Chapter 3

Experimental program

To better understand the response of high-strength concrete beams failing in shear with and without shear reinforcement, eighteen reinforced concrete beams were tested at the Structural Technology Laboratory of the Department of Construction Engineering at the School of Civil Engineering of Barcelona. The concrete compressive strength of the beams at the age of the tests ranged from 50 to 87 MPa. The primary design variables were the amount of shear and longitudinal reinforcement. This chapter describes the objectives of the experimental campaign, details of the beam specimens, their construction, material properties, the instrumentation utilized, and the testing procedure that was used. The results of the tests and a discussion are presented in Chapter 4.

3.1 Objectives of the Experimental Campaign

The main objectives of the experimental campaign carried out were:

- To study the influence of the concrete compressive strength in beams with and without shear reinforcement. Current procedure in Spain holds that the failure shear strength does not increase when concrete compressive strength is higher than 60 MPa for beams both with and without web reinforcement.

- To propose and verify a minimum amount of web reinforcement to provide more realistic values than those obtained in the current EHE proposal, in accordance with experimental results and other codes of practice formulations.

- To evaluate the efficiency of the amount of shear reinforcement as a function of the concrete compressive strength. Some authors believe that for high-strength concrete beams, stirrups are more efficient than for normal-strength beam specimens, as was explained in §2.4.4.

- To evaluate the influence of the amount of longitudinal reinforcement. The majority of current codes' limitation of the amount of longitudinal reinforcement to 2% will be studied for high-strength concrete beams.

- To study the influence of longitudinally-distributed web reinforcement for high-strength members without stirrups, as this variable has an important effect on the failure shear strength according to Collins and Kuchma (1999), as explained in §2.4.3.

3.2 Design of the Test Specimens

In order to achieve the previous objectives, eighteen beam specimens were designed and tested. Table 3.1 and Figure 3.1 show the details of the 200 mm wide × 400 mm deep beam specimens that were tested with a shear span of 1080 mm.

The test program consisted of four series of beams: 1) the H50 series, designed to have a concrete strength of 50 MPa; 2) the H60 series, designed to have a concrete strength of 60 MPa; 3) the H75 series, designed to have a concrete strength of 75 MPa with silica fume; and 4) the H100 series, designed to have a concrete strength of 100 MPa. The actual concrete strength at the time of testing is presented in Table 3.1.

Beam specimen number one in each series (H50/1, H60/1, H75/1, and H100/1) did not have shear reinforcement. The longitudinal reinforcement consisted of two 32 mm diameter bars (ρ_l = 2.24%), with a characteristic yielding stress of 500 MPa.

Beam number two in each series (H50/2, H60/2, H75/2, and H100/2) contained the proposed minimum amount of shear reinforcement. The longitudinal reinforcement was equal to that provided in series 1.

Based on §2.4.2, it is proposed that the minimum amount of shear reinforcement be proportional to the tensile strength of the concrete. In the Spanish EHE Code the average concrete tensile strength is equal to:

