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SHEAR STRENGTH OF BEAMS OF HIGH STRENGTH CONCRETE

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Preface

This report is a part of the work carried out by Chen Ganwei during his Ph.D study at the Department of Structural Engineering, the Technical University of Denmark, Prof. Dr. Techn. M. P. Nielsen acted as supervisor during the study. The work was supported by AEC Consulting Engineers Ltd.

Resume

Adskillige grupper af forskydningsforsøg med bjælker udført af højstyrkebeton fundet i litteraturen er blevet sammenlignet med teoretiske løsninger. Overenstemmelse mellem forsøg og teori er fundet tilfredsstillende.

Summary

Several groups of shear tests of beams with high strength concrete, which are found so far in the literature, have been compared with the theoretical solutions of beam shear. The agreement between tests and theory has been found to be satisfactory.

List of Symbols

a	clear shear span
A_s	cross sectional area of horizontal tensile reinforcement
A_{swv}	cross sectional area of vertical web reinforcement per unit length
b	web width of beam
C	parameter, $C = \begin{cases} 1-2\Phi^* & \Phi^* \leq 1/2 \\ 0 & \Phi^* > 1/2 \end{cases}$
C_v	coefficient of variation
D	parameter, $D = \begin{cases} (\mu - 2\Phi^*)/\lambda & \Phi^* \leq \mu/2 \\ 0 & \Phi^* \geq \mu/2 \end{cases}$
f_c	mean uniaxial compressive strength of concrete
f_c^*	plastic compressive strength of concrete, defined as $f_c^* = \nu f_c$
f_t	uniaxial tensile strength
f_t^*	plastic tensile strength of concrete, defined as $f_t^* = \rho^* f_c^*$
f_y	yield strength of tensile reinforcement
f_{yw}	yield strength of web reinforcement
h	total depth of cross section
h^*	effective shear depth
h_e	effective depth of cross section
k	material constant, $k = (1 + \sin\varphi)/(1 - \sin\varphi)$ (φ angle of friction)
n	number of tests
V	shear force
V_{cal}	calculated ultimate shear force
V_y	yield load
V_{test}, V_t	observed ultimate shear force in test

$\overline{x}$	mean value
η	ratio, $\eta = \lambda/\mu$
λ	material constant, $\lambda = 1 - (k - 1)\rho^*$
μ	material constant, $\mu = 1 - (k + 1)\rho^*$
ν	effectiveness factor for the compressive strength of concrete
ρ^*	effectiveness factor, $\rho^* = f_t^*/f_c^*$ for the tensile strength of concrete
σ	standard deviation
τ	shear stress defined as $V/(bh)$ or $V/(bh^*)$
φ	reinforcement ratio, defined as A_s/bh, angle of friction
φ_v	vertical reinforcement ratio
Φ^*	effective horizontal reinforcement degree, $\Phi^* = A_s f_y/(bhf_c^*)$
Φ_h^*	effective horizontal reinforcement degree, $\Phi_h^* = A_s f_y/(bh^* f_c^*)$
Φ_v^*	effective vertical web reinforcement degree, $\Phi_v^* = A_{swv} f_{yw}/(bf_c^*)$
Φ_{vo}^*	critical vertical of web reinforcement degree

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1. Introduction

As is well known, concrete ductility decreases appreciably with increasing strength. Therefore it is necessary to check the validity of theoretical shear solutions when applied to high concrete strengths.

2. Theoretical solutions

The theoretical solutions will not be derived here. The reader is referred to [2], [3], [6], [7], [8] and [9].

2.1. Beams without shear web reinforcement.

2.1.1 Complete solutions

The most complete theory of shear strength is based on a modified theory of plasticity. The empirical formulas still used in many countries will not be evaluated in this report.

The theoretical plastic solutions are modified by replacing the compressive strength of concrete f_c by $f_c^* = \nu f_c$ and the tensile strength f_t by $f_t^* = \rho^* f_c^*$.

The complete plastic solutions [3], in which the effect of the tensile strength of concrete has been considered are shown in figure 2.1.

