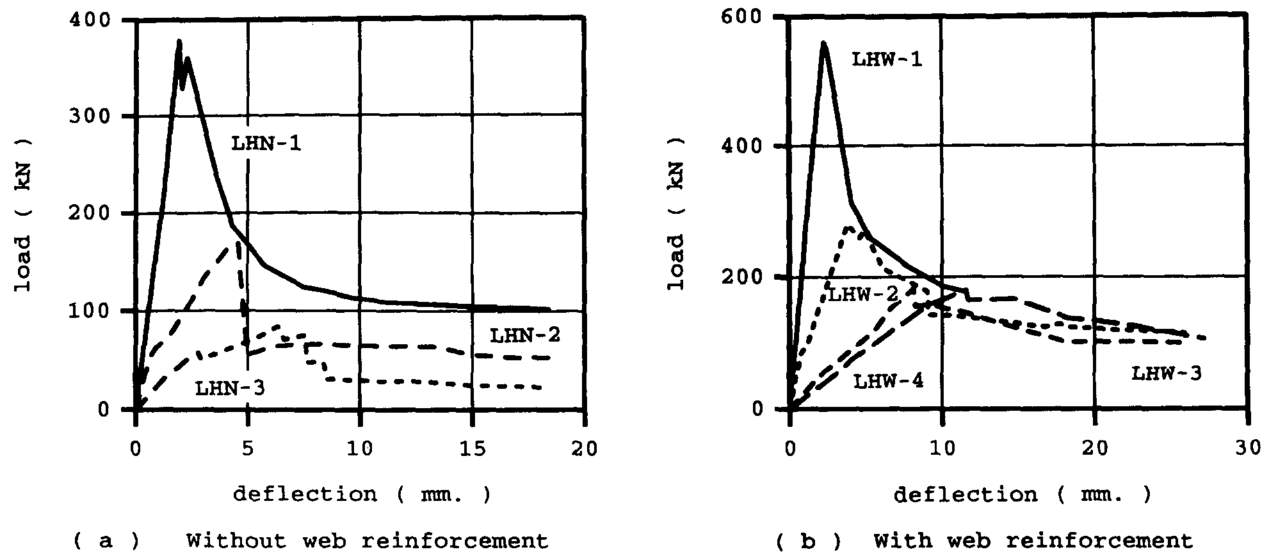


Fig. 8. Effect of shear span–depth ratio (a/d) for lightweight high strength concrete beams.

shear deformation only, in this study complete shear and flexure deformations are used to define shear ductility. The values of shear ductility reported in this paper are strictly the deflection ductility of shear-critical beams. Note that Δ_0 is the deflection corresponding to the maximum load (P_{max}) for each individual's reinforced concrete beam. The shear ductility indices were computed from the test results of the 15 test beams, and the results are presented in Table 4. Also shown in the Table 4 are the maximum loads (P_{max}) and the corresponding deflections (Δ_0) recorded during the tests.

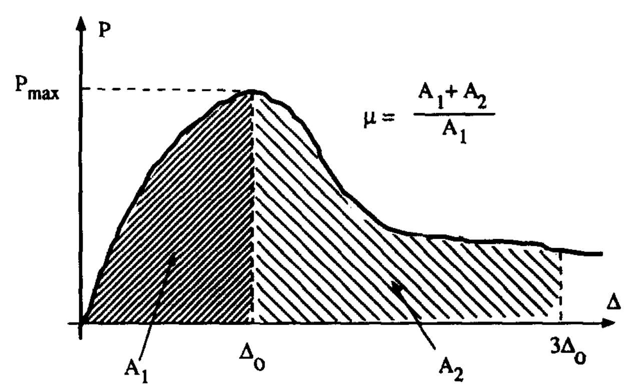


Fig. 9. Schematic diagram for definition of deflection ductility ratio (μ).

Influence of compressive strength

The influence of the compressive strength of concrete on the shear ductility index (μ) of beams without and with shear reinforcement are shown in Figs 10(a) and 10(b). Figure 10(a) shows that for beams without shear reinforcement and with a/d of 1, 2 and 3, the shear ductility index decreases with the increase in the concrete strength. A similar trend is also observed for beams with shear reinforcement and a/d of 1, 2 and 3 (Fig. 10(b)).