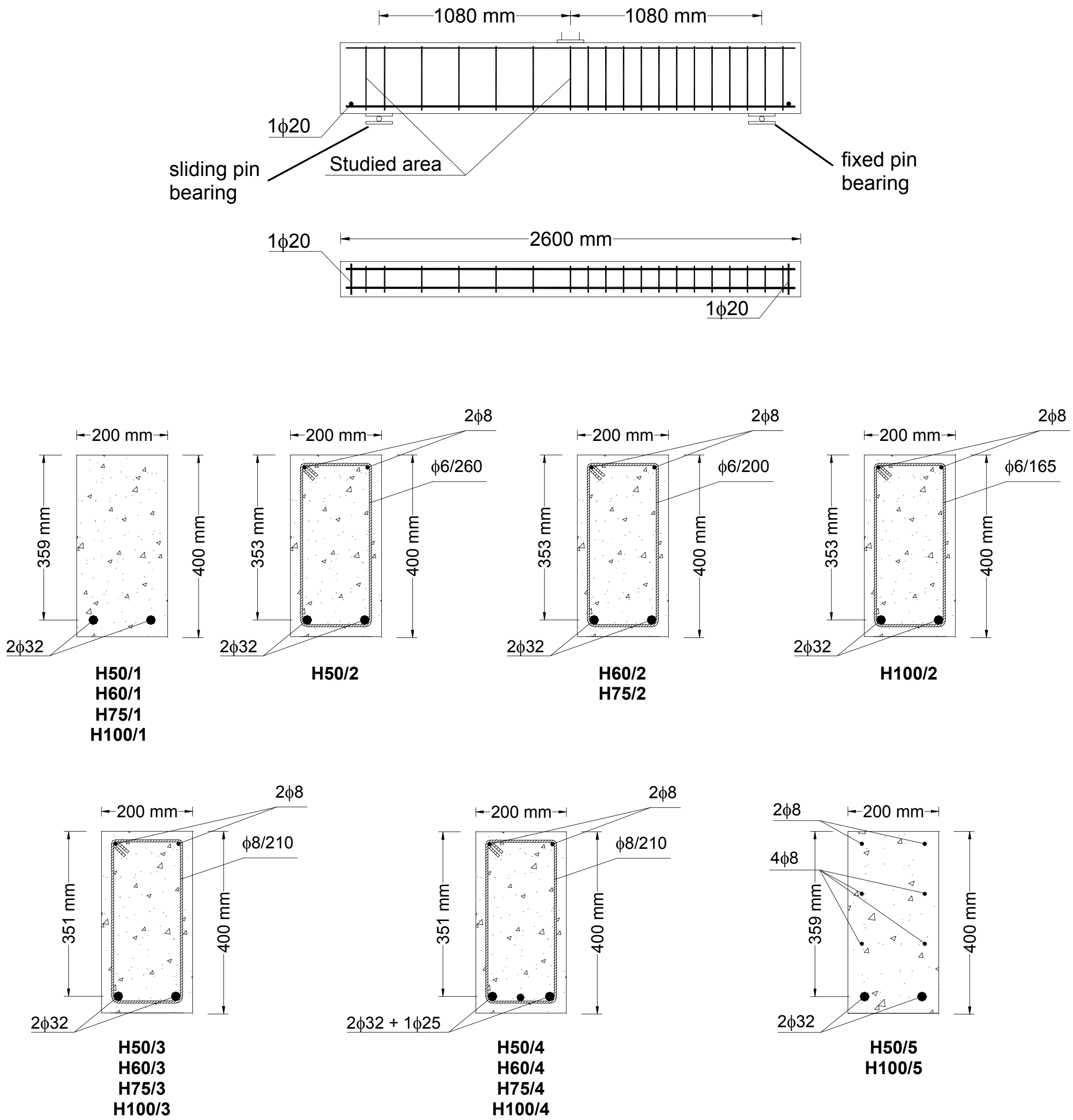

Figure 3.1. Test set-up and cross-section of the beam specimens.

$$f_{ct,m} = 0.30\sqrt[3]{f_{ck}^2} \text{ MPa} \tag{3.1}$$

In the 'Design guidance for high-strength concrete' it is suggested that equation 3.1 is unconservative for high-strength concrete. For concretes with compressive strengths higher than 60 MPa it proposes the following equation:

$$f_{ct,m} = 0.58\sqrt[2]{f_{ck}} \text{ MPa} \tag{3.2}$$

Our proposed amount of minimum shear reinforcement is given in equation 3.3 and compared in Figure 3.2 with some other code proposals.

$$A_{w,min} = \frac{f_{ct,m}}{7.5}\frac{b_w s}{f_y} \text{ MPa} \tag{3.3}$$

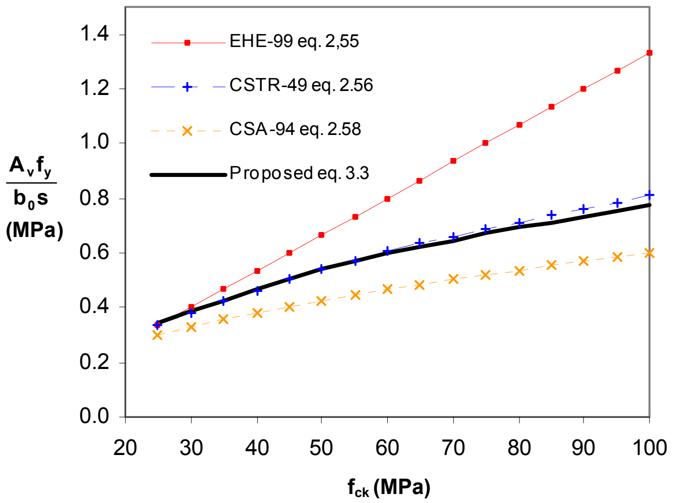

Figure 3.2: Comparison of minimum amount of web reinforcement and proposed equation

The third specimen in each series (H50/3, H60/3, H75/3, and H100/3) had the same amount of web reinforcement for all beams, 8 mm diameter stirrups spaced 210 mm. It was designed to have the highest amount of web reinforcement in order to produce a shear failure with the provided amount of longitudinal reinforcement. Additionally, the flexural tension reinforcement for all the beams consisted of 2ϕ32 bars.

