Earthquakes and Structures, Vol. 4, No. 3 (2013) 265-283
DOI: http://dx.doi.org/10.12989/eas.2013.4.3.265

Design in shear of reinforced concrete short columns

M.L. Moretti[*1] and T.P. Tassios[2]

[1]*Faculty of Engineering (Civil),University of Thessaly, Volos, Greece*
[2]*Faculty of Engineering (Civil), NTUA, Athens, Greece*

(Received January 8, 2012, Revised April 25, 2012, Accepted May 25, 2012)

Abstract. This research was prompted by the paucity of specific code provisions regarding the design of short columns for shear. The purpose of this paper was to investigate whether the use of the normal shear design procedure of various codes may or may not be applied to reliably calculate the shear strength of short columns. Provisions of the codes American ACI 318M-08, Canadian CSA A23.3-04, Japanese AIJ Guidelines, New Zealand NZS 3101, European EN 1998 (EC8) parts 1 and 3, combined with EN 1992-1-1 (EC2), and draft fib Model Code 2010, as well as a strut-and-tie model are applied on short columns tested under cyclic loading that failed in shear. Actual shear resistances are compared to predictions, and the resulting shortcomings of the codes are identified. EN1998-3 appears to be the only code among those considered that may be reliably applied to estimate the shear resistance of short columns. Further, the proposed strut-and tie model can be a useful tool for the detailed design and assessment of short columns.

Keywords: short column; shear resistance; codes; safety; confinement; strut-and-tie

1. Introduction

Columns with low length-to-depth ratio are prone to brittle behavior when subjected to reversed loading; the shorter the column the more the brittleness (Moretti and Tassios 2007, Galal *et al*. 2005, Yamada and Furui 1968). Short columns may occur either because the original column length-to-depth ratio is small, or due to the presence of an obstacle along a certain height of the column (e.g. low brick masonry wall) in which case the effective length of the column is reduced and equals the column's unflanked length. The sudden and explosive nature of shear failure in a short column has been first pointed out by Yamada and Furui in 1968. Although considerable experimental and theoretical research has been conducted on this subject, especially in the 1970's and 80's, neither a generally accepted way of predicting the behavior of short columns exists, nor special provisions concerning the design of such elements are included in the majority of codes.

In Greece it was only after the destructive earthquake of Athens in 1999 (during which many partial and total collapses occurred due to failure of short columns) that special provisions were incorporated in the Greek national code. Similar provisions have been recently included in a relevant national Greek Annex to EC8.

[*]Corresponding author, Ph.D., E-mail: marmor@civ.uth.gr

Shear design provisions in the current codes are applicable to non-discontinuity regions of linear elements and have resulted from extensive research conducted on beams. Consequently, they are not intended for the design of short columns.

Available models especially formulated to describe the shear behavior of short columns presuppose a superimposition of some kind of truss and of an arch mechanism that represent the contribution to shear resistance of the reinforcement and of the compressive concrete diagonal strut (arch), respectively (Kowalsky and Priestley 2000, Priestley *et al.* 1994, Tegos and Penelis 1988, Minami and Wakabayashi 1981, Shohara and Kato 1981). The resulting equation is the sum of the resistances contributed by each of these load-transfer mechanisms. The activation of the arch is enhanced for lower length-to-depth ratios, while for larger ratios the truss mechanism prevails. In the ultimate state of design of short elements the interaction of truss and arch mechanism is crucial, provided that detailing will enable their full activation. Thus, sufficient longitudinal reinforcement is a prerequisite so that premature flexural failure does not occur. However, this requirement stands in opposition to the objective of ensuring a ductile mode of failure through yielding of the longitudinal reinforcement.

The lack of a reliable method to evaluate the contribution and the influence of the various parameters involved in the design of short columns is significant. In new structures short columns may be avoided in the light of recent knowledge concerning their inadequate seismic behavior. But in the assessment of existing structures a reliable estimation of the structural behavior of any present short column is indispensable (Eleftheriadou and Karabinis 2012, Sullivan 2010, Teran-Gilmore *et al.* 2010).

Codified shear strength design methods are intended to provide a conservative and lower bound solution, and they cannot be considered as models for predicting the actual shear strength. Thus, given the fact that no generally accepted theory for the prediction of shear strength of short columns is available, the objective of this paper is to examine if, and under which circumstances, some widely used codes could be possibly applied to safely calculate the shear strength of short columns. To evaluate the predictive power of the codes examined, experimental results of short columns that failed in shear were selected. The selected columns had, all but one, a length-to-depth ratio $L/h \leq 3.0$ so that they may be considered as short columns (EN1998-1), and therefore the interaction between truss and arch mechanism is rendered more pronounced. Furthermore, the majority of specimens had high percentage of shear and longitudinal reinforcement so that the load transfer mechanisms could be fully activated.

A strut-and-tie model (Moretti and Tassios 2006) is also applied to calculate the shear strength of the specimens considered. To demonstrate the suitability of this model in describing the failure conditions of short columns some applications are presented in Appendix B.

2. Shear design code provisions

Shear design provisions in the current codes are applicable to linear elements in non-discontinuity regions. They have resulted from extensive experimental research conducted on beams, with or without web reinforcement, over the past 70 years. The dramatic differences between codes regarding the concept upon which the shear design method each code adopts is based, have often been pointed out (NCHRP 2005, Collins *et al* 2008). The methods for shear design consist in (1) sectional models derived either empirically from test data, or from equilibrium models, or from a combination of both, (2) smeared crack models (MCFT, Vecchio and Collins 1986) and (3) strut-and-tie models. The prevailing model, however, for describing the flow of shear forces in a linear element is still the parallel chord truss model. The concrete contribution to shear resistance results from shear transfer in the compression zone, as well as along crack interfaces. Even in cases in which codes adopt the same design method, variations in the range and limits of design parameters may lead to significant variability of the results.

Strut-and-tie models, incorporated in many codes in the last decade, may be applied to simulate the force transfer at ultimate limit state, particularly in discontinuity regions. Strut-and-tie models offer flexibility but, precisely because of their potential versatility, they incorporate inherent parameters (i.e., form of the model, proportioning of struts) the choice of which may lead to considerable difference in results. It should be pointed out that code provisions for strut-and-tie models are mainly applicable to beams, and may not be directly used in the case of short columns.

Short columns, because of their low length-to-depth ratio, constitute a discontinuity region along their entire height. Therefore, they ought to be designed by use of a strut-and-tie model (Hong *et al.* 2011, Kim *et al.* 2011). However, no specific relevant code guidelines are available. More precisely, the vast majority of current codes do not even refer to the design of short columns (Kotsovos 2007, 2008, Moretti 2008). Among the codes discussed in this work, only the JCI Guidelines offer a procedure for design of elements with low length-to-depth ratio (presented in detail in Appendix A). Furthermore, EN1998-1 includes some construction details, and EN1998-3 offers an upper limit for the shear strength of short columns.

Given the rarity of specific code provisions for the design of short columns, this work was aimed at investigating the adequacy of the normal shear design provisions in various codes regarding the reliable estimation of the shear strength of short columns.

3. Characteristics of the shear design provisions of the codes

In this paper the CSA simplified method has been used (Bentz *et al*. 2006). Furthermore, the draft Model Code 2010 Approximation Level II is applied based on the conventional European concepts of plasticity and modified with an estimate of the strain state at shear failure (Bentz 2011). Predictions according to Eurocodes are based on EN1998-1 and EN1998-3 (assessment of existing structures), combined with EN1992-1-1.

In all the codes examined shear strength is in general determined as the sum $V=V_c+V_s$, where V_c is the shear resistance attributed to concrete and to longitudinal reinforcement, and V_s the shear resistance provided by shear reinforcement. The term V_c in CSA and in draft MC 2010 results from taking into account the strain and stress condition of cracked concrete, in AIJ it is calculated by the lower bound theorem, while in ACI and NZS from empirical equations formulated to provide statistical fit to experimental data. The term V_c is ignored in draft MC2010 Level II Approximation, in EN1992-1-1 (therefore also in EC8), and in ACI for earthquake forces and low axial forces, while in CSA it is reduced for seismic design.