Influence of shear span–depth ratio

The influence of shear span–depth ratio on the shear ductility index is shown in Table 4 and Figs 10(a) and 10(b). It can be seen that for all the test beams, the shear ductility index (μ) for beams with a/d of 2 is lower than those for beams with a/d of 1 and 3. This can be attributed to the different failure mechanisms for beams with a/d of 1 and 3.

Influence of shear reinforcement ratio

The influence of the shear reinforcement ratio on the shear ductility index (μ) is shown in Fig. 11. From this figure, it can be seen that increasing the shear reinforcement ratio (γ_W) up to 0·51% has an insignificant effect on shear ductility of beams with a/d of 1. However, for beams with a/d of 2 and 3, the shear ductility index increases with increasing shear reinforcement. For beams with a/d of 3, increasing the shear reinforcement ratio from 0·51 to 0·65% increases the shear ductility index by 25%. Further increase in the shear reinforcement ratio does not increase the shear ductility index.

Although the shear ductility index indicates an improvement in the post-peak behavior of the shear-critical reinforced LWA concrete beams due to the presence of the shear (web) reinforcement, it does not demonstrate the effectiveness of

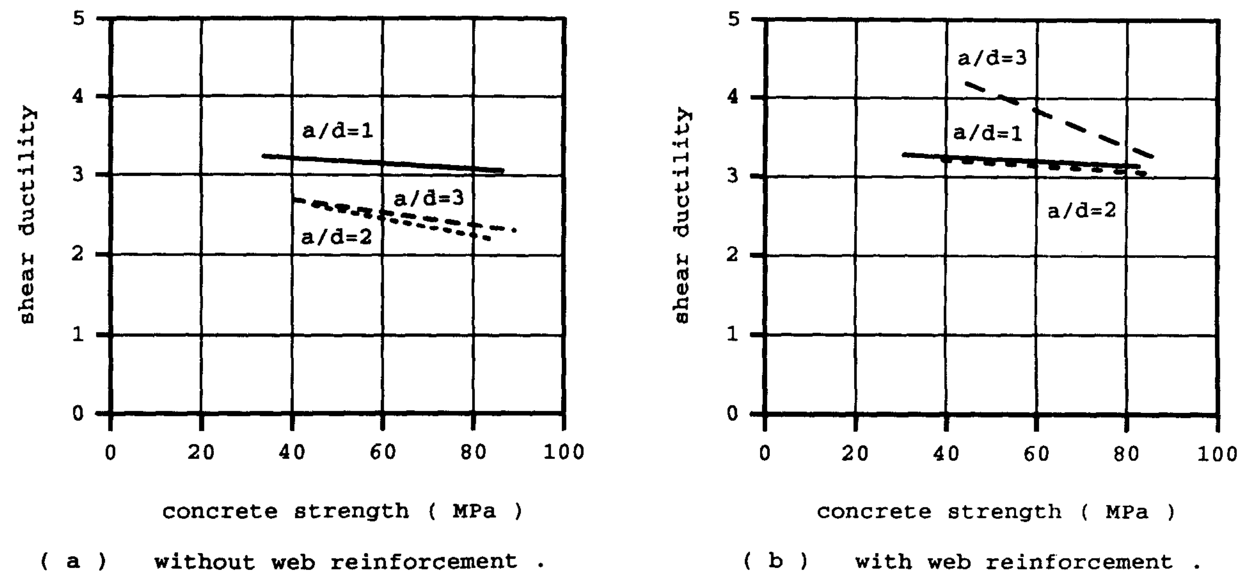


Fig. 10. Effect of concrete strength on shear ductility of lightweight concrete beams.