The fourth beam in each series (H50/4, H60/4, H75/4, and H100/4) had the same shear reinforcement as the third series but the flexural tension reinforcement consisted of 2ϕ32 bars plus a 1ϕ25 bar. Hence, the amount of longitudinal reinforcement was equal to 2.99%.

The fifth beam specimen in each series (H50/5 and H100/5) did not have stirrups but instead contained small longitudinal bars (8 mm diameter) distributed along the web. As was mentioned earlier, Collins et al. postulated that the size effect is not only a function of the beam depth, but also of the distance between distributed longitudinal reinforcement (s_z). Each layer of this crack control reinforcement must have an area of at least $0.004b_ws_z$, according to AASHTO Specifications. The area provided in specimens H50/5 and H100/5 verified the previous expression.

Beam	f_c MPa	b mm	D mm	a/d	Shear reinf.			Long. Reinf.		Cast date	Test Date
					Stirrup/spacing Mm	ρ_w %	ρ_w MPa	Longitudinal reinforcement	ρ_l		
H50/1	49.9	200	359	3.01	-	0	0	2ϕ32	2.24	Jan. 11,02	Feb. 20,02
H50/2	49.9	200	353	3.06	ϕ6/260	0.109	0.577	2ϕ32	2.28	Jan. 11,02	Feb. 21,02
H50/3	49.9	200	351	3.08	ϕ8/210	0.239	1.291	2ϕ32	2.29	Jan. 11,02	Feb. 25,02
H50/4	49.9	200	351	3.08	ϕ8/210	0.239	1.291	2ϕ32 + 1ϕ25	2.99	Jan. 11,02	Feb. 26,02
H50/5	49.9	200	359	3.01	-	0	0	2ϕ32 + 6ϕ8	2.24	Jan. 11,02	Feb. 26,02
H60/1	60.8	200	359	3.01	-	0	0	2ϕ32	2.24	Feb. 23,01	Apr. 09, 01
H60/2	60.8	200	353	3.06	ϕ6/200	0.141	0.747	2ϕ32	2.28	Feb. 23,01	Apr. 19, 01
H60/3	60.8	200	351	3.08	ϕ8/210	0.239	1.267	2ϕ32	2.29	Feb. 23,01	Apr. 20, 01
H60/4	60.8	200	351	3.08	ϕ8/210	0.239	1.267	2ϕ32 + 1ϕ25	2.99	Feb. 23,01	Apr. 20, 01
H75/1	68.9	200	359	3.01	-	0	0	2ϕ32	2.24	Feb. 23,02	Apr. 04,01
H75/2	68.9	200	353	3.06	ϕ6/200	0.141	0.747	2ϕ32	2.28	Feb. 23,02	Apr. 05,01
H75/3	68.9	200	351	3.08	ϕ8/210	0.239	1.267	2ϕ32	2.29	Feb. 23,02	Apr. 05,01
H75/4	68.9	200	351	3.08	ϕ8/210	0.239	1.267	2ϕ32 + 1ϕ25	2.99	Feb. 23,02	Apr. 06,01
H100/1	87.0	200	359	3.01	-	0	0	2ϕ32	2.24	Jan. 11,02	Apr. 09,02
H100/2	87.0	200	353	3.06	ϕ6/165	0.171	0.906	2ϕ32	2.28	Jan. 11,02	Apr. 10,02
H100/3	87.0	200	351	3.08	ϕ8/210	0.239	1.291	2ϕ32	2.29	Jan. 11,02	Apr. 11,02
H100/4	87.0	200	351	3.08	ϕ8/210	0.239	1.291	2ϕ32 + 1ϕ25	2.99	Jan. 11,02	Apr. 15,02
H100/5	87.0	200	359	3.01	-	0	0	2ϕ32 + 6ϕ8	2.24	Jan. 11,02	Apr. 16,02

Table 3.1: Details of beam specimens

3.3 Specimen Details

3.3.1 Material Properties

Concrete Properties

Concrete mixes were designed by Alvisa, a precast bridge plant in Huesca (Spain). A maximum aggregate size of 12 mm was used throughout the series. Standard 150mm x 300 mm cylinders were cast with the specimens to obtain the compressive strength and splitting strength of each concrete mix. These cylinders were kept under the same environmental conditions as the beam specimens until the time of testing. Annex B summarises the test results for the cylinders. Figure 3.3 shows the explosive behaviour of an H100 cylinder as compared to that of an H50 cylinder.

Table 3.2 lists the concrete mixes, the compressive and splitting strength of the four concretes considered. The cement used was standard Portland cement CEM I 52.5 R for mixtures H50, H60 and H75. Cement in the H100 concrete was a Uniland cement commercially marked as Standford. Its main characteristic is that is finer than a standard 52.5 R. Only the H75 mix contained silica fume.