Shear resistance V_s, provided by shear reinforcement, is in all cases calculated by truss analogy, the only difference being the adopted value of the angle θ (theta) of the compression strut and the element axis perpendicular to the shear force. The assumed angle θ is significant since lower values lead to increased contribution of stirrups.

In ACI and NZS it is assumed θ=45 degrees, while in CSA for seismic design it is $\theta\geq 45$ degrees. In AIJ cotθ depends on the angle of the arch mechanism, on the concrete strength, and on the amount of shear reinforcement (see Appendix A). In MC 2010 the value of θ is associated to the value of the longitudinal strain ε_x of the member and results through iterations.

In EN1998-1 angle *theta* is variable ($21.8 \leq \theta \leq 45$ degrees) and results through iterations as the optimum angle to fit both the demand in stirrups and also to avoid failure of concrete in compression, according to EN1992-1-1. In EN1998-3 it is assumed θ=45 degrees for the calculation of the resistance of shear reinforcement V_s.

To avoid concrete failure additional upper limits are specified in the codes. They consist mainly in restrictions regarding the maximum permissible values of strength of materials, or a limit in the maximum shear resistance of the element. These restrictions are listed below.

In ACI material strength limitations concern the compressive concrete strength $f_c \leq 69$ MPa and the yield strength of shear reinforcement $f_{yw} \leq 420$ MPa. An upper limit in the contribution of shear reinforcement V_s is also imposed ($V_s \leq 0{,}66\sqrt{f_c}\, b_w d$, f_c in MPa, b_w =web width, d=effective depth, in meters).

In NZS for shear reinforcement calculation the steel strength of stirrups is limited to $f_{wy} \leq 500$ MPa. In calculating the permissible concrete stress $v_b = (0.07+10\rho_s)\sqrt{f_c}$ it is $f_c \leq 50$ MPa, with $0.08\sqrt{f_c} < v_b < 0.2\sqrt{f_c}$ (ρ_s = effective area of flexural tension reinforcement in the section lying between the extreme tension reinforcement and a line located at a distance of one third of the distance between the extreme compression fiber and the extreme tension reinforcement). Additionally, total shear strength V should not exceed the value $A_{cv}\, min(0.2\, f_c$ or 8 MPa), where $A_{cv} = b_w\, d$.

In AIJ the steel strength for ties is limited to $f_{wy} = 25 f_c$ if $f_{wy} >$ (400 MPa, or $25 f_c$). An additional limitation concerns the maximum contribution of shear reinforcement in relation to the concrete strength, $\rho_w f_{wy} = \nu f_c/2$ if $\rho_w f_{wy} > \nu f_c/2$, where ρ_w = shear reinforcement ratio and $\nu = 0.70 - f_c/200$.

In CSA concrete strength should not exceed the value of 64 MPa when calculating the concrete contribution to shear. Shear resistance upper limit is $V_{R,\max} = 0{,}162\, f_c\, b_w\, d$. No limit was assumed for steel yield strength.

In draft MC2010 in Level II of Approximation in use in this work, shear resistance upper limit is $V_{R,max} = k_c f_c\, b_w\, z \cot\theta/(1+\cot^2\theta)$, where $k_c = 0.55(30/f_c)^{1/3} \leq 0.55$.

In EN1998-1 the upper limit of shear resistance for avoiding compressive failure of concrete is derived from EN1992-1-1, and it is $V_{R,\max} = \alpha_{cw}\, b_w z\, \nu f_c/(\cot\theta + \tan\theta)$, where ν=0.60 for $f_c \leq 60$ MPa, and α_{cw} depends on the axial load. In EN1998-3 are considered both the previously mentioned limit, as well as the following empirical Eq. (1) which is adequate for columns with length-to-depth ratio $L/h \leq 4$, where $\delta = L/h$, γ_{el}=1,15, ρ_{tot}= total longitudinal reinforcement ratio, $\mu_\Delta^{pl} = \theta_{pl}/\theta_y$, θ=chord rotation at member end, z=internal lever arm, ($V_{R,max}$ in MN, f_c in MPa, b_w and z in m). In this work μ_Δ^{pl} =1 has been assumed.

$$V_{R,\max} = \frac{4/7}{\gamma_{el}}(1-0{,}02\min(5;\mu_\Delta^{pl}))\left(1+1{,}35\frac{N}{A_c f_c}\right)(1+0{,}45(100\rho_{tot}))\sqrt{\min(40;f_c)}b_w z \sin 2\delta \qquad (1)$$

Table 1 Characteristics of the specimens

Spec.	Label	Ref.	*b* (m)	*d* (m)	L/h	Axial load ratio *ν*	f_c (MPa)	Tensile reinf. ρ_s (%)	f_{sy} (MPa)	Ties and cross ties	f_{wy} (MPa)	$\rho_w f_{wy}$ (MPa)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	HT6-4BL	a	0.30	0.26	3	0.17	61.1	1.47	959	4∅5@80	1287	4.25
2	HT6-4CL	a	0.30	0.26	3	0.17	61.1	1.47	959	4∅7.4@80	1275	8.54
3	HT6-2AH	a	0.30	0.26	3	0.33	61.1	1.47	959	2∅5@60	1287	2.83
4	HT6-2BH	a	0.30	0.26	3	0.33	61.1	1.47	959	2∅5@40	1287	4.25
5	HT6-4BH	a	0.30	0.26	3	0.33	61.1	1.47	959	4∅5@80	1287	4.25
6	HT6-4CH	a	0.30	0.26	3	0.33	61.1	1.47	959	4∅7.4@80	1275	8.54
7	CA06-6-1	a	0.30	0.26	3	0.17	72.1	1.47	742	4∅6@80	365	1.93
8	CA06-6-2	a	0.30	0.26	3	0.17	72.1	1.47	742	4∅10@80	405	4.82
9	CA06-6-3	a	0.30	0.26	3	0.17	72.1	1.47	742	4∅6@80	875	4.64
10	CA06-6-4	a	0.30	0.26	3	0.17	72.1	1.47	742	4∅10@80	1053	12.46
11	CA06-3-1	a	0.30	0.26	3	0.33	72.1	1.47	742	4∅6@80	365	1.93
12	CA06-3-2	a	0.30	0.26	3	0.33	72.1	1.47	742	4∅10@80	405	4.82
13	CA06-3-3	a	0.30	0.26	3	0.33	72.1	1.47	742	4∅6@80	875	4.64
14	CA06-3-4	a	0.30	0.26	3	0.33	72.1	1.47	742	4∅10@80	1053	12.46
15	P1	b	0.20	0.27	3	0	14.8	0.74	482	2∅8@100	503	2.51
16	P2	b	0.20	0.27	3	0.10	18.8	0.74	482	2∅8@100	503	2.51
17	P3	b	0.20	0.27	3	0.26	12.7	0.74	482	2∅8@100	503	2.51
18	P4	b	0.20	0.27	3	0	16.0	0.74	482	2∅8@50	503	5.02
19	P5	b	0.20	0.27	3	0.10	17.0	0.74	482	2∅8@50	503	5.02
20	Σ13	c	0.20	0.18	3	0.32	47.5	1.79	475	2∅8@75	495	3.32
21	CAAA	d	0.25	0.22	3	0.11	27.0	1.44	401	2∅13@50	350	7.42
22	CAAB	d	0.25	0.22	3	0.11	27.0	1.44	401	2∅13@75	350	3.71
23	SAAA	d	0.25	0.22	3	0.11	24.0	1.44	401	2∅13@50	350	7.42
24	SAAB	d	0.25	0.22	3	0.11	24.0	1.44	401	2∅13@75	350	3.71
25	No.2	a	0.20	0.18	2	0.30	57.1	3.13	959	4∅6@106	846	5.08
26	No.3	a	0.20	0.18	2	0.30	57.1	3.13	959	2∅6@53	846	5.08
27	No.4	a	0.20	0.18	2	0.30	57.1	3.13	959	4∅6@106	846	5.08
28	No.5	a	0.20	0.18	2	0.30	57.1	3.13	959	4∅6@212	846	2.54
29	No.6	a	0.20	0.18	2	0.30	57.1	3.13	959	4∅6@53	846	10.15
30	No.7	a	0.20	0.18	2	0.30	57.1	3.13	959	4∅6@36	846	15.23
31	No.8	a	0.20	0.18	2	0	57.1	3.13	959	4∅6@106	846	5.08
32	No.9	a	0.20	0.18	2	0.15	57.1	3.13	959	4∅6@106	846	5.08
33	No.10	a	0.20	0.18	2	0.60	57.1	3.13	959	4∅6@106	846	5.08
34	CUS	e	0.23	0.38	2	0.14	34.9	1.33	441	2∅6@89	414	0.62
35	2CUS	e	0.23	0.38	2	0.27	42.0	1.33	441	2∅6@89	414	0.62
36	CUW	e	0.41	0.20	4	0.16	34.9	1.76	441	4∅6@89	414	1.16
37	L02	f	0.30	0.26	3	0.10	26.3	1.28	378	2∅6@90	253	0.53
38	L04	f	0.30	0.26	3	0.10	27.6	1.28	378	2∅6@45	253	1.06

