Chapter 4

Test results and discussion

This chapter presents a discussion of the results obtained from eighteen beam specimens tested at the Structural Technology Laboratory of the Technical University of Catalonia. The primary objective of the study was to investigate the behaviour of reinforced high-strength concrete beams failing in shear. Specific objectives and a complete description of the beam specimens and testing procedure are presented in Chapter 3.

A large amount of data were collected during the experimental investigation. Each test usually had more than 15 channels of data acquired from the data acquisition system. Since data were collected every 2-3 seconds, thousands of data sets were stored for each beam specimen. For brevity, it was decided to five a three to six-page review of each test in Annex A.

4.1 Introduction to Experimental Results

Table 4.1 summarises the results of the 18 tests of beam specimens subjected to bending moment and shear. The table gives the main characteristics of each beam specimen, its failure shear strength, and its approximate cracking strength.

Beam	f_c MPa	f_{sp} MPa	b mm	d mm	a/d	Shear reinf.		Long. Reinf.		$V_{failure}$ (KN)	V_{cr} (KN) Aprox.
						Stirrup/spacing mm	ρ_w † MPa	Longitudinal reinforcement	ρ_l		
H50/1	49.9	3.46	200	359	3.01	-	0	2ϕ32	2.24	99.69	95
H50/2	49.9	3.46	200	353	3.06	ϕ6/260	0.577	2ϕ32	2.28	177.64	85
H50/3	49.9	3.46	200	351	3.08	ϕ8/210	1.291	2ϕ32	2.29	242.07	90
H50/4	49.9	3.46	200	351	3.08	ϕ8/210	1.291	2ϕ32 + 1ϕ25	2.99	246.34	110
H50/5	49.9	3.46	200	359	3.01	-	0	2ϕ32 + 6ϕ8	2.24	129.65	85
H60/1	60.8	4.22	200	359	3.01	-	0	2ϕ32	2.24	108.14	104
H60/2	60.8	4.22	200	353	3.06	ϕ6/200	0.747	2ϕ32	2.28	179.74	95
H60/3	60.8	4.22	200	351	3.08	ϕ8/210	1.267	2ϕ32	2.29	258.78	100
H60/4	60.8	4.22	200	351	3.08	ϕ8/210	1.267	2ϕ32 + 1ϕ25	2.99	308.71	-
H75/1	68.9	3.69	200	359	3.01	-	0	2ϕ32	2.24	99.93	99
H75/2	68.9	3.69	200	353	3.06	ϕ6/200	0.747	2ϕ32	2.28	203.94	95
H75/3	68.9	3.69	200	351	3.08	ϕ8/210	1.267	2ϕ32	2.29	269.35	95
H75/4	68.9	3.69	200	351	3.08	ϕ8/210	1.267	2ϕ32 + 1ϕ25	2.99	255.23	100
H100/1	87.0	4.05	200	359	3.01	-	0	2ϕ32	2.24	117.85	117.85
H100/2	87.0	4.05	200	353	3.06	ϕ6/165	0.906	2ϕ32	2.28	225.55	110
H100/3	87.0	4.05	200	351	3.08	ϕ8/210	1.291	2ϕ32	2.29	253.64	110
H100/4	87.0	4.05	200	351	3.08	ϕ8/210	1.291	2ϕ32 + 1ϕ25	2.99	266.53	85
H100/5	87.0	4.05	200	359	3.01	-	0	2ϕ32 + 6ϕ8	2.24	140.09	85

† Calculated using the real yielding stress of the stirrups

Table 4.1: Summary of experimental results

The experimental data are divided into five sections in Annex A. The first page gives an overall summary and also describes details of the specimen, its material properties and reinforcement ratios. Another brief paragraph gives a summary of test observations and a graph shows the overall response of each test specimen.

Secondly, the summary table lists the significant parameters for selected data sets. In some cases the data given by the strain gauges were unreliable. The data from these strain gauges have been omitted from the table. The third section shows the plots of the previous data and the location and designation of the strain gauges and Temposonic© transducers.

The fourth section describes the cracking pattern at different load stages. Finally, the last section summarises the test with two or three pictures. For beams without web reinforcement, and hence with fewer strain gauges, some sections are organised differently.

In general, no important problems occurred during the tests except with specimen H60/4. In this case problems were reported with the hydraulics and by accident the beam specimen was heavily loaded before testing. Fortunately, it was possible to stop the hydraulic actuator quickly and the beam did not collapse, although it was completely cracked. Therefore, during the real tests it was not possible to track the formation of cracks.

4.2 Modes of failure

All beam specimens failed in shear. Nevertheless, beam specimen H60/3 collapsed due to a combination of shear and high longitudinal strain, and shear cracks did not cross the compression zone of the beam.

As was discussed in §2.2 and §2.3, the mode of failure for beams without stirrups is different from that of beams with shear reinforcement. Beams H50/1, H60/1, H75/1 and H100/1 failed suddenly with the appearance of a single shear crack. In general, the higher a beam’s concrete compressive strength, the brisker its failure. Section 4.3 compares the experimental results for these beam specimens.

For beam specimen H75/1 it was not possible to see the formation of a shear crack prior to failure and the crack surface went through aggregates. However, for H50/1 and H60/1, the following behaviour was seen: after the formation of the first shear crack in the web, it developed into a splitting crack in the concrete along the longitudinal reinforcement. Finally, the compression region was crushed, leading to the failure of the specimen. This behaviour was similar to that observed in beam specimens containing longitudinally-distributed reinforcement along the web (§4.5), in which failure was also brittle, yet we were able to track the formation of several diagonal cracks. Beam H100/1, as it will be commented in §4.3, exhibited a different behaviour, showing a very fragile collapse.

On the other hand, beams containing stirrups (§4.4) presented a more ductile response. After the formation of the first shear crack, stirrups started to work and further shear cracks developed. At failure, the compressed top part of the beam was crushed due to the combination of compressive and shear stresses. In the photographs in Figure 4.1 the spalling of the concrete next to the crack prior to the failure can be seen. This spalling was best observed in beams with the highest concrete compressive strength.

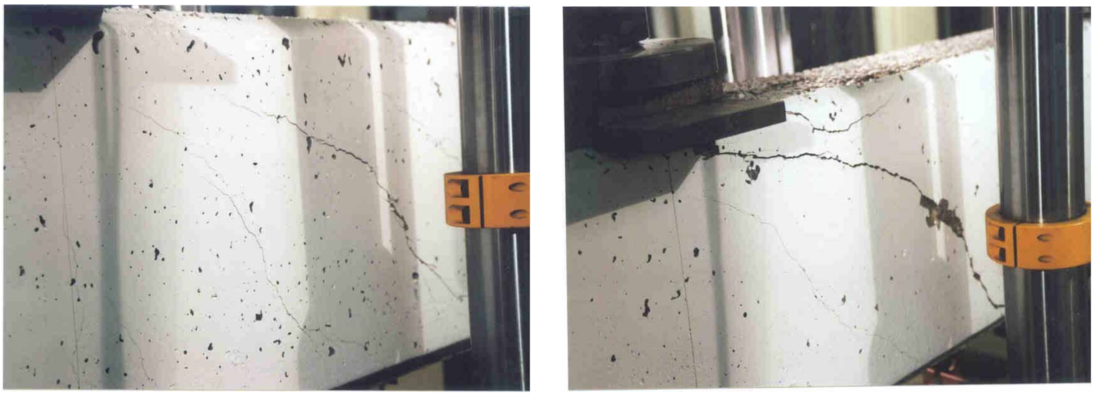

Figure 4.1: Cracking prior to failure and at failure in a beam with web reinforcement. Concrete spalling near the diagonal crack.