	H50	H60	H75	H100
Cement 52.5 R, kg/m^3	350	415	410	415
Silica fume, kg/m^3	-	-	60	-
Water, L/m^3	96	100	104	123
Aggregate 6/12 mm, kg/m^3	1100	1120	1100	1073
Aggregate 0/6 mm, kg/m^3	850	860	930	848
Superplasticer, L/m^3	6.0	6.5	6.5	7.86
W / C (added water)	0.27	0.24	0.22	0.30
Compression strength	49.9	60.8	68.9	87.0
Splitting strength	3.46	4.22	3.69	4.05

Table 3.2: Composition of four concretes

Reinforcing Steel Properties

Spanish standard B 500 S reinforcing bars, with a characteristic yielding stress of 500 MPa, were used. Table 3.3 lists the actual yield stress, f_y, and the ultimate stress, f_u, for the web reinforcing bars, which were tested following EN 10002-1, and UNE 4-474-92

Standards. Typical stress-strain curves are shown in Figure 3.4. Longitudinal rebars were not tested.

Annex B summarises the test procedure carried out to test the 6 mm and 8 mm diameter bars.

Figure 3.3: Comparison of the compression failure of H50 and H100 concrete cylinders

Size – series	Area mm^2	f_y MPa	f_u MPa
ϕ6 - H60 and H75	28.27	530	680
ϕ8 - H60 and H75	50.27	530	685
ϕ6 - H50 and H100	28.27	530	680
ϕ8 - H50 and H100	50.27	540	672

Table 3.3: Properties of web reinforcing bars

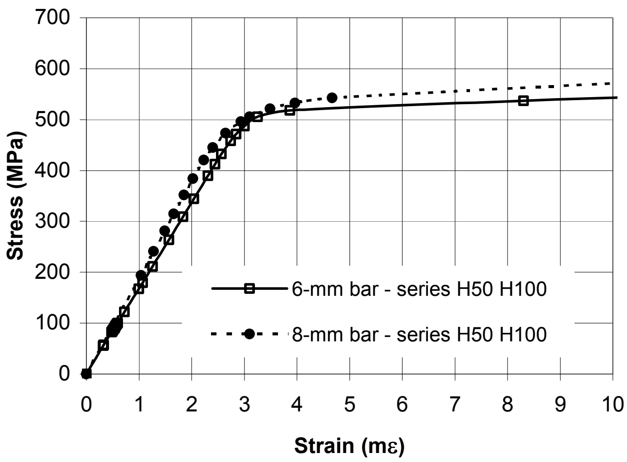

Figure 3.4: Typical stress-strain curves for web reinforcing steel

3.3.2 Fabrication of the Test Specimens

The beams were cast at the Alvisa precast concrete plant, located in Selgua (Huesca, Spain). The concrete components, reinforcement bars, moulds, and procedures were those actually used at that plant. Figure 3.5 shows some pictures of the fabrication of the beam specimens used in series H60 and H75.

Series H60 and H75 were cast on 23 February 2001. The eight beams and the 150 x 300 mm cylinders were stored at the plant for approximately 28 days. Series H50 and H100 were cast on 11 January 2002. After one week, the beams and the cylinders were transported to the laboratory in Barcelona. Note that for each series, only a single two-cubic-metre batch was used.

3.4 Instrumentation

To monitor the behaviour of the tested beams, the applied loads, strains at the reinforcement and at the concrete surface, and displacements were measured using different instruments such as load cells, strain gauges and magnetostrictive transducers (LVDT´s). All the variables were monitored continuously by the data acquisition system. Photography and video equipment were also utilised.

Beams tested in 2001 and 2002 were instrumented in slightly different ways. During the second campaign some techniques were used that had been observed by the author at the University of Toronto, which has a long history of testing beams failing in shear.

In order to determine the load-deflection curve, displacements were measured in all the beam specimens by means of Temposonics© magnetostrictive transducers located below the loading point (midspan), ¼ span and at the supports (Figure 3.6).

The total applied force was obtained using an Instron© 1000 KN static load cell.