a: Watanabe *et al*. 1999, b: Papanikolaou *et al*. 1992, c: Tegos 1984, d: BRI 1978a, e: Umehara *et al*. 1982 and f: Minami *et al*. 1977

Table 2 Experimental and calculated shear resistances

Spec.	Label	V_{exp} (kN)	V_{ACI} (kN)	V_{CSA} (kN)	V_{AIJ} (kN)	V_{NZS} (kN)	$V_{EN98\text{-}1}$ (kN)	$V_{EN98\text{-}3}$ (kN)	$V_{EN98\ 1\text{-}3}$ (kN)	V_{MC2010} (kN)	$V_{strut\text{-}tie}$ (kN)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	HT6-4BL	560	298	350	582	293	588	295	588	538	578
2	HT6-4CL	656	409	652	760	425	597	589	589	863	616
3	HT6-2AH	532	349	252	443	306	492	197	492	541	485
4	HT6-2BH	557	385	350	582	348	738	295	710	669	550
5	HT6-4BH	623	385	350	582	348	709	295	709	679	704
6	HT6-4CH	632	496	652	760	481	1046	597	710	597	800
7	CA06-6-1	456	375	192	363	316	340	136	340	216	475
8	CA06-6-2	652	598	393	639	539	842	337	589	535	630
9	CA06-6-3	559	398	382	622	372	814	326	589	517	630
10	CA06-6-4	690	612	931	827	624	1200	589	589	875	720
11	CA06-3-1	521	484	192	363	370	340	136	340	373	550
12	CA06-3-2	692	707	393	639	593	842	337	708	703	810
13	CA06-3-3	573	507	382	622	426	814	326	708	705	652
14	CA06-3-4	732	721	931	827	624	1263	708	708	939	900
15	P1	99	113	141	104	160	186	91	91	122	55
16	P2	130	157	143	129	177	224	116	116	122	88
17	P3	128	155	140	89	186	197	118	118	162	125
18	P4	106	143	194	136	173	218	95	95	214	71
19	P5	141	190	207	144	184	245	112	112	227	103
20	Σ13	162	179	126	194	190	260	104	211	238	198
21	CAAA	181	244	327	343	290	388	198	198	359	157
22	CAAB	181	244	269	322	290	360	198	198	244	144
23	SAAA	181	230	290	334	258	354	190	190	319	153
24	SAAB	181	230	268	300	258	336	190	190	244	140
25	No.2	334	200	190	311	278	363	166	363	329	260
26	No.3	327	200	190	311	205	387	166	387	329	294
27	No.4	365	200	190	311	205	387	166	387	329	309
28	No.5	281	155	107	214	151	207	83	207	207	209
29	No.6	398	289	355	375	288	494	331	494	401	500
30	No.7	499	289	463	487	288	533	488	533	411	533
31	No.8	269	91	190	311	160	387	166	387	173	235
32	No.9	327	169	190	311	183	338	166	338	309	305
33	No.10	346	263	190	311	251	368	166	368	323	235
34	CUS	329	224	135	253	250	222	89	222	205	304
35	2CUS	405	278	139	276	301	222	89	222	250	360
36	CUW	267	221	135	237	243	231	92	231	92	250
37	L02	206	125	73	160	145	93	37	93	37	140
38	L04	222	168	111	215	188	186	75	186	75	219

No other limits for material strengths are used except for the concrete compressive strength in Eq. (1) where the minimum value of (f_c; 40 MPa) is applied.

It should be noted that in the calculations the actual steel strength of stirrups was taken into account except for the codes in which un upper steel strength limit is explicitly set for the calculation in shear, i.e., ACI, NZS and AIJ. If the values of stirrup yield strength had been reduced to comply with the steel grades prescribed by the codes for new structures (in order to ensure the desired ductility class) very conservative shear predictions would have resulted, given that more than half of the specimens examined had high yield strength of steel (f_{sy}>500 MPa).

4. Comparison of code and strut-and-tie model predictions to experimental results

Experimental results of test specimens subjected to antisymmetrical cyclic loading and axial force are used for comparison with the predictions of the codes and of the strut-and-tie model (Watanabe and Kabeyasawa 1999, Papanikolaou *et al.* 1992, Tegos 1984, Umehara and Jirsa 1982, B.R.I. No.2 1978, Minami and Wakabayashi 1977). All specimens had values L/h =2.0, 2.2, 3.0 and 4.0 (L, h being the unflanked length and the section height parallel to the direction of the shear force, respectively). The axial load ratios $v = N/bhf_c$ vary from zero to 0.60. In Table 1 the characteristics of the column specimens are shown (geometry, strength of materials and reinforcement). The majority of them possess high amount of shear reinforcement and high material strengths. Various specimens shown in Table 1 with the same value of $\rho_w f_{wy}$ (all the other parameters being equal) differ in the individual shear reinforcement parameters.

Only columns were included for which the calculated shear strength is lower than the shear force corresponding to code-calculated bending failure, although in the literature it is not always mentioned whether shear or flexural failure actually occurred first.

In Table 2, the maximum experimental shear force V_{exp} is indicated, as well as the code-calculated shear resistances. Shear force $V_{strut\text{-}tie}$ calculated by means of the proposed strut-and-tie model is also shown. In calculating the shear resistance according to the codes, overall reduction factors and partial safety factors were set equal to unity for the purpose of comparing the shear resistance with the corresponding experimental values (not for design purposes). Besides, strengths of materials measured in laboratory specimens are almost deterministic nominal values. All code limitations previously mentioned were respected.

A comparison between the analytical predictions against the product $\rho_w f_{wy}$ and the compressive concrete strength f_c are shown in Figs. 1 and 2, respectively (ρ_w = percentage of shear reinforcement, f_{wy} = yield strength of shear reinforcement). The actual strengths of materials, not the reduced ones specified as upper limits by the codes, are depicted on the figures.

A key essential observation made is that the upper limits these codes include (i.e., upper limits either in material strengths or in shear forces carried by concrete and stirrups as discussed previously) are not adequate in case of short columns.

The absence of appropriate upper limits is made evident for low values of compressive concrete strength f_c and stirrup yield strength f_{wy} and with considerable amount of shear reinforcement (specimens 15 to 24, Tables 1 and 2). For these specimens the codes considered, with the exception of EN1998-3, overestimate shear resistance. It is to be noted that for these columns shear resistances are calculated with the actual material strengths since no reduction in strength materials is required by the codes.

Conservative predictions in specimens with high material strengths (specimens 1 to 14, and 25 to 33, Tables 1 and 2) are attributed mainly to the reduction of the material strengths due to code limitations, and secondarily to the omission of the shear transferred through the diagonal strut mechanism which is important in case of low length-to-depth ratio elements.