In the following sections, the behaviour of the tested beams is studied carefully. To emphasise the ideas described in this section, Figure 4.2 shows the typical failure cracking patterns observed in the experimental campaign.

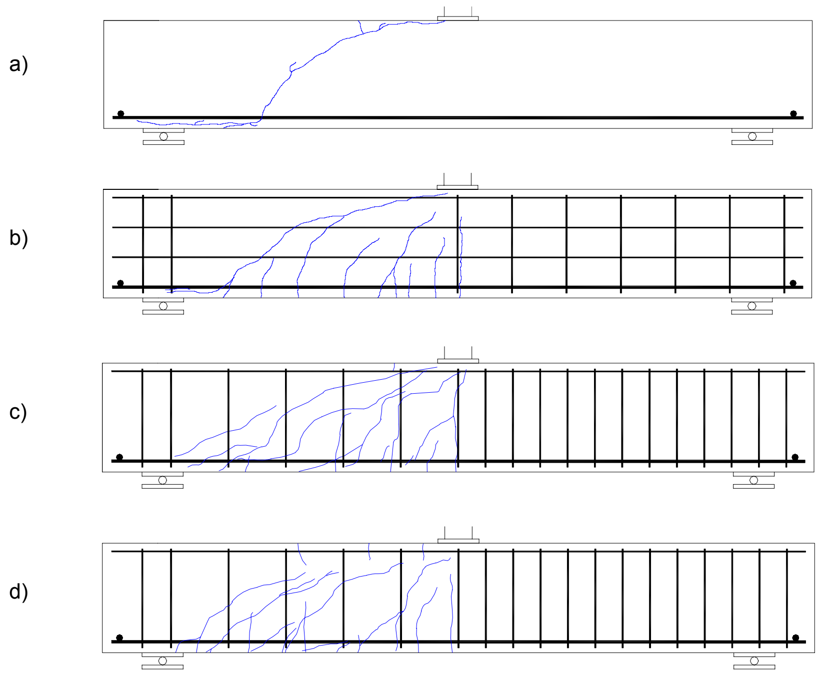

Figure 4.2: Typical crack patterns at failure in the tested beams. a) Beam specimen without web reinforcement. b) Beam specimen with longitudinal reinforcement distributed along the web. c)Beam specimen with stirrups. d) Beam specimen H60/3 – shear cracks did not reach the load application zone.

4.3 Beam specimens without web reinforcement

Specimens H50/1, H60/1, H75/1, and H100/1 did not contain shear reinforcement. The only parameter which varied for all beams was the concrete mix. Longitudinal reinforcement was constant and equal to 2.24%. Their failure shear strengths were 99.69 KN, 108.14 KN, 99.93 KN, and 117.85 KN respectively (Figure 4.3).

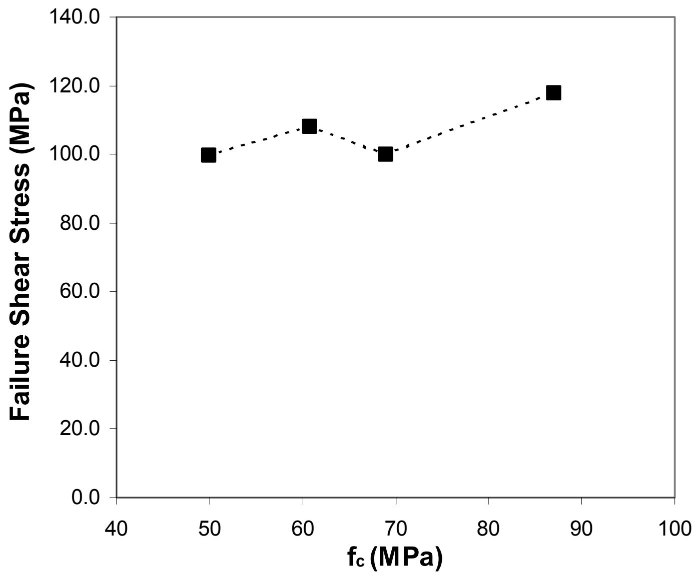

Figure 4.3: Beam specimens without web reinforcement. Influence of the concrete compressive strength.

Therefore, there was a slight increase in failure shear strength as the concrete compressive strength increased, except for beam specimen H75/1, whose splitting concrete strength was lower than the H60 splitting strength (see Table 4.1). The relationship between the failure shear strength and the splitting strength is highlighted in the non linear finite element analyses presented in Annex E. In fact, the same scattered behaviour can be seen for tests described in the current literature. In general, for beam specimens cast using concretes with a compressive strength higher than 60 MPa, the failure shear strength increases, but the scatter of the data is significant. Furthermore, the size effect is closely related to the concrete compressive strength, as will be discussed in Chapter 5.

In beam specimens H50/1 and H60/1 a shear crack was reported before failure. However, it could not be seen in beam H75/1. Upon the formation of the first shear crack in beam specimen H100/1, located on the right side of the beam, the load dropped suddenly, but it increased again (see Figure 4.6). The failure load was considered to be the first peak. Finally, this beam collapsed very briskly and instantly upon the formation of a crack on the left side, and the crack surface totally divided the beam specimen into two pieces. The beam did not collapse at the formation of the first shear crack because the right bearing was fixed and a change in the mechanism of resistance was possible due to the appearance of an axial force between the support and the application point. On the other hand, the sliding bearing on the left side did not allow the redistribution of the internal forces.

The photographs in Figure 4.4 illustrate the difference between the crack state at failure for beams H50/1 and H100/1, and Figure 4.5 shows the surface of the crack for high-strength concrete. It can be seen that the shear crack went through aggregates completely and not around them, as is the case for normal-strength concrete beams.

(a) *(b)*

Figure 4.4: Beam specimens without web reinforcement. Comparison of the final state of cracking in beams H50/1 (a) and H100/1 (b).

Figure 4.5: Beam specimens without web reinforcement. Crack surface in beam H100/1. Crack goes through aggregates.

Figure 4.6 plots the load-deflection response for the beams tested in Series 1. Deflections were very similar in the four beams tested, with beam specimen H50/1 showing the lowest stiffness.

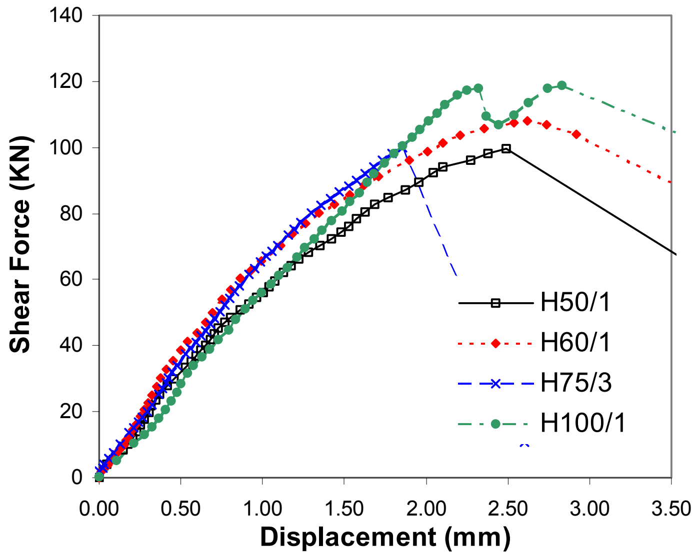

Figure 4.6: Beam specimens without web reinforcement. Load-deflection response for beam specimens without shear reinforcement

4.4 Beam specimens with stirrups

4.4.1 Specimens with the minimum amount of web reinforcement

As discussed earlier, in these beams a the minimum amount of shear reinforcement was provided to ensure that the beams did not collapse immediately following the formation of the first shear crack and to control diagonal cracking widths at service load levels. The minimum amount of shear reinforcement allowed by the EHE procedure is too conservative for high-strength concrete, as it is proportional to the concrete compressive strength. The proposed equation for the minimum amount of web reinforcement, equation 3.3, is proportional to the tensile strength of the concrete.