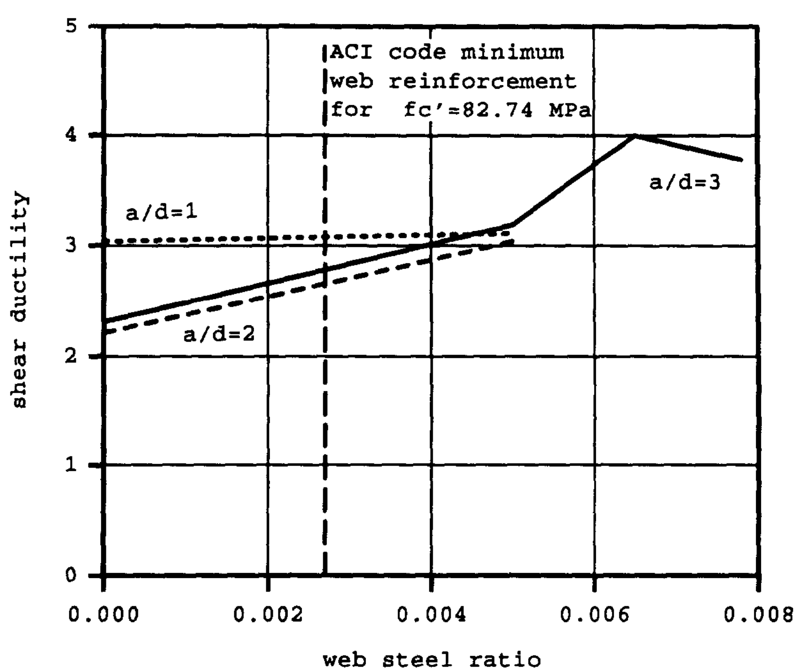
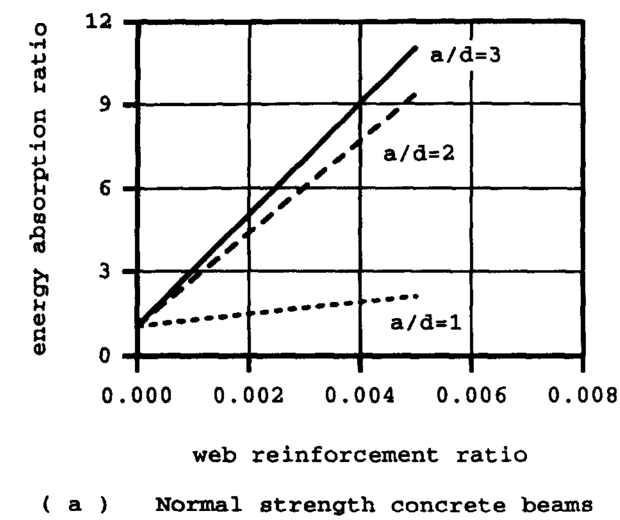


Fig. 11. Effect of web steel ratio on shear ductility for lightweight high strength concrete beams.

the shear stirrup reinforcement in increasing the ultimate load capacity and the post-peak deformability of the shear critical beams. The effectiveness of the shear reinforcement in improving the ultimate load capcity and the post-peak deformability can be more clearly demonstrated by computing the 'energy absorption' ratio for each of the beams with shear reinforcement. The 'energy absorption' ratio is defined as the ratio of the energy absorbed by a beam with shear (web) reinforcement to the energy absorbed by a similar beam without shear (web) reinforcement. The computed 'energy absorption' ratios for all the beams with shear reinforcement is presented in Table 5. The 'energy absorption' ratio for each beam with shear reinforcement was computed by calculating the total area under the load versus mid-span deflection up to $3\Delta_0$ and dividing this by the area under the load–deflection curve up to $3\Delta_0$ for a similar beam without shear reinforcement. Note that Δ_0 is the deflection corresponding to the maximum load for each individual beam.



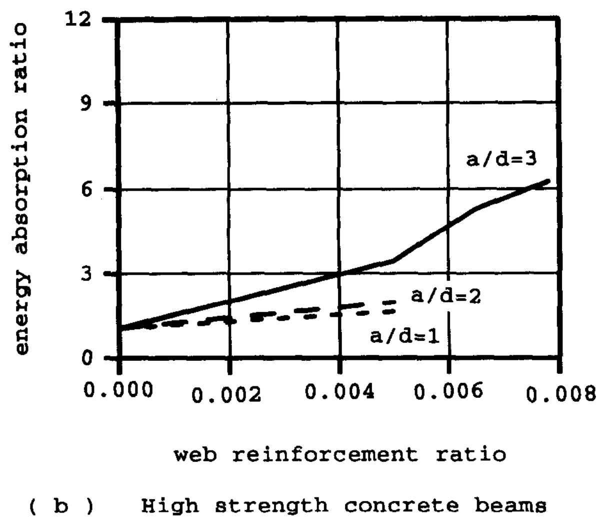


Fig. 12. Effectiveness of shear reinforcement on the energy absorption capacity for reinforced lightweight concrete beams.