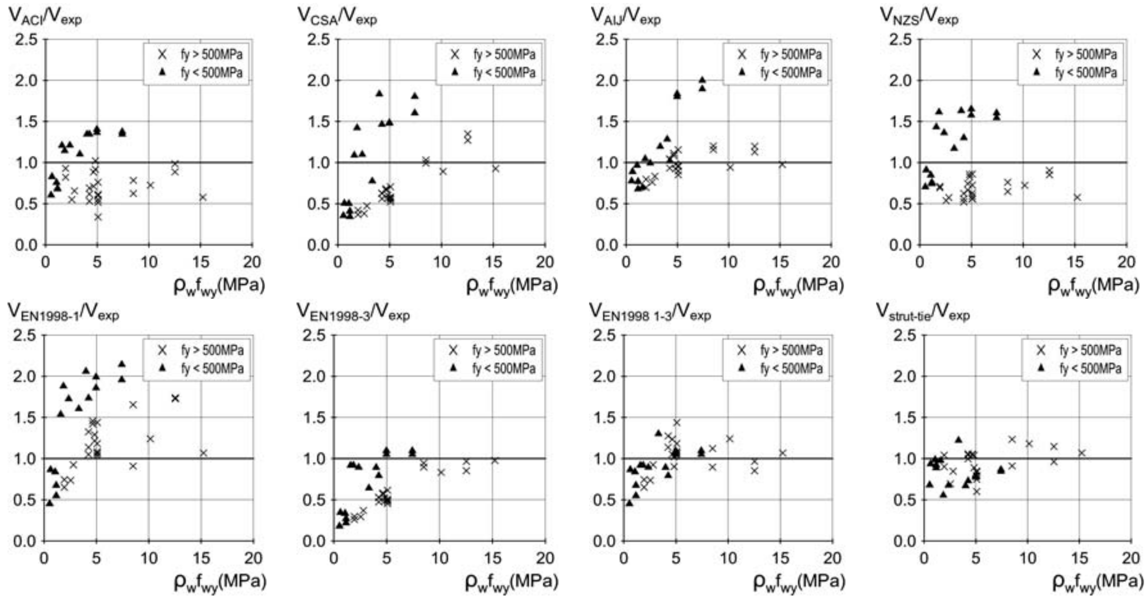

Fig. 1 Shear resistance ratios of the analytical predictions as a function of the product $\rho_w f_{wy}$ of the shear reinforcement

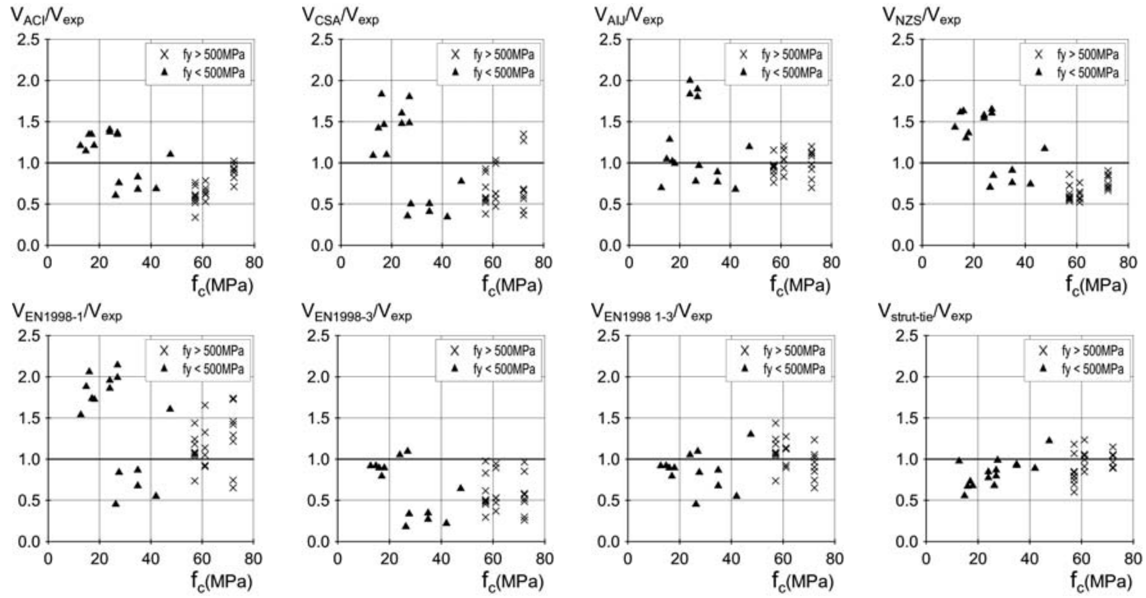

Fig. 2 Shear resistance ratios of the analytical predictions as a function of the concrete compressive strength f_c

Conservative predictions in case of specimens with low amount of shear reinforcement (specimens 34 to 38, Tables 1 and 2) are mainly due to the omission of the diagonal strut mechanism, since in this case the resistance calculated is mainly determined by the contribution of stirrups. Most conservative are the predictions of codes that assume null contribution of concrete,

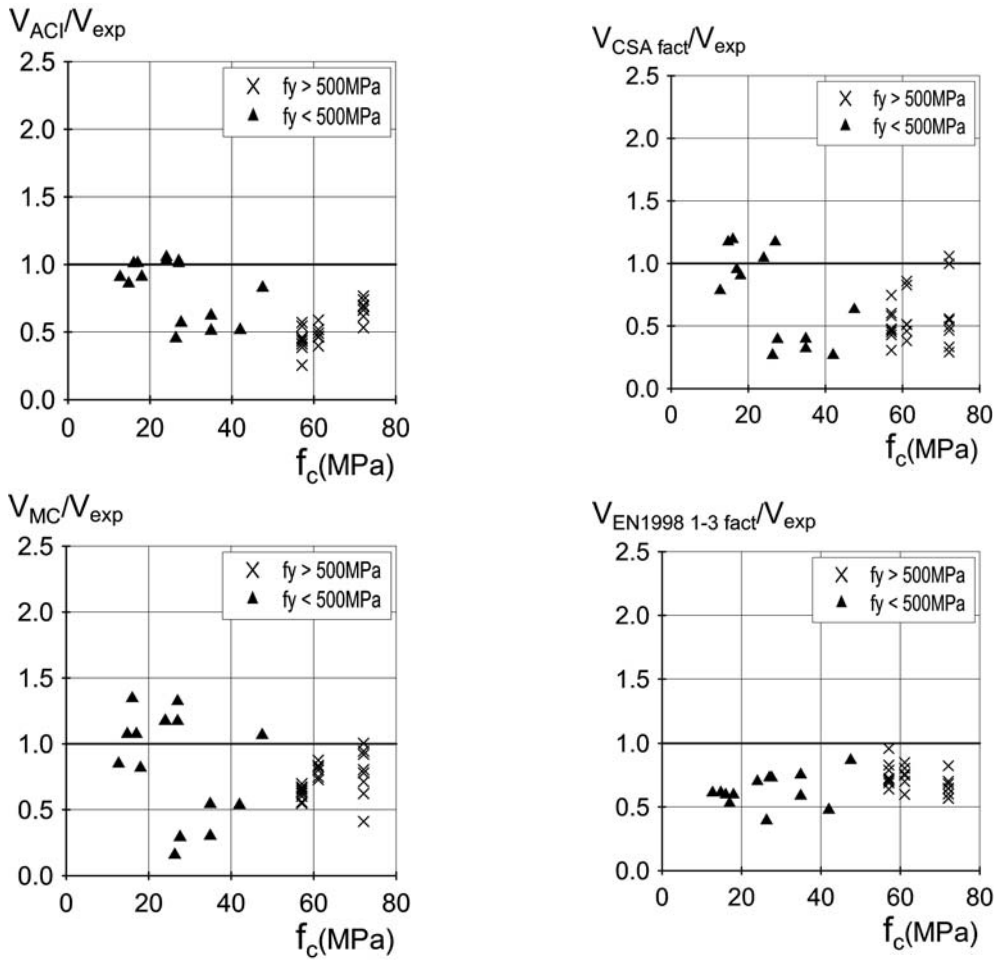

Fig. 3 Shear resistance ratios of factored analytical predictions as a function of the compressive concrete strength f_c

V_c=0, and/or strut angle θ=45 degrees, i.e., CSA, EN1998-3, and MC2010.