Beam specimens H50/2, H60/2, H75/2, and H100/2 were provided with the minimum amount of web reinforcement as proposed in this thesis. Table 4.2 summarises the EHE minimum amount of shear reinforcement, what is proposed here and how much the actual beams were given. The differences between the proposed amount of web reinforcement and the amount used are due to the use of the actual concrete strength and yielding stress values in calculations and not the design values.

The value of the shear stresses as measured at stirrup yielding, V_y in Table 4.2, was taken to be the shear strength when the second stirrup crossing the crack yielded. Figure 4.7 and Table 4.2 demonstrate that the amount of web reinforcement provided is appropriate, because the beams show a significant reserve of strength after cracking. Moreover, the crack patterns in Figure 4.8 indicate that the provided amount in beam H100/2 is relatively higher than that provided in specimen H50/2, as more shear cracks developed.

Beam	f_c MPa	$A_{w,EHE}$ (MPA)	$A_{w,prop}$ (MPA)	Provided Shear reinf.		$V_{failure}$ (KN)	V_y (KN)	V_{cr}^* (KN)	V_y / V_{cr}	V_{fail} / V_{cr}	V_{serv} EHE	V_{serv} LRFD
				Stirrup/spacing mm	ρ_w † MPa							
H50/2	49.9	0.665	0.542	ϕ6/260	0.577	177.64	158	85	1.86	2.09	60	77
H60/2	60.8	0.811	0.603	ϕ6/200	0.747	179.74	140	95	1.47	1.89	69	87
H75/2	68.9	0.919	0.642	ϕ6/200	0.747	203.94	144	95	1.52	2.15	69	89
H100/2	87.0	1.333	0.721	ϕ6/165	0.906	225.55	194	110	1.76	2.32	72	97

† Calculated using the real yielding stress of the stirrups
** Approximate cracking shear force*

Table 4.2: Minimum amount of web reinforcement, observed failure, yielding and cracking shear for each specimen.

The shear force at service loads is estimated to be the failure shear strength divided by 1.80, so that it takes into account the load (1.50) and material partial safety factors. It is given in Table 4.2 for the EHE and AASHTO LRFD failure shear strengths (Table 4.24). For both codes, diagonal cracking does not occur at the service load, and therefore, the minimum reinforcement does not have to control crack widths.

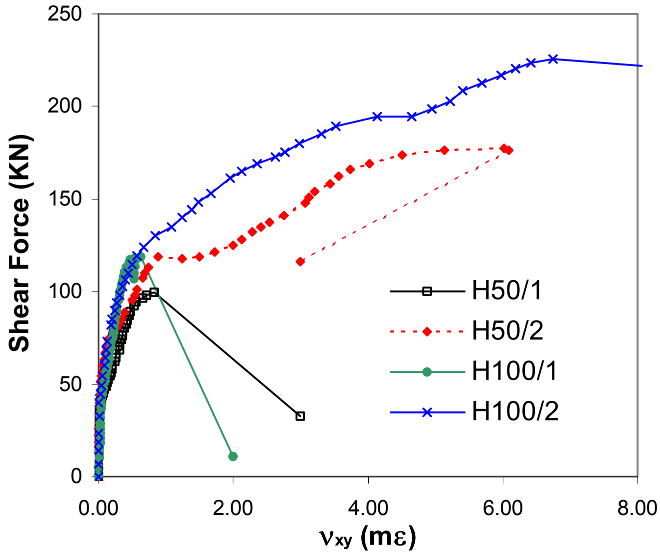

Figure 4.7: Shear deformation in beams without shear reinforcement and with the proposed minimum amount.

Figure 4.8: Crack pattern at failure for test beams with the proposed minimum amount of web reinforcement

4.4.2 Influence of the concrete compressive strength

For elements with stirrups the influence of the concrete compressive strength can be studied from the beam specimens in Series 3 and 4. The failure shear strength of these test specimens is shown in Figure 4.9. As a general trend, it can be seen to increase as the concrete compressive strength increases except for in beam H100/3. Beam specimen H60/4 collapsed under a very high force after the longitudinal reinforcement had yielded. No apparent reason for this behaviour was discovered.

The load-deflection responses of beam specimens H50/3, H60/3, H75/3 and H100/3 are plotted in Figure 4.10. Concrete H60 had the highest splitting strength (Table 4.1), and consequently beam specimen H60/3 exhibited smaller deflections.

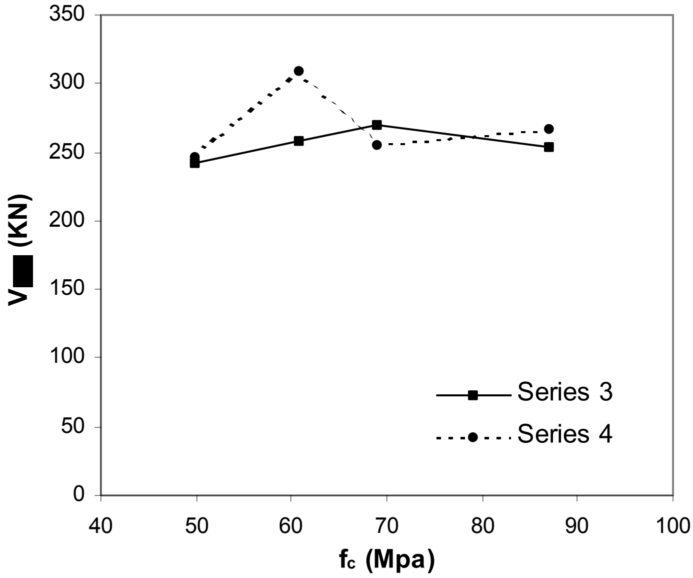

Figure 4.9: Beam specimens with web reinforcement. Failure shear strength vs. concrete compressive strength for series 3 and 4.

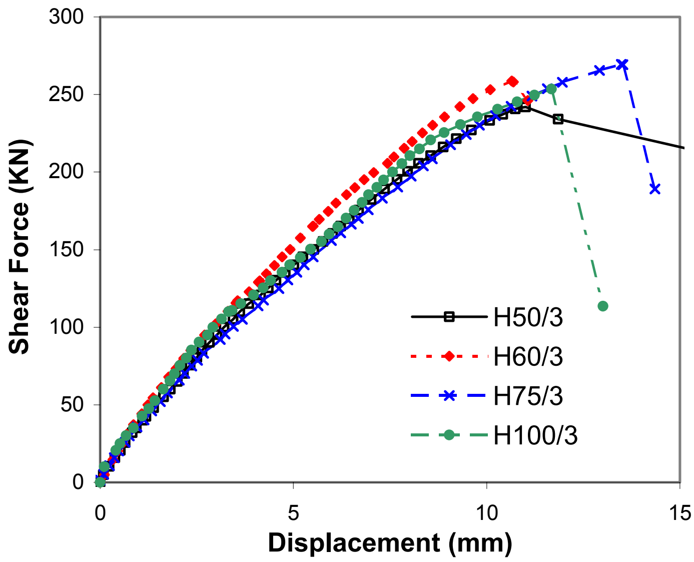

Figure 4.10: Beam specimens with web reinforcement. Load-deflection response for series 3 beam specimen.

4.4.3 Influence of the amount of shear reinforcement

The amount of shear reinforcement was a primary variable during the test design. In this section this influence will be analysed separately for each different concrete mix.