$\lambda + \mu + D \geq 1$

$\frac{a}{h}$ \ $\frac{\tau}{f_c^*}$ \ ϕ^*	$\phi^* \leq \frac{\mu}{2}$	$\frac{\mu}{2} \leq \phi^* \leq \frac{1}{2}$	$\phi^* \geq \frac{1}{2}$
$\sqrt{\frac{1-D^2}{\eta^2-1}} \leq \frac{a}{h}$	$\frac{1}{2}\sqrt{(\lambda^2-\mu^2)(1-D^2)}$	$\frac{1}{2}\sqrt{\lambda^2-\mu^2}$	
$\tan\varphi - \frac{D}{\cos\varphi} \leq \frac{a}{h} \leq \sqrt{\frac{1-D^2}{\eta^2-1}}$	$\frac{1}{2}[\lambda\sqrt{(\frac{a}{h})^2+1-D^2} - \mu\frac{a}{h}]$	$\frac{1}{2}[\lambda\sqrt{(\frac{a}{h})^2+1} - \mu\frac{a}{h}]$	
$\tan\varphi - \frac{C}{\cos\varphi} \leq \frac{a}{h} \leq \tan\varphi - \frac{D}{\cos\varphi}$	$\frac{[(\frac{a}{h})^2+1](1-\sin\varphi) - 2\phi^*(\frac{a}{h}\cos\varphi - \sin\varphi)}{2(\frac{a}{h}\sin\varphi + \cos\varphi)}$		
$\frac{a}{h} \leq \tan\varphi - \frac{C}{\cos\varphi}$	$\frac{1}{2}[\sqrt{(\frac{a}{h})^2+1-C^2} - \frac{a}{h}]$		$\frac{1}{2}[\sqrt{(\frac{a}{h})^2+1} - \frac{a}{h}]$

$$\rho^* = \frac{f_t^*}{f_c^*}, \qquad k = \frac{1+\sin\varphi}{1-\sin\varphi} = 4, \qquad \varphi = 37^{\circ}$$

$$\lambda = 1 - \rho^*(k-1) = 1 - 3\rho^*, \qquad \eta = \frac{\lambda}{\mu}$$

$$\mu = 1 - \rho^*(k+1) = 1 - 5\rho^*,$$

$$\frac{\tau}{f_c^*} = \frac{V}{bhf_c^*}, \qquad \phi^* = \frac{A_s f_y}{bh\, f_c^*}$$

$$C = \begin{cases} 1 - 2\phi^* & \text{for } \phi^* \leq \frac{1}{2} \\ 0 & \text{for } \phi^* > \frac{1}{2} \end{cases}, \qquad D = \begin{cases} \frac{\mu - 2\phi^*}{\lambda} & \text{for } \phi^* \leq \frac{\mu}{2} \\ 0 & \text{for } \phi^* > \frac{\mu}{2} \end{cases}$$

Figure 2.1 The complete plastic solutions and their boundaries.

The corresponding empirical ν–formula with $\rho^* = 0.03$ has been found as

$$\rho^* = 0.03 \tag{2.1}$$

$$\nu = \frac{0.35(3.5-\frac{a}{h})\ (\varphi+2)\ (1-0.2h)}{\sqrt{f_c}} \leq 1.0 \qquad \begin{bmatrix} \frac{a}{h} \ngtr 2.5 \\ \varphi \ngtr 2.0 \\ h \ngtr 1.25 \end{bmatrix} \tag{2.2}$$

Here, f_c is in MPa, φ in percent and h in meter.

The tests of beams without shear reinforcement have been compared with the theoretical formulas.

Examples of the dependence of ν on f_c, a/h, φ and h in beams with high strength concrete can be seen in figures 2.2, 2.3, 2.4 and 2.5, respectively.

The statistics of the ratios of test to theory and the comparison between test results and theoretical calculations are presented in figure 2.6.