Among the codes considered the best approximation is offered by a combined application of EN1998-1 and EN1998-3, and by AIJ, i.e., by the two codes that include provisions for low length-to-depth ratio elements. The proposed strut-and-tie model (see Appendix B) estimates shear resistances with less scatter than the codes.

In the factored shear predictions for CSA and draft MC some unsafe predictions persist, and conservative predictions become still more conservative (Fig. 3). In EN1998 (parts 1 and 3) all factored predictions are safe. The same is practically valid for ACI, but with increased scatter and conservativeness in the predictions as compared to EN1998 (Fig. 3).

It should be pointed out that overestimation of shear resistance is undesirable, since it leads to unsafe predictions. Considerable underestimation of the shear resistance may as well be equally undesirable as it can lead to misleading assumptions in assessing the strength and stiffness of the element in question.

Apart from the general trends mentioned above, no uniform criteria may be formulated regarding the terms that determined the shear predictions of each code. Some individual examples and observations made are discussed in the following.

4.1 Codes ACI, CSA, NZS and draft MC

These codes showed more or less the same trends in predicting the shear resistance, although different calculation procedures and strength restrictions are adopted in each of them.

More particularly, ACI and NZS codes yield similar results, both making the assumption θ=45 degrees for the angle of the compression strut. Furthermore, similar are also the predictions of CSA (simplified method) and MC 2010 (Approximation Level II).

According to the ACI model, conservative predictions may occur either when the stirrup contribution is restricted depending on the concrete strength (e.g. spec. No.7, $\rho_w f_{wy}$=15.23, V/V_{exp}=0.58), or when concrete and stirrup contributions are fully taken into account in shear resistance (e.g. spec. HT6-4BL, $\rho_w f_{wy}$=4.25, V/V_{exp}=0.53). Non-conservative predictions may equally result in both the above-mentioned cases, e.g. spec. SAAB, $\rho_w f_{wy}$=3.71, V/V_{exp}=1.40 and spec. P5, $\rho_w f_{wy}$=5.01, V/V_{exp}=1.35, respectively.

In NZS the multiple limitations imposed on maximum material strengths and concrete stresses, in combination to the 45-degree assumption for the stirrup activation, often lead to conservative results (e.g. spec. HT6-4BL, V_s=128 kN, V_c=165 kN, V_{NZS}/V_{exp}=0.52). Non-conservative predictions result in specimens with low concrete strength (e.g. P1, CAAA, V_{NZS}/V_{exp}=1.60) the shear prediction of which is determined by the upper shear force limit for concrete.

In CSA, the strength limitations, combined with the small concrete participation for seismic actions and the 45-degree strut inclination, lead to conservative predictions (e.g. spec. HT6-2AH, $\rho_w f_{wy}$=2.83, V_s=87 kN, V_c=98 kN, V_{CSA}/V_{exp}=0.35). Non-conservative predictions result in specimens with low concrete strength (e.g. M1, CAAA) the shear resistance of which is determined by the upper shear force limit $V_{R,\max}$.

For draft MC2010 similar conclusions hold as for CSA, bur with less conservative predictions due to the increased stirrup activation owing to the varying strut angle θ.

4.2 AIJ provisions

This approach (see Appendix A) leads to a better overall approximation of the observed shear resistances compared to the codes, as the analytical concept used is more appropriate for low length-to-depth ratio elements. Occasional unconservative predictions (e.g. spec. SAAA, cotφ=1, V/V_{exp}=2.00) may be also attributed to inadequate limitation of the permissible shear force. However, the adoption in AIJ of limited stirrup contribution depending on the concrete strength seems to be more appropriate than the arbitrary upper limits concerning strength of materials imposed by the previously discussed codes.

Regarding the procedure adopted in AIJ it should be pointed out that it does not depict either the enhanced contribution of the diagonal strut to shear resistance with the increase of axial load or the beneficial effect of high shear reinforcement on concrete strength. Both of these factors are essential in describing the behavior of short columns. However, the omission of the axial force in the model does not seem to particularly influence shear strength predictions.

Furthermore, the arch mechanism involved in the model seldom contributes to shear strength in the specimens considered. It is activated only when the amount of stirrups is small compared to the concrete strength (e.g. in specimens CA06-6-1 and CUS the contribution of the arch to the shear strength is 30% and 50%, respectively). According to the equations, when the value cotθ of the compressive strut angle in the truss mechanism (among the three possible values given) equals the square root value, then β=1, which then leads to null arch contribution. This occurs regardless of the length-to-depth ratio. The model thus fails to consider the enhanced contribution of the arch (diagonal strut) mechanism for low length-to-depth ratios in case of high amounts of stirrups.

4.3 EN1998 part 1 provisions

EN1998-1 leads to particularly unsafe strength predictions of shear resistance for the columns considered (Figs. 1 and 2), although no contribution of concrete is taken into account (V_c=0).

This is attributed to the variable strut angle theta which leads to increased stirrup activation, combined to the inadequate upper shear resistance limit $V_{R,\max}$ of EN1992-1-1. It is recalled that the strut angle θ (21.8≤θ≤ 45 degrees) affects –in an inverse manner- both the shear resistance of stirrups, and the ultimate shear resistance $V_{R,\max}$.

Unsafe predictions result when the shear force is determined either by the upper limit in shear force $V_{R,\max}$ (e.g. P4, $V/V_{\exp}$=2.06), or by the shear resistance of stirrups V_s (e.g. spec.CAAA, $V/V_{\exp}$=2.14).

In case of small stirrup amounts EN1998-1 results in very conservative estimates (e.g. spec. HT6-2AH, V/V_{exp}=0.43 and spec. No.5, V/V_{exp}=0.52), due to the omission of the shear transferred through the diagonal strut mechanism.

(It is noted that if the yield steel stress 600 MPa (demanded for new structures) is set as upper limit, the predictions are more conservative).

4.4 EN1998 part 3 provisions

The procedure applied here consists in the use of an angle θ=45 degrees for calculating the contribution V_s of shear reinforcement, and also of an upper limit for elements with length-to-depth ratio L/h≤4 (Eq. (1)) which depends on the ratio L/h and is independent of the angle θ. The minimum shear resistance calculated by these provisions and by the provisions of EN1992-1-1 (i.e., calculated as previously discussed for EN1998 part 1, with variable angle θ) is used for assessing the element shear resistance. In all the columns considered in this work the shear predictions according to EN1998-3, $V_{EN98\text{-}3}$, are lower than the respective predictions of EN1998-1, $V_{EN98\text{-}1}$ (Table 2).

Application of EN 1998-3 leads to safe predictions $V_{EN\ 98\text{-}3}$ in practically all the specimens considered (Figs. 1 and 2). In general the predictions are conservative due to a) smaller activation of stirrups owing to the assumption of θ=45 degrees, *b*) omission of the shear transferred directly through the diagonal strut (arch mechanism). The most conservative predictions result in columns with low amounts of shear reinforcement (Fig. 1) the shear resistance of which is determined uniquely by stirrups (e.g. spec. CA06-3-1, $V/V_{\exp}$=0.26 and spec. 2CUS, $V/V_{\exp}$=0.22).

4.5 Complementary contribution of EN1998-3 to EN1998-1

To overcome the inadequacies resulting from the application of parts 1 and 3 of EN1998, EN1998-1 has been applied making use also of the upper limit for shear resistance $V_{R,\max}$ (Eq. (1)) found in EN1998-3. The predictions thus calculated are shown in Table 2 as $V_{EN98\ 1\text{-}3}$. This assumption results in less scatter in the predictions of the test values, but also to some unconservative predictions (Figs. 1 and 2). If the partial factors for material properties are used (γ_s=1.15 and γ_c=1.50 for steel and concrete, respectively) all predictions are safe, as seen in Fig. 3. Besides, in EN1998-3 the use of partial factors is mandatory when verification for shear resistance is performed.