Beam specimens H50/1, H50/2 and H50/3

The failure shear strength of Series H50 beams is shown in Figure 4.12. As indicated in Table 4.1, beam H50/1 did not contain stirrups. Beam specimens H50/2 and H50/3 each had an amount of web reinforcement of 0.577 and 1.291 MPa respectively.

The failure shear strengths were 1.388, 2.516, and 3.448 MPa. A trend line and its equation is represented in Figure 4.11 by a dashed red line in the graph. The trend lines for each different concrete mix will be compared later in this section.

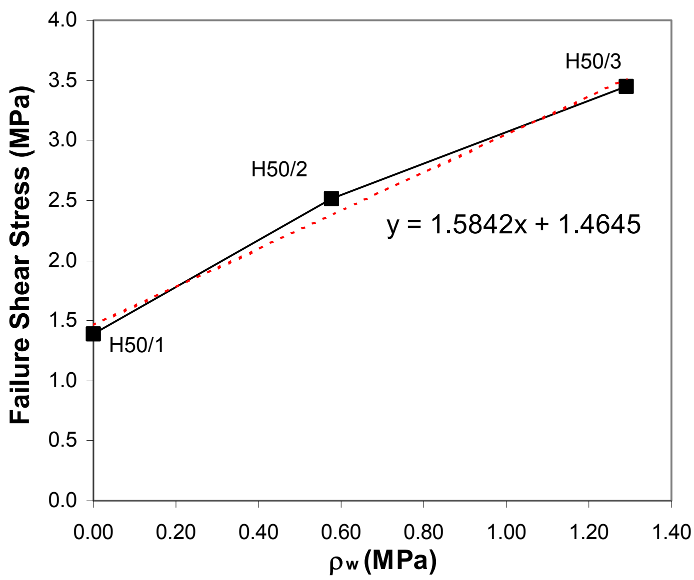

Figure 4.11: Influence of the amount of shear reinforcement. Failure shear stress of beams H50/1, H50/2, and H50/3.

The web shear strain of the H50 beams is plotted in Figure 4.12. The addition of web reinforcement improves the shear response of the specimen by increasing the failure shear strength and a higher ductile response. The cracking pattern also changed. In beam H50/1, a single shear crack was reported, while two shear cracks were noticed in beam H50/2 and three to four shear cracks in beam specimen H50/3 (see Figure 4.13).

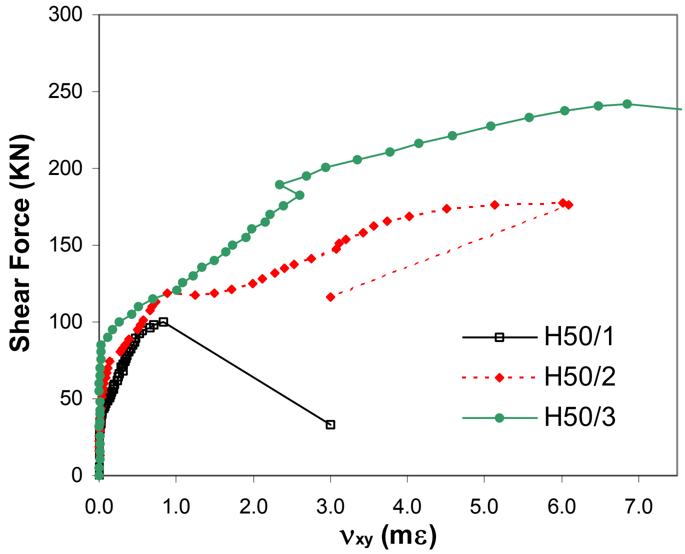

Figure 4.12: Influence of the amount of shear reinforcement. Shear strain of beams H50/1, H50/2, and H50/3.

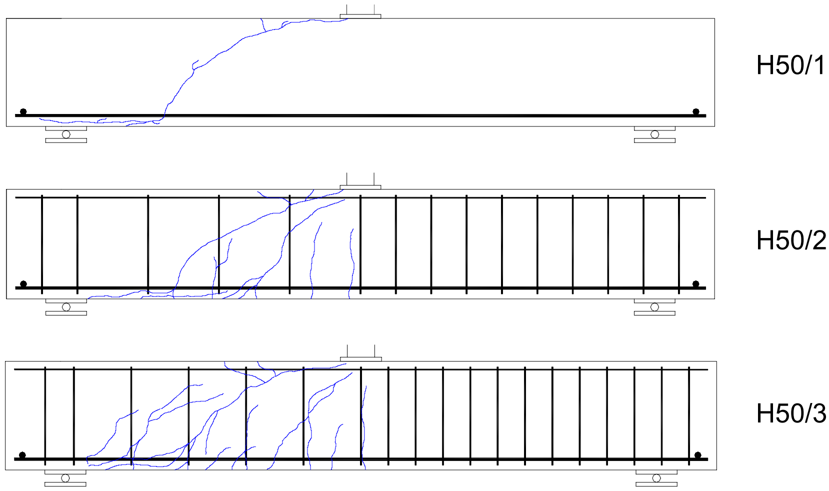

Figure 4.13: Influence of the amount of shear reinforcement. Crack pattern at failure for beams H50/1, H50/2, and H50/3.

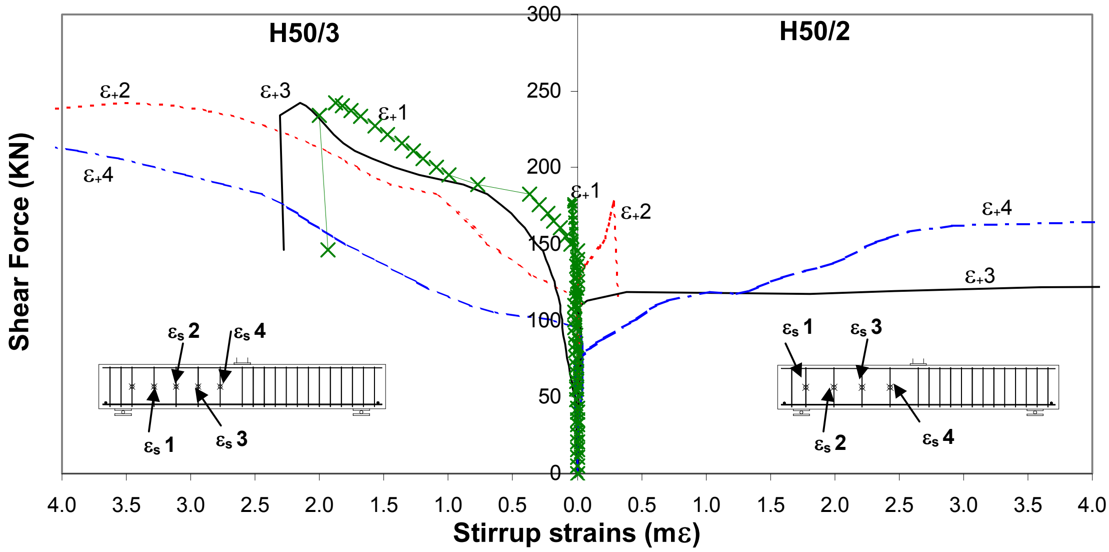

Figure 4.14: Influence of the amount of shear reinforcement. Stirrup strains for beams H50/2 and H50/3.