The effect of amount of shear reinforcement ratio on the 'energy absorption' ratio is shown in Fig. 12, which shows that the 'energy absorption' ratio increases with increase in a/d ratio. This indicates that the effectiveness of the shear reinforcement in improving the ultimate load capacity and the post-peak deformation characteristics increases with an increase in a/d ratio. Figure 12 also shows that for beams with a/d of 1, the effectiveness of the shear reinforcement in increasing the energy absorption capacity is similar in LWA beams with normal strength as well as high strength concrete. For beams with a/d of 2 and 3, the stirrup reinforcement seems to be more effective for the beams with LWA normal strength concrete as compared to the beams with LWA high strength concrete. However, it should be noted that, although the tensile steel reinforcement ratios for the normal strength concrete beams were much lower than those for the high strength concrete beams, the ratios (ρ/ρ_b) were quite comparable. It is recognized that the post-cracking apparent lateral dilation of higher strength concretes is relatively smaller than for normal strength concretes[3] and, since the lateral dilation of concrete is an important parameter which governs the degree of the usefulness of the confining stirrups, it appears that for higher strength LWA concrete beams, more stringent shear reinforcement requirements may be needed. The figure also shows that for high strength LWA concrete beams with a/d of 3, increasing the shear reinforcement ratio from 0·51% to 0·65% increases the 'energy absorption' ratio by 50%, however further increase in the shear reinforcement ratio (up to 78%) results in only a 20% increase in the 'energy absorption' ratio.

SUMMARY AND CONCLUSIONS

Experimental results of shear-critical reinforced lightweight normal strength as well as high strength concrete beams without and with shear (web) reinforcement are presented. On the basis of results obtained in this study, the following conclusions can be drawn:

(1) For LWA concentrate beams, with or without shear reinforcement, the shear ductility index (μ) decreases with an increase in the concrete strength. The effect is more pronounced for beams with a/d of 3 as compared to beams with a/d of 1 or 2.

(2) The LWA normal strength concrete beams with a/d of 3 exhibited a near plastic post-peak response, when the shear reinforcement provided was about five times that of the minimum amount required by Section 11.1.2.1 of the ACI 318-89 Code.

(3) The post-peak deformation characteristics, expressed in terms of shear ductility index (μ), indicates that beams with a/d of 2 exhibit lower values as compared to beams with a/d of 1 and 3.

(4) Increasing the shear reinforcement ratio (γ_W) up to 0·51% has an insignificant effect on the shear ductility index of beams with a/d of 1. However, for beams with a/d of 2 and 3, the shear ductility index increases.

For beams with a/d of 3, increasing the shear reinforcement ratio from 0·51 to 0·65% increases the shear ductility index by 25%. Further increase in the shear reinforcement ratio does not increase the shear ductility index.

(5) The effectiveness of the shear reinforcement in improving the load capacity and the post-peak deformation characteristics, when quantified by the 'energy absorption' ratio, indicates that, for beams with a/d of 3, increasing the shear reinforcement ratio from 0·51 to 0·65% increases the 'energy absorption' ratio by 50%. Further increase in the shear reinforcement ratio (up to 0·78%) increases the 'energy absorption' ratio by only 20%.