Adoption of the variable angle theta ($21.8 \leq \theta \leq 45$ degrees) leads to greater activation of stirrups and depicts more accurately the actual behavior of small length-to-depth ratio elements for which θ<45 degrees, compared to EN1998-3 where θ=45 degrees.

On the other hand, application of $V_{R,\max}$ (Eq. (1)) of EN1998-3 seems to be a most appropriate upper limit for shear resistance of columns with $L/h \leq 4$ as it considers all the parameters involved and reduces the unconservative estimations of EN1998-1 in which $V_{R,\max}$ for linear elements is applied.

It is noted that the limit $V_{R,\max}$ of EN1998-3 (Eq. (1)) was in all cases significantly lower than $V_{R,\max}$ for linear elements (EN1992-1-1), with the exception of specimens No. 1 to No. 10 (Tables 1 and 2) with high percentage of total longitudinal reinforcement and ratio L/h=2, the shear resistance of which was determined by $V_{R,\max}$ of EN1992-1-1.

4.6 Proposed strut-and-tie model

The proposed strut-and-tie model (Moretti and Tassios 2006) considers simultaneously all the activated load transfer mechanisms. It has been calibrated and checked against a vast number of experimental data. No assumptions of yield conditions of the various resisting components are made, and no upper limits for material strengths and maximum shear values are set. The application of this model to the experimental data produces more uniform results than the codes examined, being a model appropriate for short columns. Unsafe predictions correspond, all but one, to values of concrete strength greater than approximately 60MPa. This may be due to the different influence of confinement on concrete in case of high concrete strength. If a safety factor equal to 1.2 is applied, all shear predictions are safe ($V_{strut\text{-}tie}/V_{\exp} \leq 1$).

Significant advantage of the model over the codes discussed here is that it enables the designer to consider the contribution of the various resistance mechanisms to shear capacity, and also the factors that can affect these mechanisms. Therefore it offers the possibility to alter the design so as to achieve the desired performance of the element (see Appendix B).

5. Conclusions

Based on this admittedly limited investigation the following conclusions may be derived.

Code predictions fall short from adequately anticipating the influence of the various parameters on the shear resistance of short columns - even in the case of JCI the model of which seems to be more appropriate for small length-to-depth ratio elements. This inadequacy of codes was expected to a certain extent because of the different shear transfer mechanisms inherent in short columns and in linear elements. Not only unsafe shear resistance predictions ($V/V_{\exp}$>1) but also very low predictions compared to the actual ones may in certain cases entail some risk for the safety of the structure. In the case of short columns (which bear high shear force due to their high stiffness) an underestimation of their resistance may lead to undesirable overall behavior of the structure, especially when displacement based analysis is performed for retrofit design.

The main observations from this work are summed up as follows.
1. Major shortcoming of the codes considered, with the exception of EN1998-3, is that the upper limits used for shear resistance are not adequate. For normal material strengths they lead to unsafe results. The empirical formula of EN1998-3 seems to be a most appropriate upper limit for shear resistance of columns with length-to-depth ratio less than 4.
2. Code limitations in strength of materials result in conservative estimations when applied to columns with high material strengths and considerable amount of shear reinforcement since the upper limits in material strengths are used instead of the actual (higher) strengths.
3. Absence of appropriate consideration of concrete contribution to shear resistance (arch action), as well as the 45 degrees strut angle adopted in certain codes, lead to conservative shear resistance predictions in case of short columns. This is more evident for low amounts of shear reinforcement when shear resistance is determined by stirrups, and especially when the concrete contribution is small, or ignored.
4. EN1998-3 may be used to safely predict the shear resistance of columns with length-to-depth ratio less than 4. To overcome the conservative predictions of this code it is suggested to use the shear force limit of EN1998-3 for elements with $L/h \leq 4$ in combination with EN1998-1 (variable strut angle θ). Thus the increased activation of stirrups for $\theta<45$ degrees, compared to $\theta=45$ degrees, compensates for, to some extent, the omission of the direct shear transfer through the mechanism of diagonal strut. This approach, if the factored material properties are used, seems to lead to safe predictions with reduced scatter compared to the experimental resistances.

Nonetheless, for the calculation of shear resistance in short columns adequate models ought to be used which will consider the following:

a) Most appropriate seem to be the strut-and-tie models. However, such models should be checked prior to their application against experimental data, owing to the variability of results in connection with the scheme and the cross-section of struts.

b) Two-fold influence of stirrup contribution to the ultimate shear force: (1) direct increase in shear capacity and (2) enhancement of the resistance of the concrete struts by means of the increased confinement offered.

c) Appropriate simulation of the contribution of the axial force. It should be noted that the axial load value affects shear capacity of short columns in two ways: (1) increased shear force transfer directly through the diagonal concrete strut which leads to increased shear resistance and (2) pre-activation of stirrups because of increased lateral expansion of concrete.

d) The amount of stirrups should ensure the required ductility of the column and not only contribute to shear resistance.

e) The maximum shear resistance related to the strength of concrete should be calculated by taking into account all the mechanisms activated, rather than as an upper limit (empirical or other) concerning the concrete stresses or the amount of reinforcement. In short columns uncoupling of flexural and shear interaction is not possible, as is the case in linear elements.

f) The proposed strut-and-tie model includes the above-mentioned aspects and its application to the experimental data produces more uniform results than the codes examined. The model may well serve for a more rational design of a new short column or for retrofit design of an existing short column, helping to select the most appropriate strengthening method so as to guarantee the desired level of ductility as well.

References

ACI 318M-08 (2008), "Building code requirements for structural concrete and commentary", *Am. Concrete Inst.,* Farmington Hills, Michigan, 473.

Architectural Institute of Japan (AIJ) (1994), "Structural design guidelines for reinforced concrete buildings".

Bentz, E.C. (2011), "Comparison of draft new model code shear provisions to experimental results", *fib Symposium*, Prague, Session 1-3: New Model Code, 135-138.

Bentz, E.C., Vecchio, F.J. and Collins, M.P. (2006), "The simplified MCFT for calculating the shear strength of reinforced concrete elements", *ACI Strut. J.*, **103**(4), 614-624.

Building Research Institute (B.R.I.) (1978a), "A list of experimental results on deformation ability of reinforced concrete columns under large deflection (No.2)", Japan, 115.

Building Research Institute (B.R.I.) (1978b), "A list of experimental results on deformation ability of reinforced concrete columns under large deflection (No.3)", Japan, 182.

Collins, M.P., Bentz, E.C., Sherwood, E.G. and Liping, X. (2008), "An adequate theory for the shear strength of reinforced concrete structures", *Mag. Concrete Res.,* **60**(9), 635-650.

CEB (1993), "CEB-FIP Model code 1990", Commite Euro-Internationale du Beton (CEB), *Thomas Thelford*, London, UK.

CSA Committee A23.3 (2004), "Design of concrete structures", *Canadian Standards Association,* Mississauga, Ontario, Canada, 214.

Eleftheriadou, A.K. and Karabinis, A.I. (2012), "Seismic vulnerability assessment of buildings based on damage data after a near field earthquake (7 September 1999 Athens-Greece)", *Earthq. Struct.,* **3**(2)*,*117-140.

European Committee for Standardization, CEN (2004), "Eurocode 2: Design of concrete structures. Part 1: General rules and rules for buildings", Brussels, EN 1992-1, 226.

European Committee for Standardization, CEN (2004), "Eurocode 8: Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings", Brussels, EN 1998-1.

European Committee for Standardization, CEN (2005), "Eurocode 8: Design of Structures for Earthquake Resistance. Part 3: Assessment and retrofitting of buildings", Brussels, EN 1998-3.

Galal, K., Arafa, A. and Ghobarah, A. (2005), "Retrofit of RC square short columns", *Eng. Strut*., **27**(5), 801-813.

Hong, S.G., Hong, N.K. and Chun, S.C. (2011), "Failure mechanism based strut-and-tie models", *fib Symposium*, Prague, Session 1-5: New Model Code.

Kim, W. and Jeong, J. (2011), "Decoupling of arch action in shear-critical reinforced concrete beams", *ACI Strut. J*., **108**(4), 395-404.