Figure 4.14 compares the stirrup strains in beams H50/2 and H50/3. In both cases, the stirrup closer to the load application point started to work first. Nevertheless, their behaviour at failure was very different. In beam specimen H50/2 only stirrups 3 and 4 contributed to resisting the shear strength. In beam specimen H50/3 the four instrumented stirrups reached relevant strains, but only stirrups 2 and 4 yielded. However, it is the author's opinion that extrapolations must be made carefully from stirrup strain plots, as the strain is highly dependent on the proximity of the crack. For instance, in beam specimen H50/2, a shear crack crossed stirrup 3 approximately at mid-depth (see Figure 4.13), very near to the strain gauge position. This explains the sudden increase in strain in that stirrup at such a low load level.

Beam specimens H60/1, H60/2 and H60/3

The failure shear strengths of the beams in the H60 Series are shown in Figure 4.15. Beam specimens H60/1, H60/2 and H60/3 had amounts of web reinforcement equal to 0, 0.577 and 1.267 MPa respectively. Their failure shear strengths were 1.510, 2.546, and 3.686 MPa.

The trend line in Figure 4.15 is steeper than that of the Series H50 beam specimens (Figure 4.11). The same happened for beam specimens H75, and this repeats the experimental observation, discussed in Chapter 2, that shear reinforcement seems more effective for high-strength concrete beams than for normal-strength concrete beams.

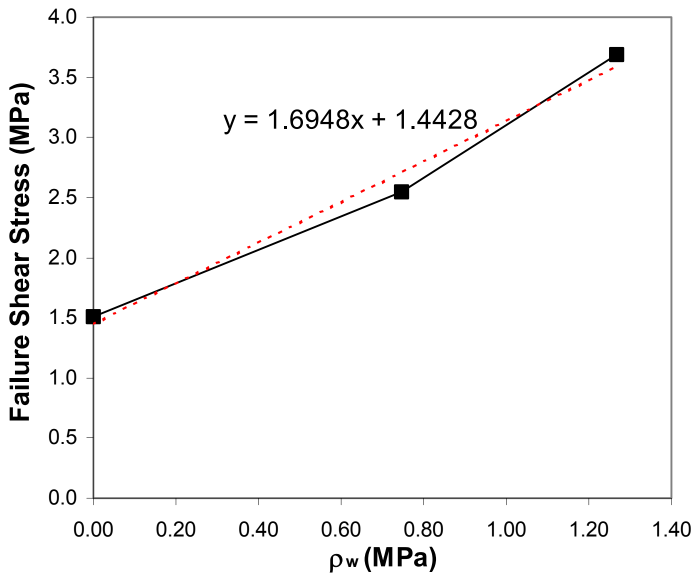

Figure 4.15: Influence of the amount of shear reinforcement. Failure shear stress of beams H60/1, H60/2, and H60/3.

Crack patterns at failure are shown in the photographs in the Figure 4.16. The same performance was reported ad in H50 beam specimens.

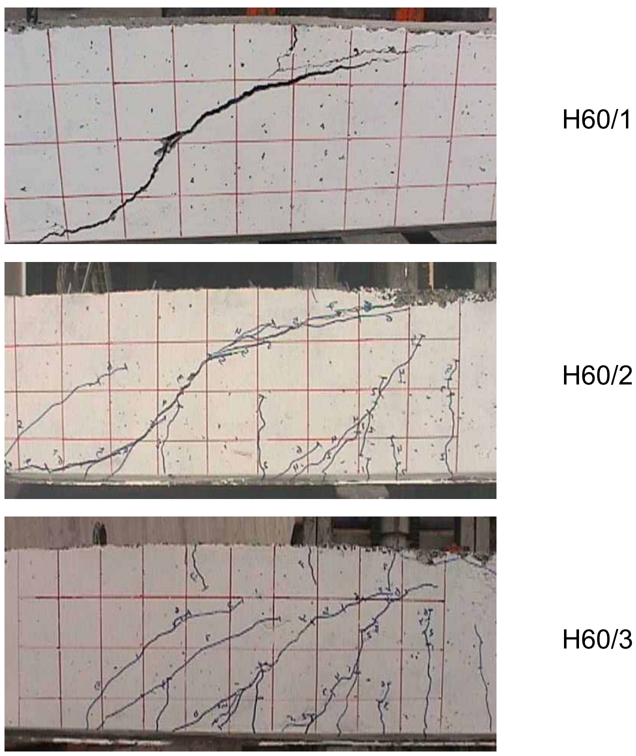

Figure 4.16: Influence of the amount of shear reinforcement. Final state of cracking in the critical shear span for beams H60/1, H60/2, and H60/3.

Beam specimens H75/1, H75/2 and H75/3

Beam specimens H75/1, H75/2 and H75/3 had amounts of web reinforcement equal to 0, 0.577 and 1.267 MPa respectively. Their reported failure shear strengths were 1.392, 2.889 and 3.837 MPa (see Figure 4.17). As was discussed earlier, the higher the concrete compressive strength, the steeper the trend line.

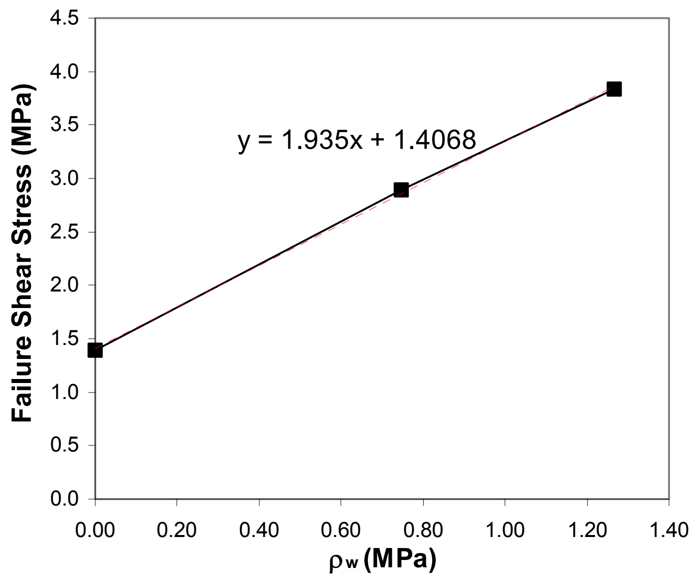

Figure 4.17: Influence of the amount of shear reinforcement. Failure shear stress of beams H75/1, H75/2, and H75/3.

Figure 4.18 compares crack development for beams H75/2 and H75/3. Stirrup spacing was very similar for both beams –200 mm for beam H75/2 and 210 mm for beam H75/3 –but the second had 8 mm diameter stirrups instead of 6 mm diameter stirrups as had H75/2. Crack spacing was reasonably similar in both beams, but in beam H75/3 cracks reached the compression flange under a higher load.