Figure 3.5: Specimen fabrication at the Alvisa precast plant in Selgua (Huesca, Spain)

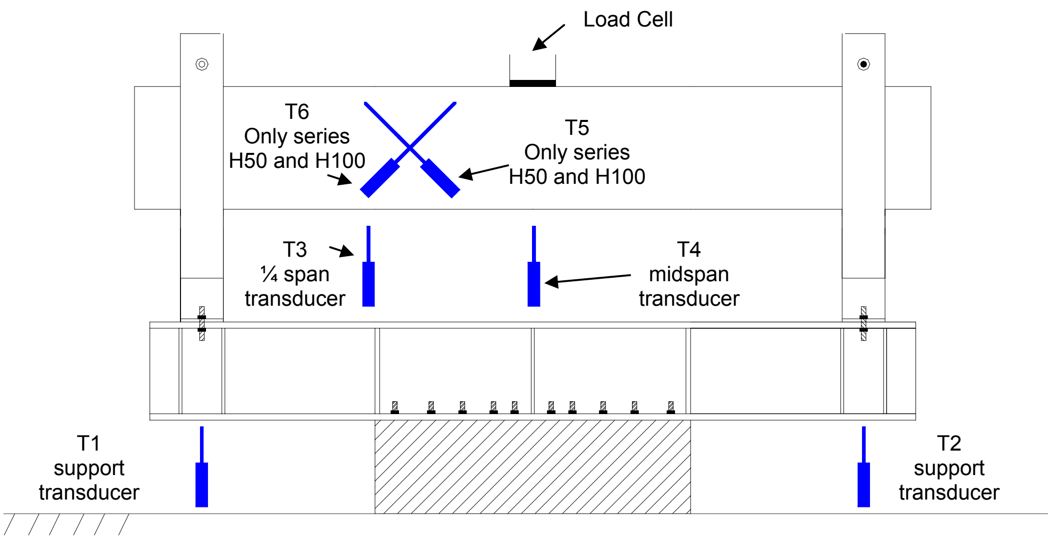

Figure 3.6: Location of the Temposonic© transducers for load-deflection measuring

3.4.1 Series H60 and H75

Strain gauges were attached at mid-depth to all the stirrups in the studied area, as is shown in Figure 3.7. The type of gauge used was Tokyo Sokki© FLA-3-11, with a longitude of 3 mm. Moreover, one strain gauge was located at a distance d_v $(d_v \approx 0.9 \cdot d)$ away from the loading plate on the longitudinal reinforcement. For this purpose, 6 mm long Tokyo Sokki© FLA-6-11 strain gauges were used. The adhesive utilised, following the indications of the strain gauge producer, was cyanocrilate (CN).

Additionally, two strain gauge rosettes (the blue marks in Figure 3.7 and photographs in Figure 3.8) were installed in the concrete web surface to measure web strains. 60 mm long Tokyo Sokki© PLR-60-11 strain gauge rosettes were used with a two-component polyester adhesive (PS). However, these rosettes did not reveal any relevant information because they measured local strains, while the problem of shear is more closely related to average strains and stresses (§2.3.4) than local stresses, which are only influential at the crack location.

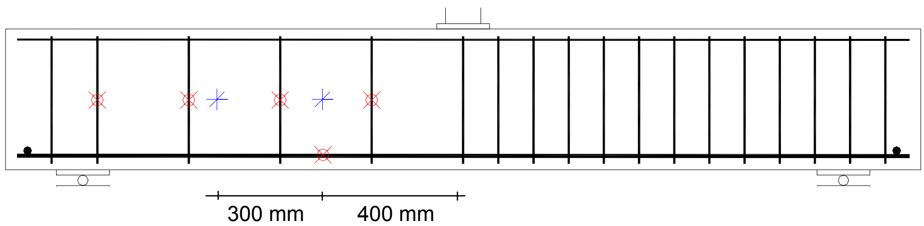

Figure 3.7: Typical strain gauge location for beam specimens H60 and H75

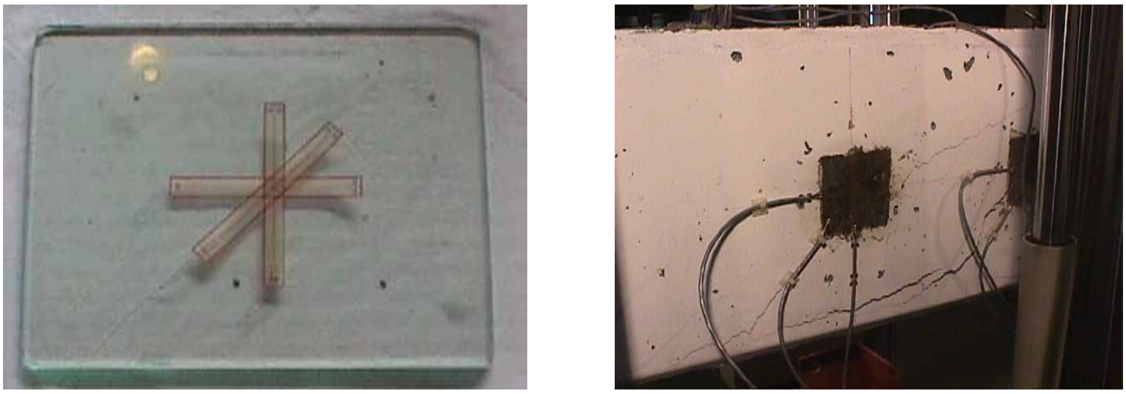

Figure 3.8: Strain gauge rosette in the concrete web for beam specimens H60 and H75

3.4.2 Series H50 and H100

Strain gauges were also attached at mid-depth to all the stirrups in the studied area. Furthermore, three strain gauges were installed on the longitudinal reinforcement; one at midspan; the second at ¼ span; and the third at the plate support border. The configuration can be seen in Figure 3.9. All of these were 3 mm long Tokyo Sokki© FLA-3-11-5L gauges. The adhesive utilised was again cyanocrilate (CN).