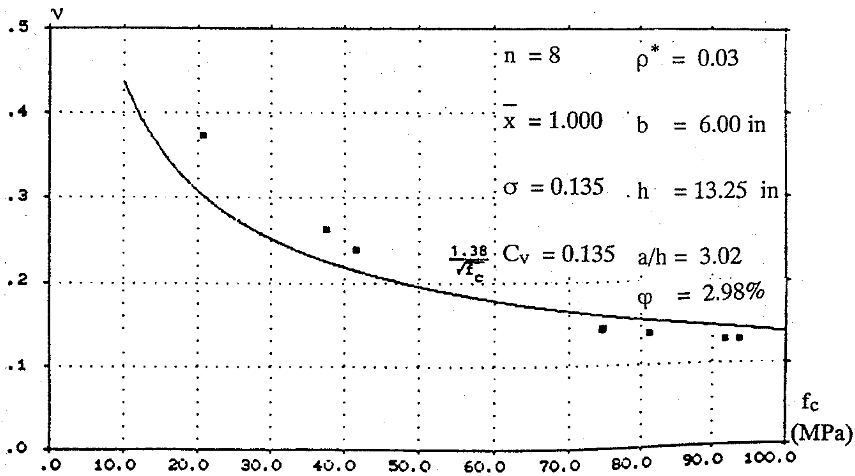


Figure 2.2 The ν–dependence on f_c.

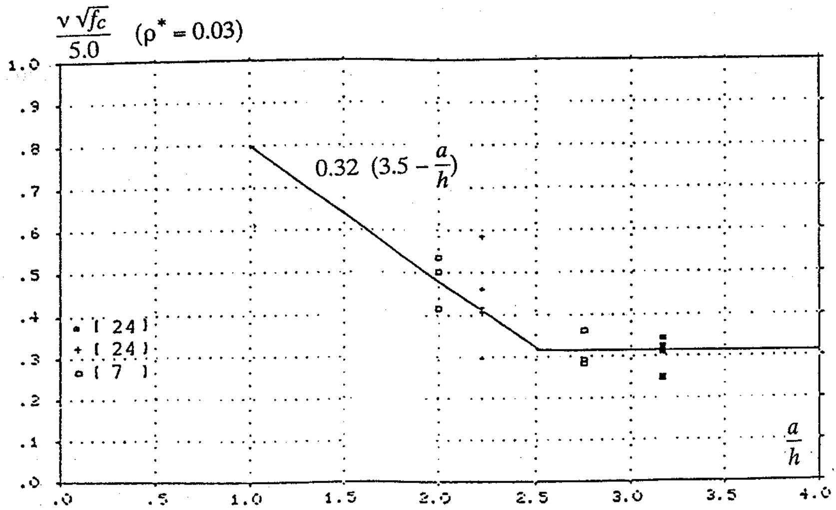


Figure 2.3 The ν–dependence on a/h.

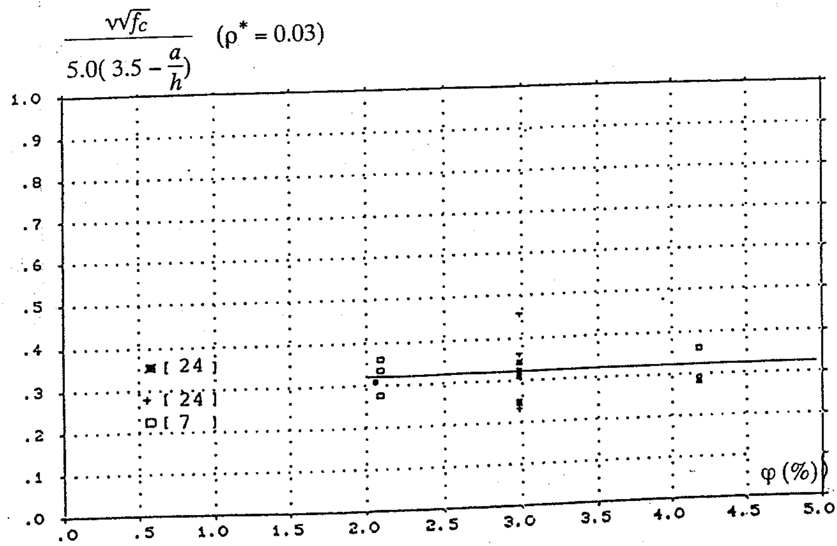


Figure 2.4 The ν–dependence on φ.