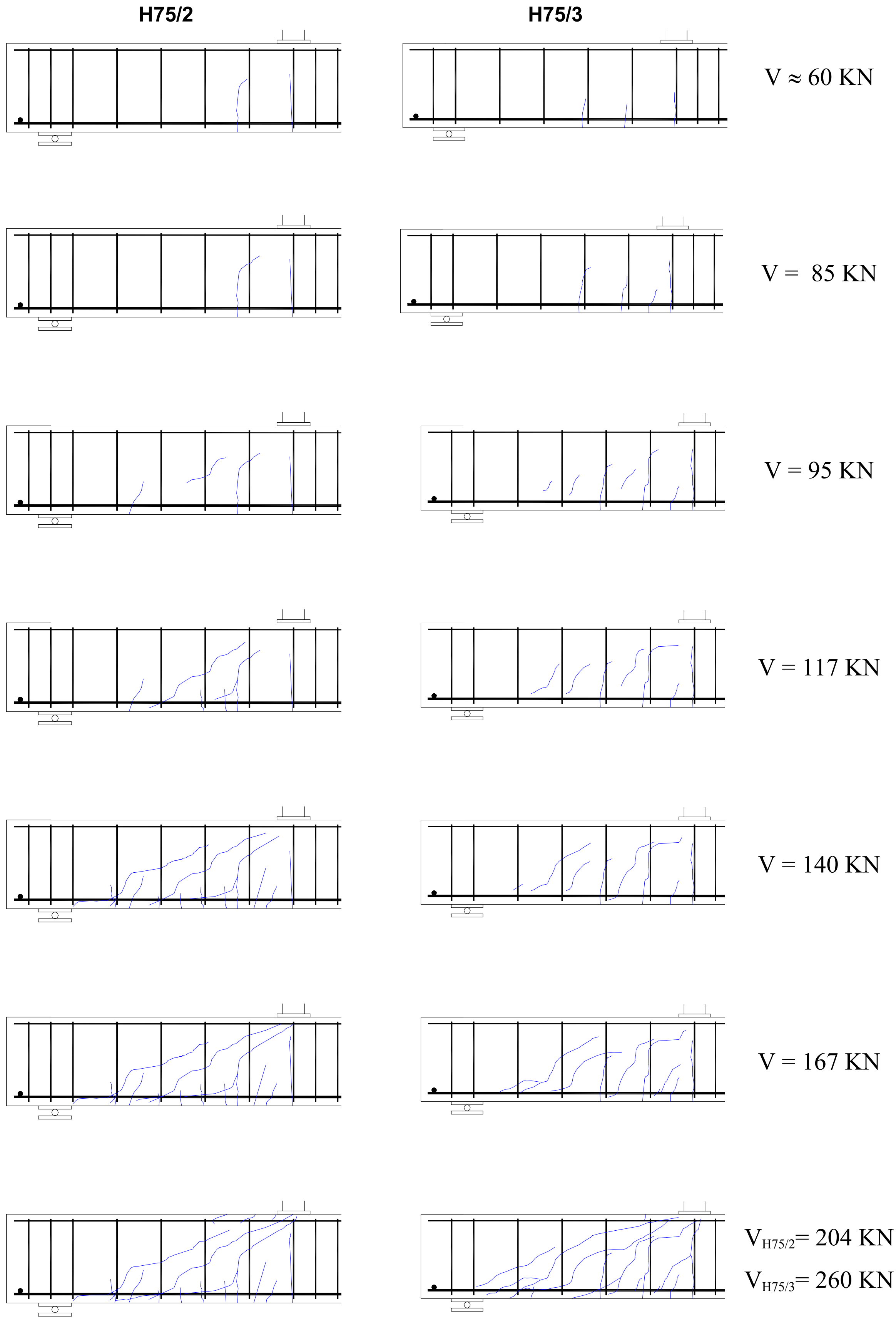

Figure 4.18: Influence of the amount of shear reinforcement. Crack pattern development in beams H75/2 and H75/3.

Beam specimens H100/1, H100/2 and H100/3

Beam specimens H100/1, H100/2 and H100/3 had amounts of web reinforcement equal to 0, 0.906 and 1.291 MPa respectively. Figure 4.19 plots the failure shear strength against the amount of web reinforcement. Their shear strengths at collapse were 1.641, 3.195 and 3.613 MPa.

As was stated earlier in this section, the trend line is flatter that it was first predicted. The author believes that this could have been caused by the high ultimate shear strength of beam H100/1 and the relatively low strength of beam H100/3. However, another reason could be a diminution of the failure shear strength due to either a decrease in shear friction or the greater fragility of HSC. In §5.5.3 the influence of the amount of shear reinforcement as it relates to the concrete compressive strength will be studied based on results of the 123 test beams with stirrups.

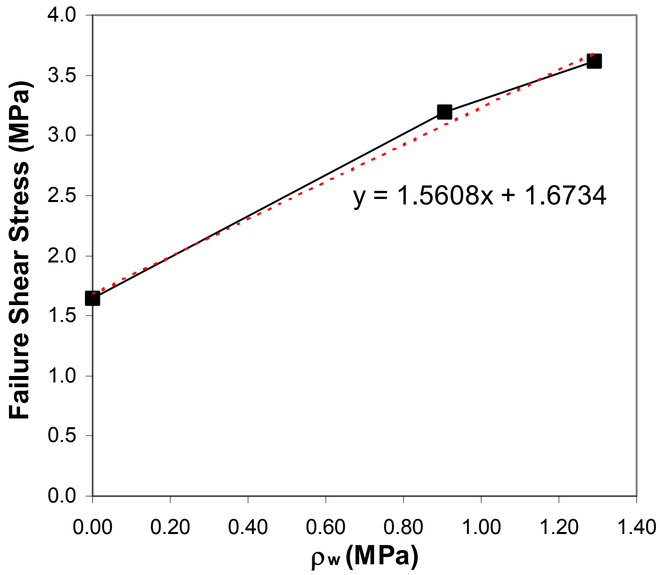

Figure 4.19: Influence of the amount of shear reinforcement. Failure shear stress of beams H100/1, H100/2, and H100/3.

4.4.4 Influence of the amount of longitudinal reinforcement

Current EHE Code, in addition to others, postulates that the failure shear strength does not increase if the amount of the longitudinal reinforcement is higher than 2%. This was evaluated for high-strength concrete beams, by means of two series of beams with different amounts of longitudinal reinforcement.

Beams H50/4, H/60/4, H75/4, and H100/4 had the same amount of shear reinforcement as the Series 3 beams, but a higher amount of longitudinal reinforcement. Series 4 beam specimens had 2.99% of longitudinal reinforcement compared with 2.24% in the Series 3 beams. Evidently, with only two series of beams it was not going to be possible to establish an upper limit.

The longitudinal reinforcement strains are plotted in Figure 4.20 for beams H50/3 and H50/4. They are higher for Series 3 beams than for Series 4 beam specimens. The failure shear strength increased slightly as the amount of longitudinal reinforcement increased except for in beam H75/4 (Figure 4.9). The average increase was approximately 5%.

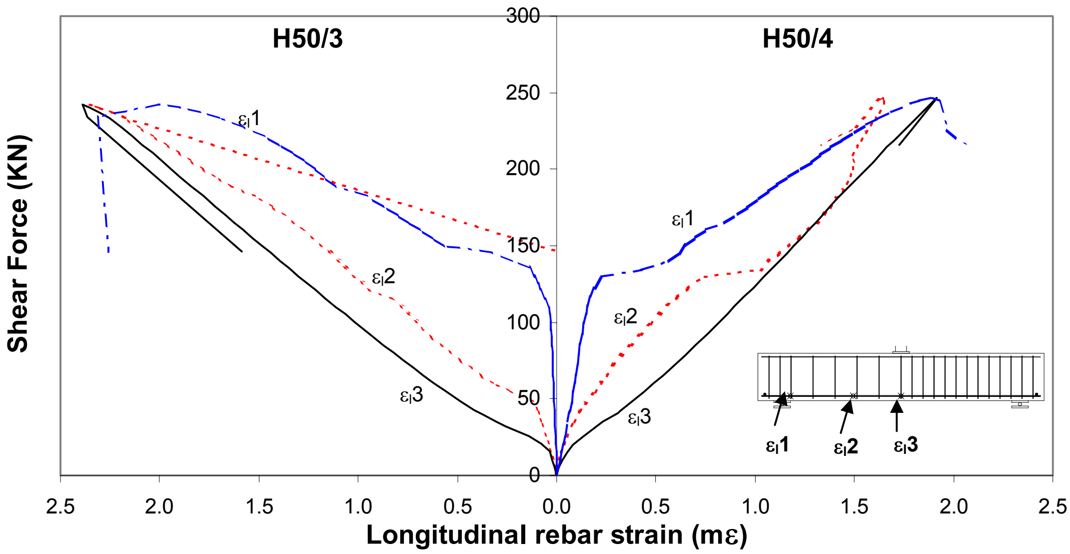

Figure 4.20: Influence of the amount of longitudinal reinforcement. Longitudinal reinforcement strains for beams H50/3 and H50/4

4.5 Beam specimens with distributed longitudinal reinforcement

During the second phase of the experimental campaign, it was decided to evaluate the influence of small longitudinal bars distributed along the web. Beams H50/5 and H100/5 were designed for this purpose based on the tests carried out by Collins and Kuchma (1999). As was mentioned in §2.4.3, they demonstrated that the size effect disappears when beams without stirrups contain well-distributed longitudinal reinforcement.