Kotsovos, M. (2007), "Concepts underlying reinforced concrete design: Time for reappraisal", *ACI Struct. J*., **104**(6), 675-684.

Kotsovos, M. (2008), "Ahthor's closure", *ACI Struct. J*., **105**(5), 646-647.

Kowalsky, M.J. and Priestley, N.M.J. (2000), "Improved analytical model for shear strength of circular reinforced concrete columns in seismic regions" *ACI Struct. J.*, **97**(3), 388-396.

Minami, K. and Wakabayashi, M. (1981), "Rational analysis of shear in reinforced concrete columns", Final report, *IABSE Colloquium on Advanced Mechanics of reinforced concrete*, Delft, 603-614.

Minami, K. and Wakabayashi, M. (1977), "Seismic resistance of reinforced concrete beam-and-column assemblages with emphasis on shear failure of column", *Proceedings of the 6th WCEE*, New Delhi, **VIII,** 3101-3106.

Model Code 1990 (1990), *First complete draft*, fib Bulletins 55, 56, Lausanne, Switzerland.

Moretti, M.L. (2008), "Discussion concerning No.104-S63", *ACI Struct. J.*, **105**(5), 645-646.

Moretti, M.L. and Tassios, T.P. (2007), "Behaviour of short columns subjected to cyclic shear displacements: Experimental results", *Eng. Struct.*, **29**(8), 2018-2029.

Moretti, M.L. and Tassios, T.P. (2006), "Behavior and ductility of reinforced concrete short columns using global truss model", *ACI Struct. J.*, **103**(3), 319-327.

NCHRP Report 549 (2005), "Simplified shear design of structural concrete members", *Transport. Res. Board Nat. Academies*, Washington, 54.
NZS-3101 (2006), "*New Zealand concrete structures standard*", The Colour Guy.
Papanikolaou, K., Tegos, J. and Penelis, G. (1992), "A comparative study on the seismic performance of conventionally and non-conventionally reinforced short members", *Eur. Earth. Eng*., **2**, 45-53.
Paulay, T. (1971), "Coupling beams of reinforced concrete shear walls", *J. Struct. Div.-ASCE*, **97**(3), 843-862.
Priestley, N.M.J., Verma, R. and Xiao, Y. (1994), "Seismic shear strength of reinforced concrete columns", *J. Struct. Div.-ASCE*, **120**(8), 2310-2329.
Shohara, R. and Kato, B. (1981), "Ultimate strength of reinforced concrete members under combined loading", Final report*, IABSE Colloquium on Advanced Mechanics of reinforced concrete*, Delft, 701-716.
Sullivan, T.J. (2010), "Capacity design considerations for RC frame-wall structures", *Earthq. Struct*., **1**(4)*,* 391-410.
Tassios, T.P., Moretti, M. and Bezas, A. (1996), "On the behavior and ductility of reinforced concrete coupling beams of shear walls", *ACI Struct. J*., **93**(6), 711-720.
Tegos, I. (1984), "Contribution to the study and improvement of earthquake resistant mechanical properties of low slenderness structural elements", *PhD thesis*, Aristotle University of Thessaloniki, 1985 (in Greek).
Tegos, I. and Penelis, G. (1988), "Seismic resistance of short columns and coupling beams reinforced with inclined bars", *ACI Struct. J*., **85**(10), 82-88.
Teran-Gilmore, A., Sanchez-Badillo, A. and Espinosa-Johnson, M. (2010), "Performance-based seismic design of reinforced concrete ductile buildings subjected to large energy demands", *Earthq. Struct.*, **1**(1)*,* 61-91.
Umehara, H. and Jirsa, J.O. (1982), "Shear strength and deterioration of short reinforced concrete columns under cyclic deformations", *PMFSEL Report No.82-3*, University of Austin, Texas, 256.
Vecchio, F.J. and Collins, M.P. (1986), "The modified compression field theory for reinforced concrete elements subjected to shear", *ACI Struct. J.*, **83**(2), 219-231.
Watanabe, F. and Kabeyasawa, T. (1999), "Shear strength of RC members with high strength concrete", ACI SP-176, High Strength concrete in seismic regions, 397-404.
Yamada, M. and Furui S. (1968), "Shear resistance and explosive cleavage failure of reinforced concrete members subjected to axial load", Final Report, *Proceedings of 8th Int. Congress IABSE*, New York, 1091-1102.

SA

Appendix A. AIJ Structural guidelines for R-C buildings (1994)

The calculation of shear strength is based on the superposition of truss and arch mechanism and results from the lower bound theorem. Failure of concrete is assumed to occur simultaneously with yielding of shear reinforcement. The difference in the angle of concrete struts between arch and truss mechanisms, as well as the effect of axial load are not taken into account. Shear strength V_u (A-method) is calculated as follows

$$V_u = b\, j_t\, \rho_w f_{wy} \cot\theta \;+ \tan\varphi\,(1-\beta) bh\, \nu f_c\, /2$$

where b=width of section, h=overall depth of section, j_t =distance between top and bottom bars, L=clear span of member, f_c = concrete compressive strength, f_{wy}=yield strength of shear reinforcement, ρ_w=shear reinforcement ratio, and ν = effectiveness factor for the compressive strength of concrete (=0.7-f_c/200, f_c in MPa). Moreover, tan $\varphi = \sqrt{(L/D)^2+1}$ -L/D, φ = angle of the arch mechanism, β=[(1+cot$^2\theta$) $\rho_w f_{wy}$]/(νf_c), θ =angle of the compressive strut in the truss

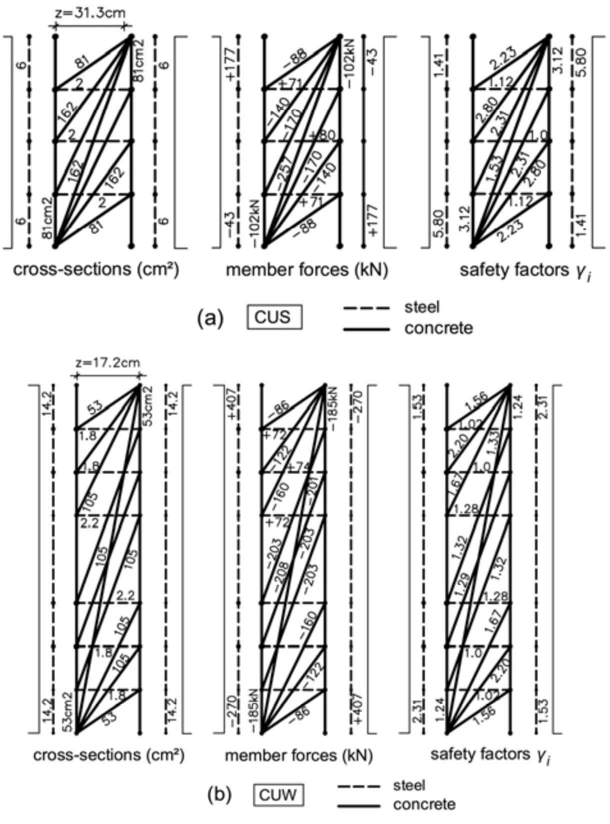

Fig. B1 Strut-and-tie model, forces and safety factors γ_i (=resistance/member force) for the shear resistance calculated by the model; (a) column CUS, L/h=2.2, $V_{strut\text{-}tie}$=305 kN, (b) column CUW, L/h =4, $V_{strut\text{-}tie}$=255 kN (Umehara and Jirsa 1982)

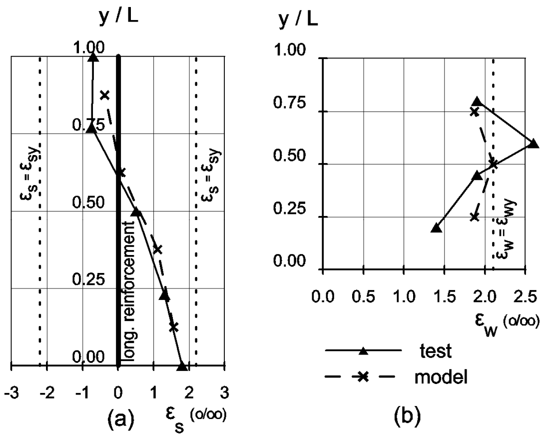

Fig. B2 Comparison of steel strains along the height L between values experimentally measured and those predicted by the strut-and-tie model for specimen CUS with L/h=2.2, for shear force $V=V_{exp}$=329 kN; (a) longitudinal reinforcement, (b) stirrups