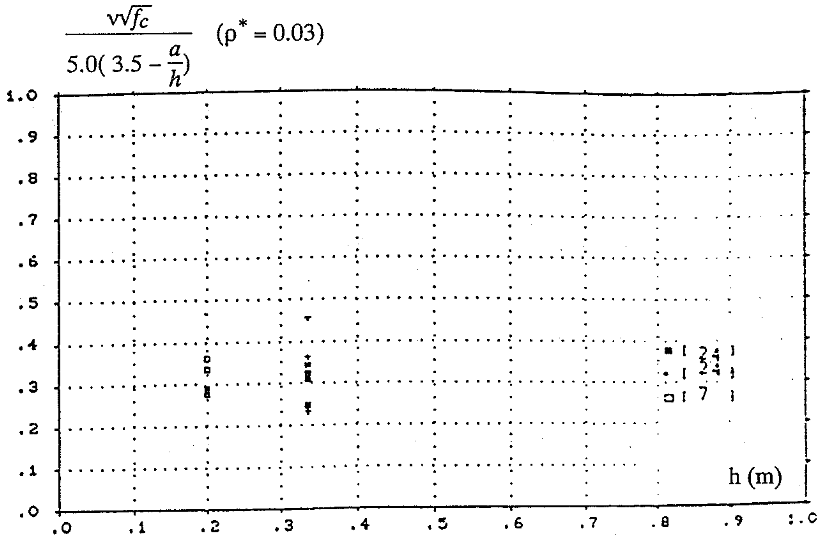


Figure 2.5 The ν–dependence on h.

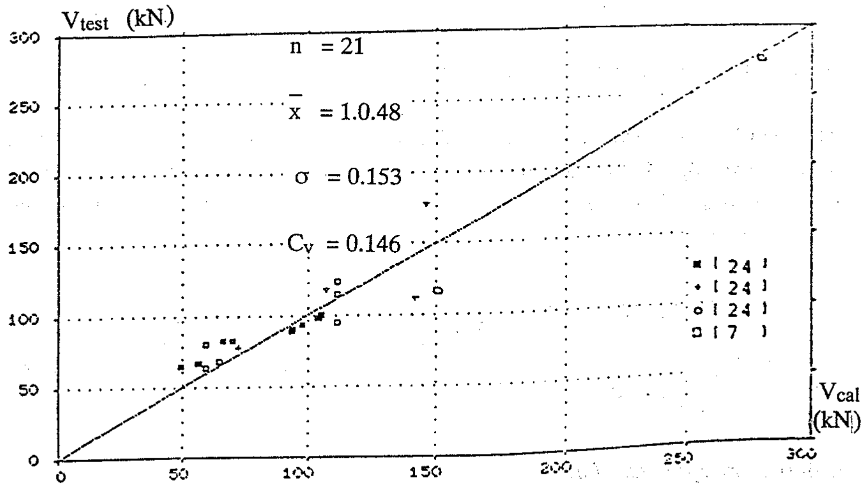


Figure 2.6 Comparison between theory and tests.

2.1.2 Simplified solutions

Inserting $\rho^{*} = 0$ into the complete plastic shear solutions, i.e. the tensile strength of concrete is neglected, we get the simple shear solutions.

$$\frac{\tau}{f_c^{*}} = \begin{cases} \frac{1}{2}\left[\sqrt{\left(\frac{a}{h}\right)^2+4\Phi^{*}(1-\Phi^{*})} - \frac{a}{h}\right] & \text{for } \Phi^{*} \le \frac{1}{2} \\ \frac{1}{2}\left[\sqrt{\left(\frac{a}{h}\right)^2+1} - \frac{a}{h}\right] & \text{for } \Phi^{*} > \frac{1}{2} \end{cases} \qquad (2.3)$$

These solutions were given by Nielsen and Bræstrup as exact plastic solutions in [6] and [7].

The corresponding ν formula was suggested by Roikjær [9]

$$\nu = \frac{3.5}{\sqrt{f_c}}\, 0.27\left(1+\frac{1}{\sqrt{h}}\right)(0.15\,\varphi+0.58)\left(1.0+0.17\left(\frac{a}{h}-2.6\right)^2\right) \qquad (2.4)$$