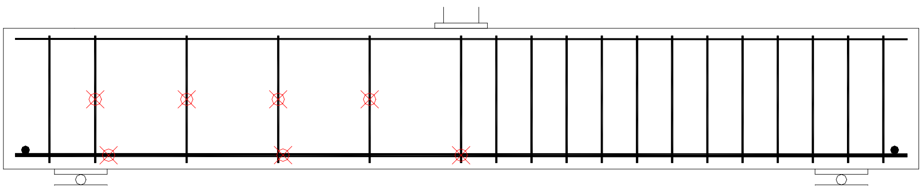

Figure 3.9: Typical strain gauge location for beam specimens H50 and H100

A rosette of two Temposonic© transducers (Figure 3.6) was mounted on one side of the beam to measure the web strain, γ_{xy}. The vertical axis of this transducer rosette was situated 400 mm away from the load plate. The photographs in Figure 3.11 show the actual configuration.

Using the Mohr's circle in Figure 3.10 one can deduce how the web strain, γ_{xy}, can be obtained from the readings from the T5 and T6 transducers. The web strain is given by the equation:

$$0.5 \cdot \gamma_{xy} = R \cdot \sin 2\theta \tag{3.4}$$

where R is the radius of the Mohr's and θ the angle of the principal compression strain.

Additionally,

$$\frac{\varepsilon_{T5} - \varepsilon_{T6}}{2} = R \cdot \sin 2\theta \qquad (3.5)$$

hence:

$$\gamma_{xy} = \varepsilon_{T5} - \varepsilon_{T6} \qquad (3.6)$$

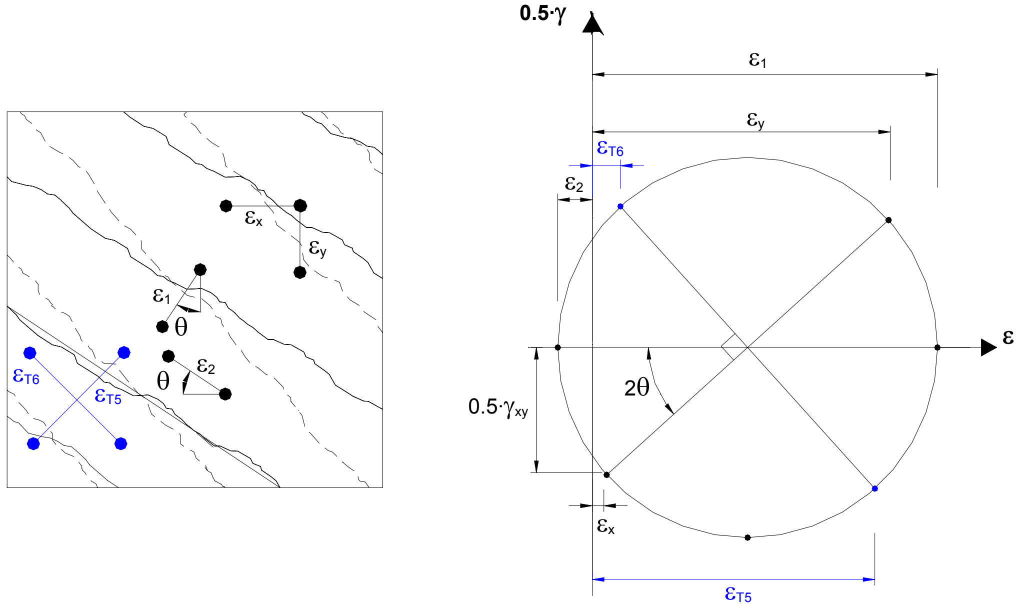

Figure 3.10: Calculation of the shear strain, γ_{xy}, from *Mohr's circle.*

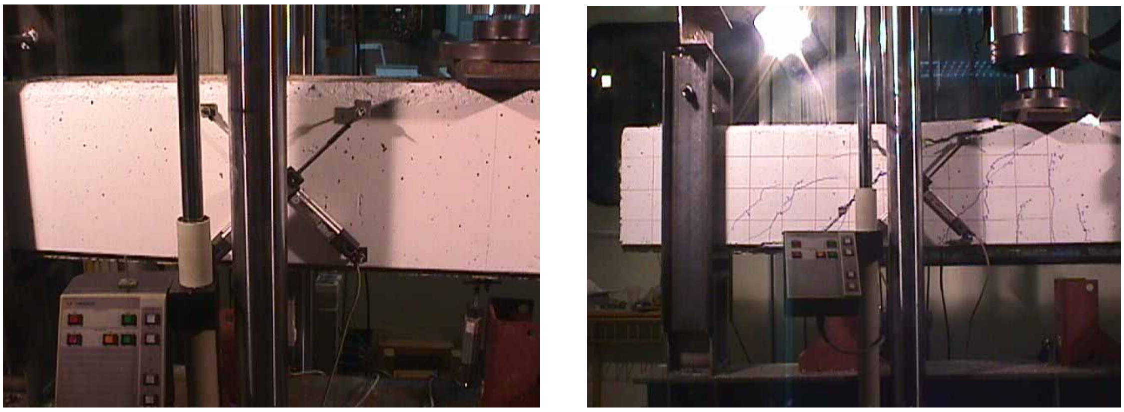

Figure 3.11: Rosette of Temposonic© transducers for measuring the shear deformation.

3.4.3 Data acquisition system

The data acquisition system utilised was an HP© 34970 A with 22 analogical inputs. Though the magnetostrictive transducers were connected directly, the strain gauges were first connected to a VISHAY 2100 control module. The software used for data collection was Data Logger. Readings were taken every 2 or 3 seconds depending on the loading rate and the number of input channels taking readings.

3.5 Testing procedure

3.5.1 Test Configuration

Figure 3.12 shows the test as it was configured. The load was applied at midspan of the beam specimen by a 150 mm wide and 28 mm thick neoprene pad under a spherical bearing.

The beam specimen was supported by a sliding pin bearing, on the instrumented side, and a fixed pin bearing on the opposite side. The importance of the bearing conditions is studied in Annex E. The diameter of the bearing cylinder was 400 mm. Bearings were supported by a reacting beam (IPE 300) specifically adapted to these tests. This beam was fixed to the loading machine base by 20 high-strength bolts.