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REFERENCES

1. Hoff, C. G., *High Strength Lightweight Aggregate Concrete–Current Status and Future Research Needs.* ACI Special Publication, SP 121-30, pp. 620–43.
2. Cariono, N. J. & Clifton, J. R., *Outline of a National Plan on High Performance Concrete: Report on the NIST/ACI Workshop*, 16–18 May 1990. NISTIR 4465, 1990.
3. Ahmed, S. H. & Shah, S. P., Structural properties of high strength concrete and its implications for precast concrete. *PCI J.*, **30** (1985) 92–119.
4. Shin, S.-W., Ghosh, S. K. & Moreno, J., Flexural ductility of ultra-high-strength concrete members. *ACI. J. Proc.*, **86** (4) (1989) 394–400.
5. Leslie, K. E., Rajagopalan, K. S. & Everard, N. J., Flexural behavior of high strength concrete beams. *ACI J. Proc.*, **73** (9) (1976) 517–21.
6. Pastor, J. A., Nilson, A. H. & Slate, F. O., Behavior of high-strength concrete beams. Research Report 84-3, Department of Structural Engineering, Cornell University, Ithaca, NY, February 1984.
7. Shin, S. W., Kamara, M. & Ghosh, S. K., *Flexural Ductility, Strength Prediction, and Hysteretic Behavior of Ultra-High-Strength Concrete Members.* Special Publication 121, American Concrete Institute, Detroit, MI, 1990, pp. 239–64.
8. Ahmad, S. H. & Barker, R., Flexural behavior of reinforced high-strength lightweight concrete beams. *ACI Structural J.*, **88** (1991) 69–77.
9. Ahmad, S. H. & Batts, J., Flexural behavior of reinforced high strength lightweight concrete beams with web reinforcement. *ACI Structural J.*, **88** (3) (1991) 251–358.
10. Mphonde, A. G. & Frantz, G. C., Shear tests of high- and low-strength concrete beams without stirrups. *ACI. J. Proc.*, **81** (4) (1984) 350–57.
11. Elzanty, A. H., Nilson, A. H. & Slate, F. O., Shear capacity of reinforced concrete beams using high-strength concrete, *ACI J.*, **83** (1986) 290–6.
12. Ahmad, S. H., Khaloo, A. R. & Poveda, A., Shear capacity of high strength concrete beams. *ACI J. Proc.*, **83** (2) (1986) 297–305.
13. ACI Committee 363, State-of-the-art report on high-strength concrete. *ACI J. Proc.*, **81** (4) (1984) 364–412.
14. Roller, J. J. & Russell, H. G., Shear strength of high-strength concrete beams with web reinforcement. *ACI Structural J.*, (1990) 191–8.
15. Salandra, M. A. & Ahmad, S. H., Shear capacity of reinforced lightweight high-strength concrete beams. *ACI Structural J.*, **86** (6)(1989) 697–704.
16. Ivey, D. L. & Buth, E., Shear capacity of lightweight concrete beams. *ACI J. Proc.*, **64** (10) (1967) 634–43.
17. Kong, F.-K. & Robins, P. J., Web reinforcement effects on lightweight concrete deep beams. *ACI J. Proc.*, **68** (7) (1971) 514–20.
18. Clarke, J. L., Shear strength of lightweight aggregate concrete beams: design to BS 8110. *Magazine of Concrete Research*, **39** (141) (1987) 205–13.
19. Hamadi, Y. D. & Regan, P. E., Behavior of normal and lightweight aggregate beams with shear cracks. *The Structural Engineer*, **54B** (4) (1980) 71–9.
20. Clark, J. L., Shear capacity of high strength concrete beams. *Concrete*, **21** (3) (1987) 24–6.
21. Swamy, R. N., Jones, R. & Clark, A. T. P., Influence of steel fibers on shear resistance of lightweight concrete I-beams. *ACI Structural J.*, **90** (1) (1993) 103–14.
22. Ahmad, S. H. & Shah, S. P., Complete stress–strain curve of concrete and non-linear design — Nonlinear design of concrete structures. *CSCE-ASCE-ACI-CEB Int. Symp.*, University of Waterloo, Ontario, Canada, 7–9 August, 1979, pp. 61–81.
23. Bazant, Z. P., Instability, ductility and size effect in strain softening concrete. *J. Engineering Mechanics, ASCE*, **102** (2) (1976) 331–44.
24. Kotsovos, M. D., Effect of testing techniques on the post-ultimate behavior of concrete in compression. *Materiaux et Construction* (RILEM, Paris), **16** (91) (1983) 3–12.
25. Kotsovos, M. D. & Newman, J. B., Plain concrete under load — A new interpretation. *IABSE Colloquium on Advanced Mathematics of Reinforced Concrete*, Delft, Holland, 1981 June 2–4.
26. Ahmad, S. H., Hino, S., Chung, W. & Xie, Y., An experimental technique for obtaining controlled tension failure of shear critical reinforced concrete beams. *Materials and Structures*, **28** (1995) 8–15.
27. Ahmad, S. H., Xie, Y. & Yu, T., Effectiveness of shear reinforcement for normal and high strength concrete beams. *Structural Engineering Rev.* (in press).
28. ACI Committee 318, *Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-89/ACI 318R-89).* American Concrete Institute, Detroit, 1989.