Here, f_c is in MPa, φ in percent and h in meter

This solution has some disadvantages. The most simple way to eliminate these is to introduce an upper limit of 2.5 in the shear span ratio in the simple solutions, i.e. when the shear span ratio is larger than 2.5 it is put equal to 2.5. The modified simple solutions therefore are

$$\frac{\tau}{f_c^{*}} = \begin{cases} \frac{1}{2}\left[\sqrt{\left(\frac{a}{h}\right)^2 + 4\Phi^{*}(1-\Phi^{*})} - \frac{a}{h}\right] & \left(\frac{a}{h} \ngtr 2.5\right) \quad \text{for } \Phi^{*} \le \frac{1}{2} \\ \frac{1}{2}\left[\sqrt{\left(\frac{a}{h}\right)^2 + 1} - \frac{a}{h}\right] & \qquad\qquad\quad \text{for } \Phi^{*} > \frac{1}{2} \end{cases} \qquad (2.5)$$

with the corresponding ν formula

$$\nu = \frac{0.60(2-0.4\frac{a}{h})\ (\varphi+2)\ (1-0.25h)}{\sqrt{f_c}} \le 1.0 \qquad \begin{bmatrix} \frac{a}{h} \ngtr 2.5 \\ \varphi \ngtr 2.0 \\ h \ngtr 1.0 \end{bmatrix} \tag{2.6}$$

The comparison between tests and the modified simple solution show that this solution is almost as good as the complete solution taken into account the tensile strength.

2.2 Beams with vertical stirrups.

For beams with vertical stirrups subjected to concentrated loads the shear carrying capacity was derived by Nielsen and Bræstrup [2] [8] using the web crushing criterion. The complete solutions are given in figure 2.7.

ϕ_h^* / τ/f_c^* / ϕ_v^*	$\phi_h^* \le \frac{1}{2}$	$\phi_h^* = \frac{1}{2}$
$\phi_v^* < \phi_{vo}^*$	$\frac{1}{2}\left[\sqrt{\left[\frac{a}{h^*}\right]^2+4\phi_h^*(1-\phi_h^*)}-\frac{a}{h^*}\right]+\phi_v^*\cdot\frac{a}{h^*}$	$\frac{1}{2}\left[\sqrt{\left[\frac{a}{h^*}\right]^2+1}-\frac{a}{h^*}\right]+\phi_v^*\cdot\frac{a}{h^*}$
$\phi_{vo}^* \le \phi_v^* \le \frac{1}{2}$	$2\sqrt{\phi_h^*(1-\phi_h^*)\phi_v^*(1-\phi_v^*)}$	$\sqrt{\phi_v^*(1-\phi_v^*)}$
$\phi_v^* > \frac{1}{2}$	$\sqrt{\phi_h^*(1-\phi_h^*)}$	$\frac{1}{2}$

(2.7)

$$\phi_{vo}^* = \begin{cases} \dfrac{\sqrt{\left[\frac{a}{h^*}\right]^2+4\phi_h^*(1-\phi_h^*)} - \frac{a}{h^*}}{2\sqrt{\left[\frac{a}{h^*}\right]^2 + 4\phi_h^*(1-\phi_h^*)}} & \text{valid for } \phi_h^* \le \frac{1}{2} \\[2ex] \dfrac{\sqrt{\left[\frac{a}{h^*}\right]^2+1} - \frac{a}{h^*}}{2\sqrt{\left[\frac{a}{h^*}\right]^2 + 1}} & \text{for } \phi_h^* > \frac{1}{2} \end{cases}$$

Figure 2.7.

Here, the effective shear depth h^* is defined as the distance between the tension and compression stringers. The Danish Code of Practice DS 411 requires the use of the internal moment lever arm "z" calculated at the section of maximum moment as the effective shear depth h^*.

For T, or I–section beams, we may identify the distance between the center of the compression flange and the centroid of tensile reinforcement as the effective shear depth h^*.

For conventional slender beams, in which the longitudinal reinforcement is governed by the bending moment calculation, the shear calculation becomes more simple. The solution for the shear carrying capacity is in this case.

$$\frac{\tau}{f_c^*} = \begin{cases} \sqrt{\Phi_v^* (1 - \Phi_v^*)} & \Phi_v^* \leq 0.5 \\ 0.5 & \Phi_v^* > 0.5 \end{cases} \tag{2.8}$$

For such beams, ν may be considered as being only depending on the concrete strength. As an average, the straight line

$$\nu = 0.7 - \frac{f_c}{200} \nless 0.5 \qquad (f_c \text{ in MPa}) \tag{2.9}$$

can be used.

3. Comparison with tests

3.1 Beams without shear reinforcement

Mphonde, A. G. and Frantz, G. C. [5] have tested three series of reinforced concrete beams without shear reinforcement to determine their diagonal cracking strength and ultimate shear capacities. In each series the shear span–depth ratio a/d was held constant at 1.5, 2.5 or 3.6, while the nominal concrete compressive strength was varied from 21 to 103 MPa. The dimensions are given in figure 3.1.