The failure mechanism were considerably different for beams with distributed longitudinal reinforcement when compared with similar beams without any kind of web reinforcement. Beams H50/1 and H100/1 failed suddenly after the formation of the first shear crack. Failure was especially brisk for the beam with the highest concrete compressive strength. Beam specimens containing distributed longitudinal reinforcement developed more than one shear crack (see Figure 4.22) and failure shear strength was 30.5% higher in beam H50/5 and 18.9% in beam specimen H100/5 than in other, similar beams without web reinforcement (Figure 4.21). However, failure was also sudden.

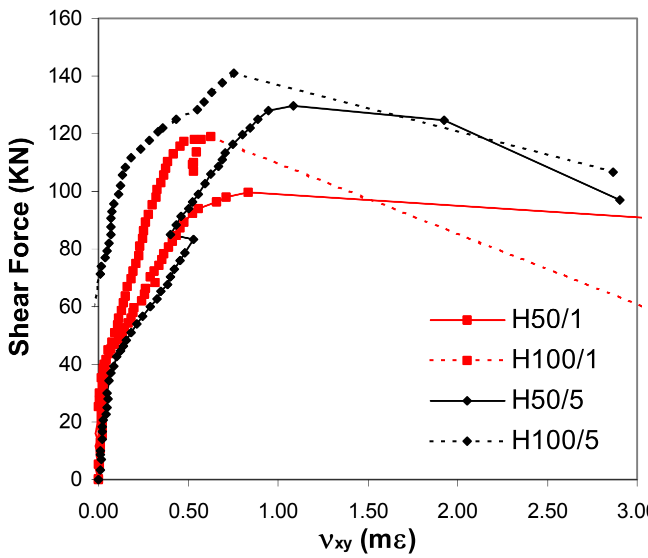

Figure 4.21: Beam specimens with distributed longitudinal reinforcement. Shear strain in beams H50/5 and H100/5.

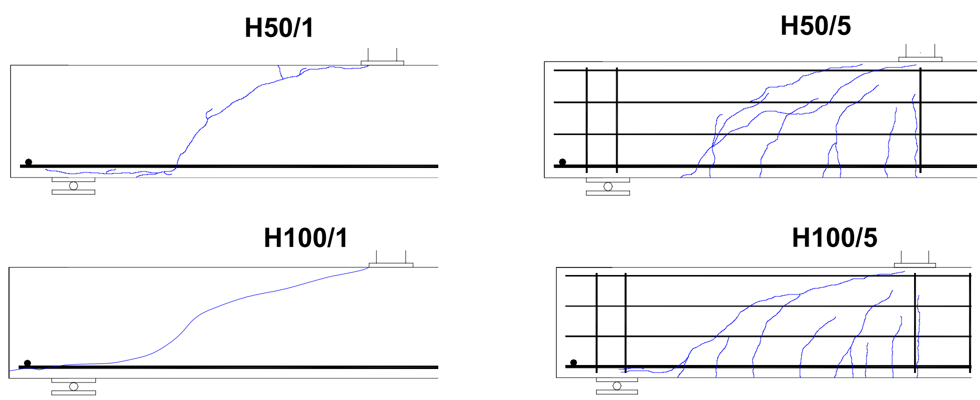

Figure 4.22: Beam specimens with distributed longitudinal reinforcement. Final cracking state for beam specimens H50/5 and H100/5 in comparison to specimens H50/1 and H100/1.

4.6 Comparison of test results with different approaches

Table 4.3 and Figure 4.23 summarise the predictions according to the procedures in the EHE Code, the 2002 Final Draft of Eurocode 2, AASHTO LRFD, ACI 318-99 and Response-2000 (Bentz, 2000), a computer program based on the modified compression field theory (§2.3.4). Section 2.2.2 in Chapter 2 presents the code procedures for members without web reinforcement and Section 2.3.6 summarises the current code methods for members with shear reinforcement. Moreover, a non linear element finite analysis has been carried out using the computer program Vector2 by Prof. Vecchio. This program implements the disturbed stress field model relationships briefly presented in §2.3.2. Its results are summarized in Annex E.

The predictions made by Response-2000 correlate much better with the empirical tests than do the various results given by different codes. For the eighteen beam specimens, the average $V_{test}/V_{predicted}$ ratio is 1.51 for the EHE formulation, 1.25 for the EC-2 and AASHTO LRFD, 1.34 for the ACI 318-99, and 1.05 for the Response-2000 predictions. The coefficient of variation (standard deviation over the average) is respectively 14.8% for EHE, 26.7% for the EC-2, 10.5% for the AASHTO LRFD, 13.7% for the ACI Code, and 8.1% for the Response-2000 predictions.

The AASHTO LRFD predictions also prove satisfactory when compared with the EHE and EC-2 predictions. Although it has been discussed in Chapter 2, it is important to highlight, that the AASHTO procedure is based on the modified compression field theory, and it satisfies not only equilibrium but compatibility.

The EHE and ACI procedures correlate better with members without web reinforcement, than with members with shear reinforcement. For members with stirrups, both codes are too conservative, especially for high-strength concrete specimens.

The EC-2 equations are also excessively conservative for members with shear reinforcement, while for beam specimens without stirrups they are unconservative, in particular for high-strength concrete beams.

Beam	f_c MPa	b mm	d mm	a/d	ρ_w	ρ_l	V_{fai} (KN)	$V_{predicted}$					V_{test} / $V_{predicted}$				
								EHE	EC	LRFD	ACI	Resp	EHE	EC	LRFD	ACI	Resp
H50/1	49.9	200	359	3.01	0	2.24	100	87	110	90	86	91	1.15	0.91	1.11	1.16	1.10
H50/2	49.9	200	353	3.06	0.577	2.28	178	108	91	138	125	162	1.65	1.96	1.29	1.42	1.10
H50/3	49.9	200	351	3.08	1.291	2.29	242	163	203	179	175	228	1.48	1.19	1.35	1.38	1.06
H50/4	49.9	200	351	3.08	1.291	2.99	246	163	203	197	179	259	1.51	1.21	1.25	1.37	0.95
H50/5	49.9	200	359	3.01	0	2.24	130	87	110	102	86	123	1.49	1.18	1.27	1.51	1.06
H60/1	60.8	200	359	3.01	0	2.24	108	93	116	95	95	97	1.16	0.93	1.11	1.14	1.11
H60/2	60.8	200	353	3.06	0.747	2.28	180	124	119	156	145	197	1.45	1.51	1.15	1.24	0.91
H60/3	60.8	200	351	3.08	1.267	2.29	259	160	200	182	180	229	1.62	1.30	1.42	1.44	1.13
H60/4	60.8	200	351	3.08	1.267	2.99	309	160	200	214	184	278	1.93	1.55	1.44	1.68	1.11
H75/1	68.9	200	359	3.01	0	2.24	100	93	145	101	99	99	1.08	0.69	0.99	1.01	1.01
H75/2	68.9	200	353	3.06	0.747	2.28	204	124	119	160	150	206	1.65	1.71	1.28	1.36	0.99
H75/3	68.9	200	351	3.08	1.267	2.29	269	160	200	185	185	230	1.68	1.35	1.45	1.45	1.17
H75/4	68.9	200	351	3.08	1.267	2.99	255	160	200	206	189	284	1.59	1.28	1.24	1.35	0.90
H100/1	87.0	200	359	3.01	0	2.24	118	93	156	110	118	100	1.27	0.76	1.07	1.00	1.18
H100/2	87.0	200	353	3.06	0.906	2.28	226	129	144	175	149	215	1.75	1.57	1.29	1.52	1.05
H100/3	87.0	200	351	3.08	1.291	2.29	254	163	204	192	175	229	1.56	1.25	1.32	1.45	1.11
H100/4	87.0	200	351	3.08	1.291	2.99	267	163	204	215	179	283	1.64	1.31	1.24	1.49	0.94
H100/5	87.0	200	359	3.01	0	2.24	140	93	156	125	118	134	1.51	0.90	1.12	1.19	1.04
										Average			1.51	1.25	1.25	1.34	1.05
										Stand. Deviation			0.22	0.33	0.13	0.18	0.08
										COV (%)			14.8	26.7	10.5	13.7	8.1
										Minimum			1.08	0.69	0.99	1.00	0.90
										Maximum			1.93	1.96	1.45	1.68	1.18