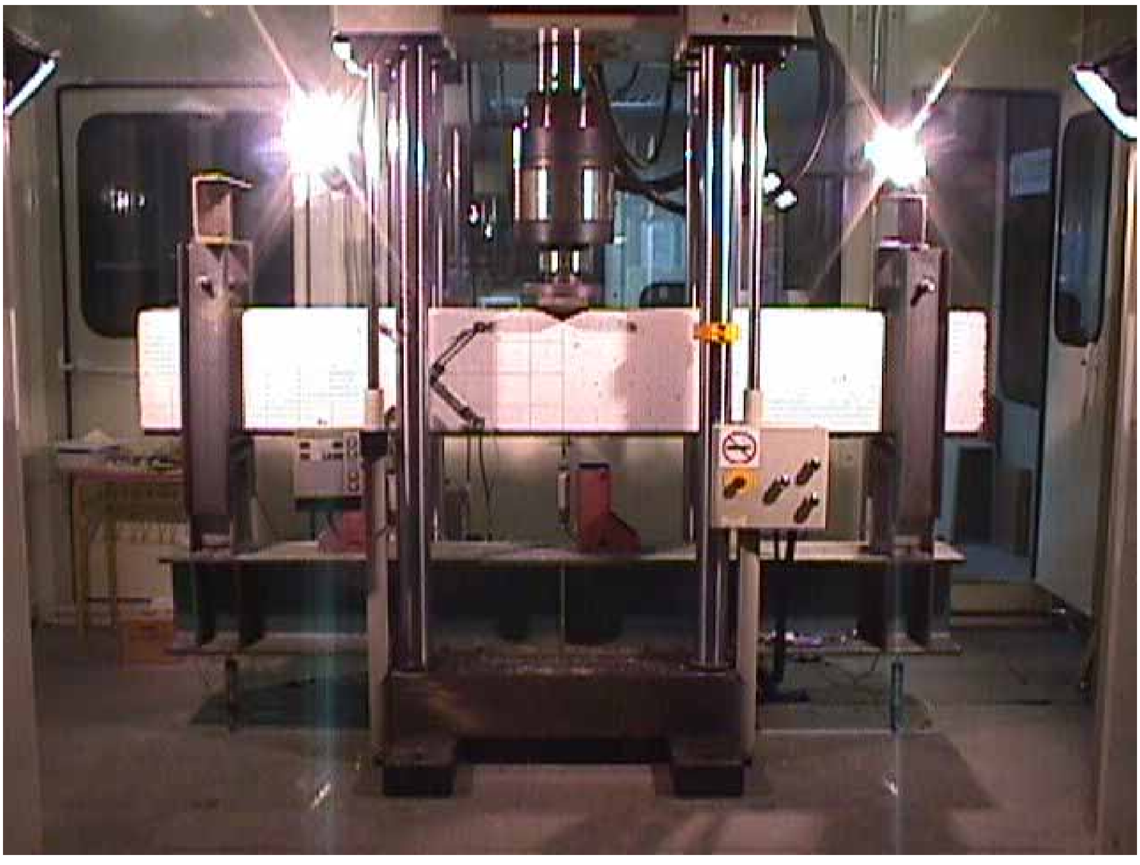

Figure 3.12: Test configuration

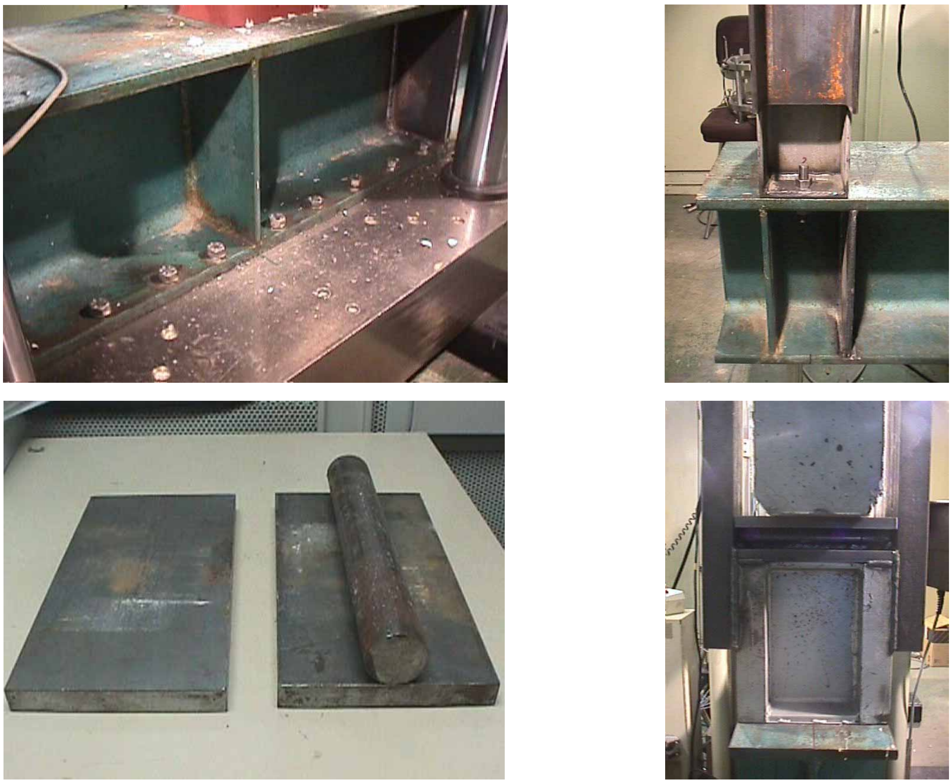

Figure 3.13: Details of the supporting beam and bearings.

For safety reasons, the bearings were located inside a U structure comprised of two UPN 140 to ensure the stability of the beam after failure. The photographs in Figure 3.13 give a detailed view of the supporting beam and the bearings.

3.5.2 Loading Procedure

The tests were carried out under displacement control using a closed-loop hydraulic Instron 8505 compression machine with a loading capacity of 1000 KN. The loading rate was varied slightly for each beam based on the amount of transversal and longitudinal reinforcement so that each would have approximately the same time to failure of 45 minutes. Table 3.4 shows the loading rate for each beam specimen along with the duration of the test. The tests were never stopped after the flexural cracking load was reached. During testing, crack developing was monitored, although crack widths were not controlled for safety reasons due to the fragility of the behaviour that was expected of high-strength concrete beams failing in shear.

Beam specimen	Loading rate mm/s	Test duration min	Beam specimen	Loading rate mm/s	Test duration min
H50/1	0.003	37	H75/1	0.003	33
H50/2	0.005	48	H75/2	0.005	50
H50/3	0.006	55	H75/3	0.005	63
H50/4	0.006	54	H75/4	0.006	44
H50/5	0.006	30	H100/1	0.003	50
H60/1	0.003	36	H100/2	0.005	47
H60/2	0.005	45	H100/3	0.005	51
H60/3	0.005	63	H100/4	0.006	43
H60/4	0.006	50	H100/5	0.003	42

Table 3.4: Loading procedure and test duration.