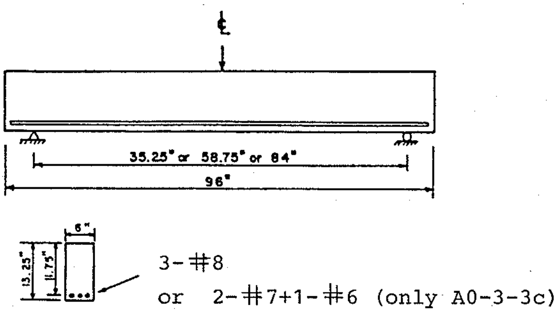


Figure 3.1 Test specimens (1 in. = 25.4 mm).

BEAM MARK	a in	f_c KSI	A_s in^2	f_y KSI	V_t KIPS	Failure mode
A0--3-3B	42.00	3.011	2.37	60.0	14.50	Diagonal tension
A0--3-3C	42.00	3.935	1.64	60.0	15.00	Diagonal tension
A0--7-3A	42.00	5.463	2.37	60.0	18.50	Diagonal tension
A0--7-3B	42.00	6.037	2.37	60.0	18.60	Diagonal tension
A0-11-3A	42.00	10.867	2.37	60.0	20.20	Diagonal tension
A0-11-3B	42.00	10.825	2.37	60.0	20.10	Diagonal tension
A0-15-3A	42.00	11.797	2.37	60.0	21.00	Diagonal tension
A0-15-3B	42.00	13.587	2.37	60.0	22.50	Diagonal tension
A0-15-3C	42.00	13.319	2.37	60.0	22.00	Diagonal tension
A0--3--2	29.38	2.986	2.37	60.0	17.50	Shear compression
A0--7--2	29.38	6.549	2.37	60.0	26.50	Shear compression
A0-11--2	29.38	11.498	2.37	60.0	25.00	Diagonal tension
A0-15-2A	29.38	12.148	2.37	60.0	40.00	Shear compression
A0-15-2B	29.38	10.065	2.37	60.0	46.20	Shear compression
A0--3--1	17.63	3.346	2.37	60.0	26.10	Shear flexure
A0-15-1A	17.63	11.524	2.37	60.0	62.00	Shear fluxure

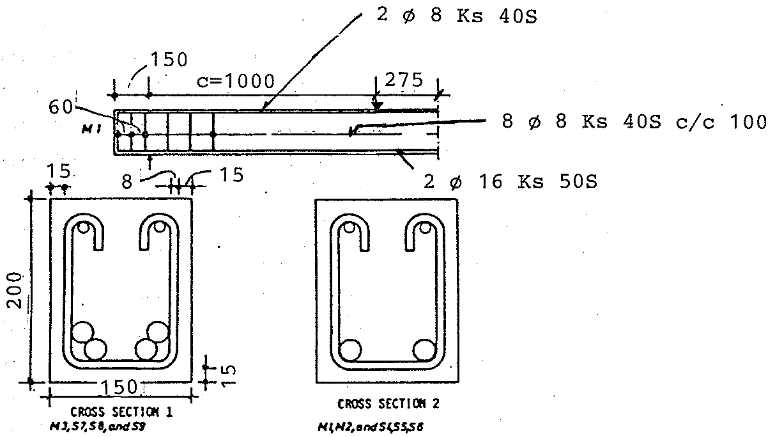
Figure 3.1 Continued

The results of these test series have been utilized in the determination of the effectiveness factor, see section 2.1.

3.2 Beams with shear reinforcement

Two groups of shear tests of beams with high strength concrete and vertical stirrups have been found in the literature [1] [4].

Bernhardt, C. J. and Fynboe, C. C. [1] has reported test results for beams in high strength concrete with ordinary aggregates. The tests included beams with cube strength up to 125 MPa. 6 beams without shear reinforcement and 5 beams with shear stirrups was observed to be failing in shear. For beams with shear reinforcement the open stirrups were used. The dimensions of the beams and the arrangements of reinforcement are shown in figure 3.2.



Cross section of beams with reinforcement and typical arrangement of loads.