Table 4.3: Summary of the predictions made by the EHE-99, Eurocode 2, AASHTO LRFD, ACI Code 318-99 procedures and Response-2000 program.

4.7 Conclusions of the test results

Based on the test results of the eighteen beam specimens, the following conclusions can be drawn:

- Beams without web reinforcement presented a very fragile behaviour. The higher their concrete compressive strength, the brisker their failure.

- For beams without web reinforcement, the failure shear strength generally increased as the concrete compressive strength increased, except for in beam H75/3.

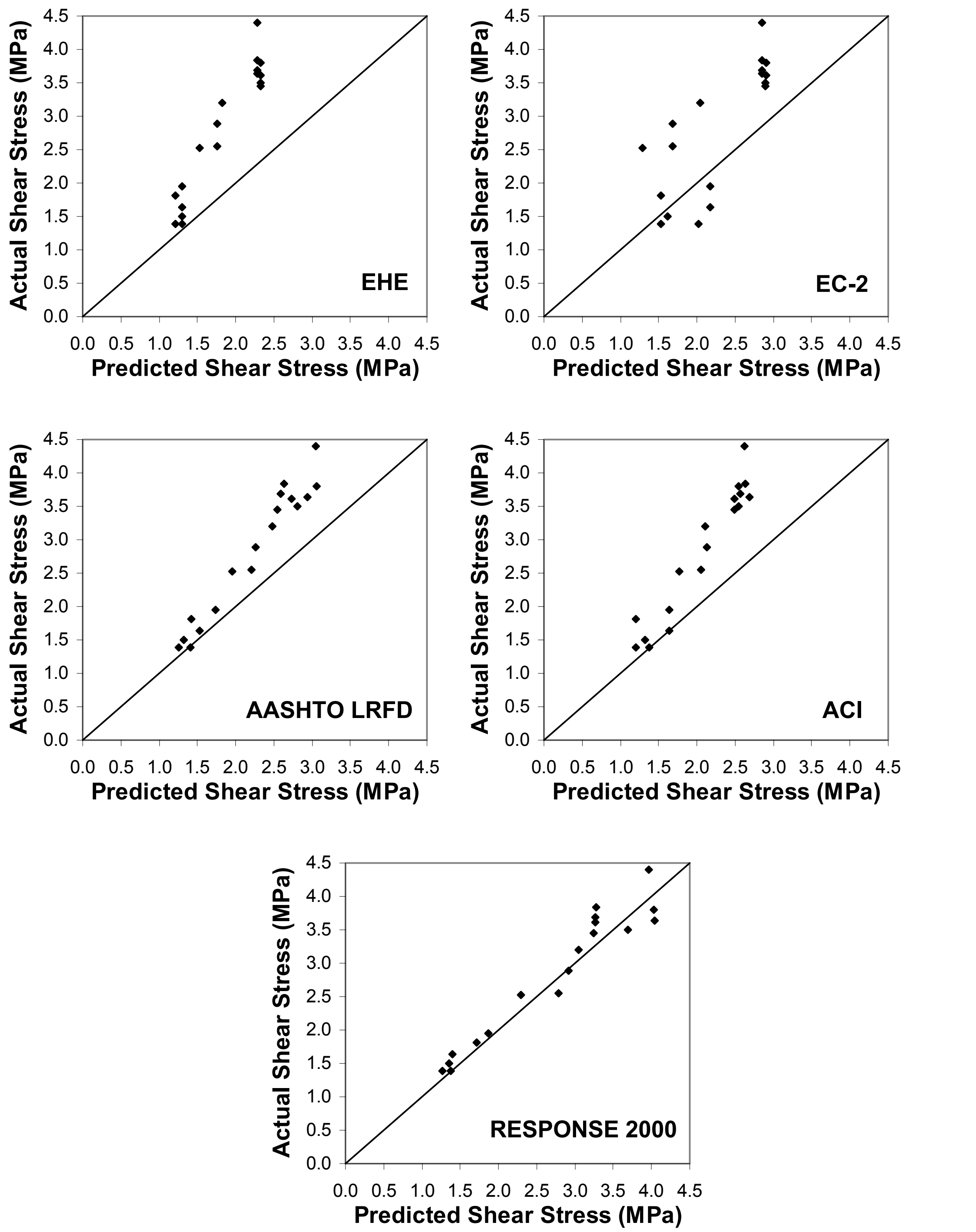

Figure 4.23: Summary of the predictions made by the EHE-99, Eurocode 2, AASHTO LRFD, ACI Code 318-99 procedures and Response-2000 program.

- High-strength concrete beams with stirrups presented a less fragile response.

- The minimum amount of web reinforcement proposed in this dissertation was sufficient in terms of the reserve of strength after shear cracking.

- For beams with the same geometric amount of transverse reinforcement, the higher their concrete compressive strength, the higher their failure shear strength.

- The influence of the amount of shear reinforcement varied according to the concrete compressive strength. Stirrups were more effective at higher compressive concrete strengths, except for in concrete mix H100.

- For high-strength concrete beams with stirrups, the limitation of the amount of longitudinal reinforcement to 2% is not experimentally justified.

- Beam specimens with longitudinally-distributed web reinforcement along the web improved in behaviour compared with similar beams without any kind of shear reinforcement. Although their failure was also fragile, several shear cracks were reported, and the failure shear strength increased about 25%.

- EHE shear procedures offer good correlation for beams without web reinforcement. However, for beams with stirrups, they are absolutely conservative. The V_{test} / V_{EHE} ratio for the eighteen beam specimens is 1.51 and the coefficient of variation 14.8%.

- The April 2002 Final Draft of EuroCode 2 is unconservative for beams without web reinforcement, but is too conservative for beams with shear reinforcement. The V_{test} / V_{EC-2} ratio is 1.25 and the coefficient of variation 26.7%.

- The AASHTO LRFD Specifications, based on the modified compression field theory, showed a close correlation to the empirical results in comparison to the other codes' correlations. The V_{test}/V_{AASHTO} ratio is 1.25 and the coefficient of variation 10.5%.

- The failure shear strength predicted by the Response-2000 computer program, also based on the modified compression field theory, shows a satisfactory correlation with the test results, with a $V_{test}/V_{Resp-2000}$ ratio of 1.05 and a coefficient of variation of 8.1%.