Measurements:mm

Figure 3.2 Cross section of beams with reinforcement and typical arrangement of loads.

BEAM MARK	H_o cm	a cm	f_c MPa	A_s cm^2	f_y MPa	V_t kN	Failure mode
S-----6A	16.7	40	83.2	6.28	510	95	Shear failure
S-----6B	16.7	40	83.2	6.28	510	123	Shear failure
S-----6C	16.7	40	83.2	6.28	510	115	Shear failure
S-----9A	16.0	55	83.2	12.56	510	80	Shear failure
S-----9B	16.0	55	83.2	12.56	510	64	Shear failure
S-----9C	16.0	55	98.4	12.56	510	68	Shear failure

BEAM MARK	H_o cm	a cm	f_c MPa	A_s cm^2	f_y MPa	r_v %	f_{yw} MPa	V_t kN	Failure mode
S--5-A	16.7	40	83.2	6.28	510	0.447	427	110	Shear failure
S--7-A	16.0	55	83.2	12.56	510	0.673	427	140	Shear failure
S--7-B	16.0	55	83.2	12.56	510	0.673	427	150	Shear failure
S--8-A	16.0	55	83.2	12.56	510	0.447	427	125	Shear failure
S--8-B	16.0	55	83.2	12.56	510	0.447	427	135	Shear failure

Figure 3.2 Continued

For the data reported in [1], it has been found that the shear capacity is only 60–70% of that predicted by theory because of the bad arrangement of the open stirrups, see figure 3.2.

By Levi, F. and Marro, P. [4] seven beams, geometrically identical, were tested according to the plan in figure 3.3. In all cases transversal reinforcement was high bend ø 12 mm two legs stirrups with different spacing.

In these tests the shear span a = 3.80 m is valid for all beams. When comparing with the theory, the effective shear depth h^* is taken as the distance between the centroid of tensile reinforcement and the middle line of the compressive flange.

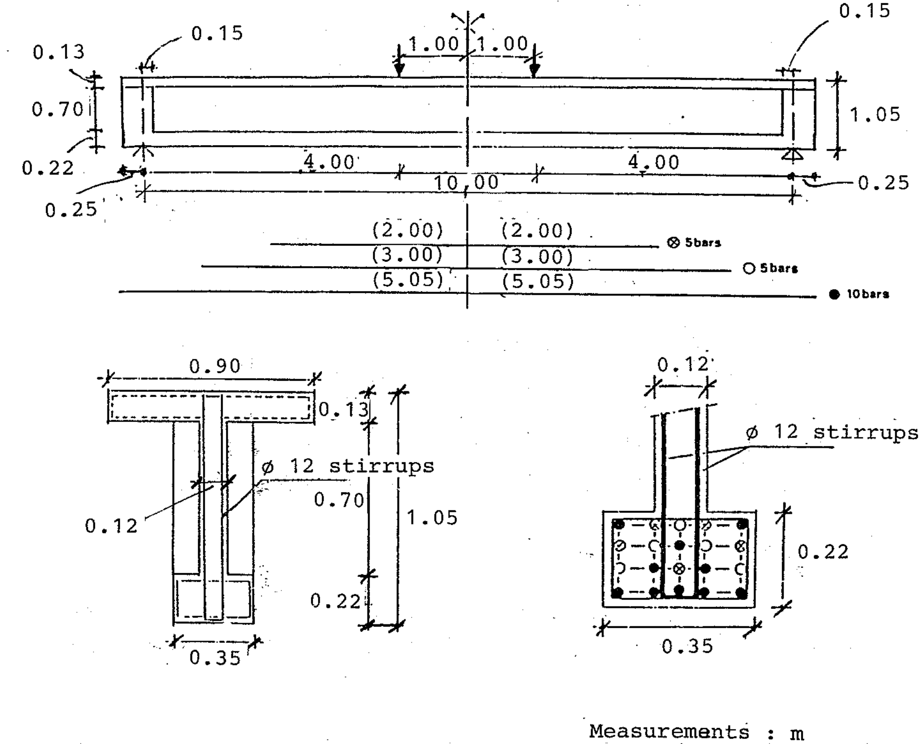


Figure 3.3 Test specimen.

BEAM MARK	f_c MPa	A_s cm^2	f_y MPa	r_v %	f_{yw} MPa	V_y kN	V_t kN	Failure mode
RC30A1	25.0	90.48	500	0.837	480	652	676	Shear failure
RC30A2	25.0	90.48	500	0.837	480	661	688	Shear failure
RC60A1	47.0	106.19	450	0.837	480	870	990	Shear failure
RC60A2	47.0	106.19	450	0.837	480	870	938	Shear failure
RC60B1	50.0	141.37	470	1.256	480	1140	1181	Shear failure
RC60B2	50.0	141.37	470	1.256	480	1160	1239	Shear failure
RC70B1	60.0	141.37	470	1.256	480	1280	1330	Shear failure

Figure 3.3 Continued

The general applicability of the web crushing criterion for beams with high strength of concrete is demonstrated by figure 3.4.

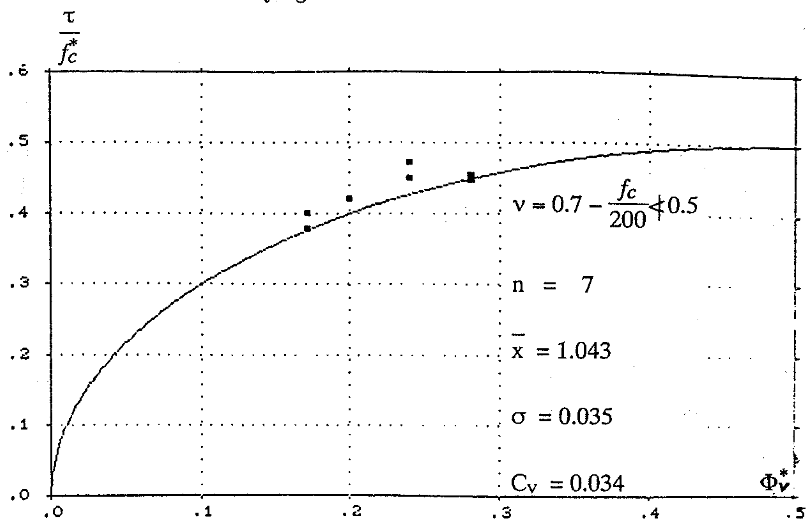


Figure 3.4 The plastic solution of beams with shear reinforcement compared with tests of high strength concrete beams [4].

For this series of large T–beams, the constant $\nu = 0.55$ is even better than formula (2.9). In this case, for 7 beams failing in shear, the mean value is 1.008, while both the standard deviation and the coefficient of variation are 0.03.

The statistics of the ratios of tests [4] to predicted values by formulae (2.7) and (2.8) and the comparison is depicted in figure 3.5.

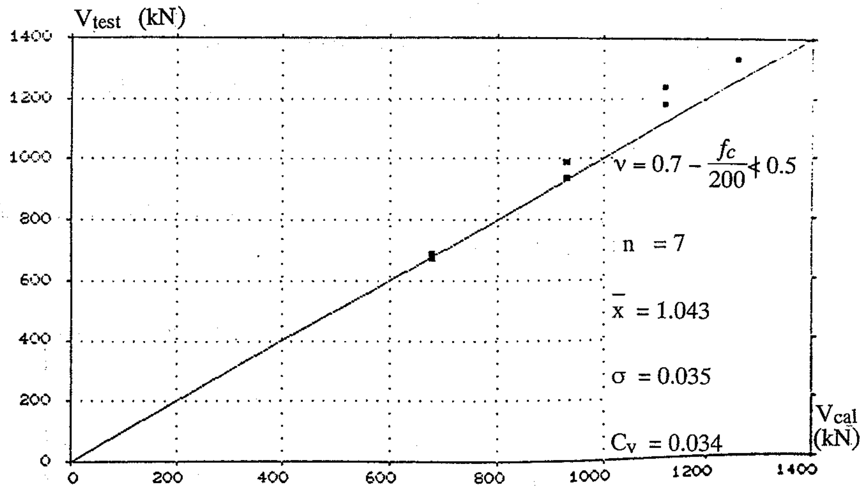


Figure 3.5 Comparison of test results with theoretical calculations of high strength concrete beams with web reinforcement [4].

4. Conclusion

Based on this analysis, it can be concluded that the theoretical solutions based on a modified theory of plasticity, without any difficulties can be extended to high strength concrete.

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