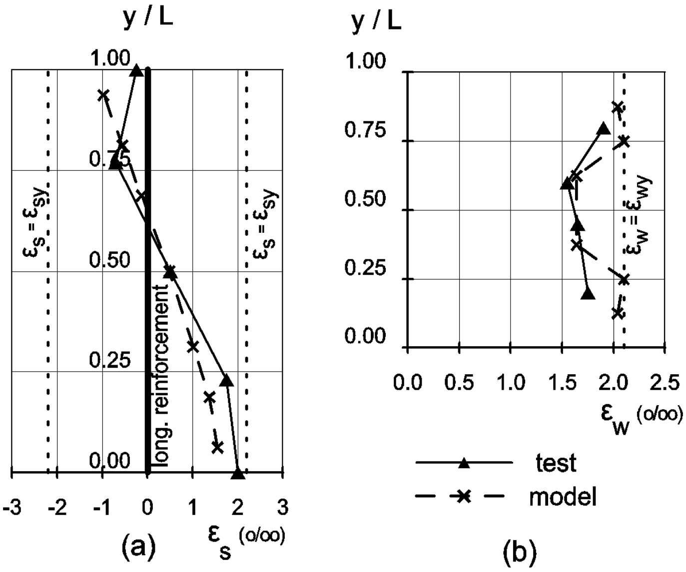

Fig. B3 Comparison of steel strains along the height L between values experimentally measured and those predicted by the strut-and-tie model for specimen CUW with L/h=4, for shear force $V=V_{exp}$=267 kN; (a) longitudinal reinforcement, (b) stirrups

mechanism, $\cot\theta$ being the minimum value of [2.0, $j_t/(D \tan \varphi)$, $\sqrt{\left(\nu f_c / \rho_w f_{wy}\right) - 1}$] and $1.0 \le \cot\theta \le 2.0$.

Yield strength of steel for stirrups is limited to f_{wy}=25 f_c if f_{wy}> (400 MPa, or 25 f_c), and the contribution of shear reinforcement is limited to $\rho_w f_{wy} = \nu f_c/2$ if $\rho_w f_{wy} > \nu f_c/2$.

Appendix B. Strut-and-tie model

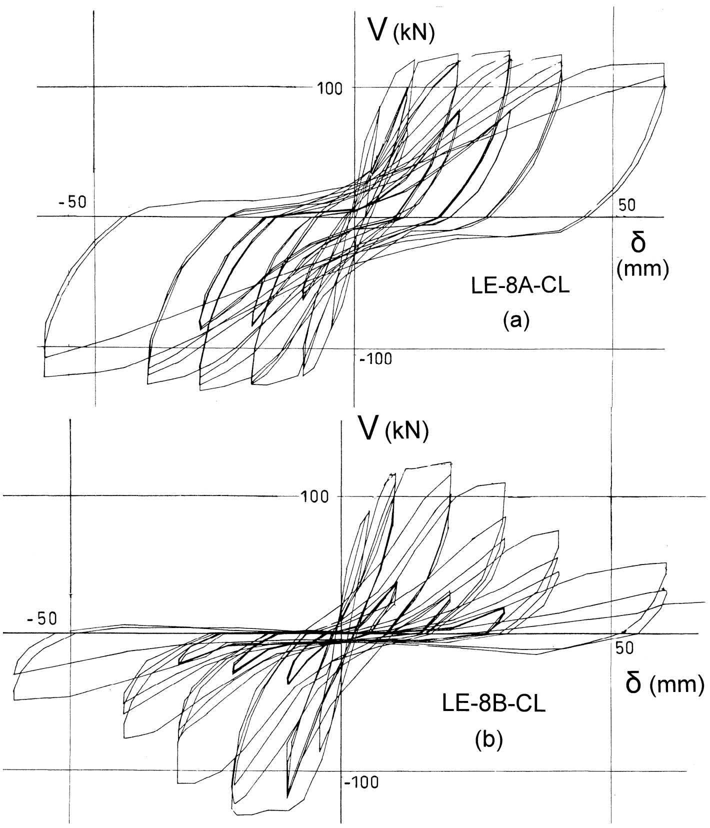

Fig. B4 Shear force-horizontal displacement relationship (BRI, 1978b); (a) column LE-8A-CL, (b) column LE-8B-CL

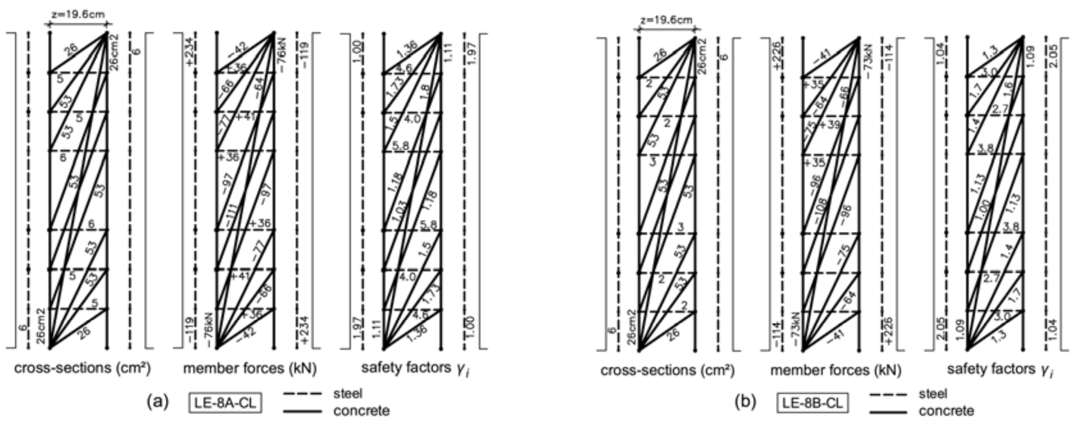

Fig. B5 Strut-and-tie model, forces and safety factors γ_i (=resistance/member force) for the shear resistance calculated by the model; (a) column LE-8A-CL, L/h=4, V_{exp}=130 kN, $V_{strut\text{-}tie}$=124 kN, (b) column LE-8B-CL, L/h=4, V_{exp}=127 kN, $V_{strut\text{-}tie}$=122 kN

The strut-and-tie model applied in this work is presented in detail elsewhere (Moretti and Tassios 2006). In the present work the advantageous influence of the confinement on the compressive strength of concrete is taken into account according to the following equations of CEB-FIP Model Code 1990.

$$f_c{}' = (1.000 + 2.5\ \alpha_n \alpha_s \omega_w) f_c \qquad \text{for } \sigma_2 < 0.05 f_c$$

$$\text{or } f_c{}' = (1.125 + 1.25\ \alpha_n \alpha_s \omega_w) f_c \qquad \text{for } \sigma_2 > 0.05 f_c$$

where $f_c{}'$=compressive concrete strength taking into account the confinement, α_n=1-$(\Sigma b_i^2)/(6b_o^2)$, α_s=$(1\text{-}0.5s/b_o)(1\text{-}0.5s/d_o)$, ω_w=(volume of stirrups/volume of corresponding concrete)×(f_{wy}/f_c), b_i=distance between bars at corners of ties, b_o and d_o=dimensions of confined cross section, σ_2 (=σ_3) = $0.5\alpha_n\alpha_s\omega_w$ = effective lateral strength due to confinement.

The potential of the model for the design of short columns is shown in the examples that follow.

a) The strut-and-tie model leads to a good estimate of the strains in the reinforcement as shown (Fig. B1) when applied to specimens 34 (CUS, α_s=1.1) and 36 (CUW, α_s=2) of Table 1. Comparison between calculated (model) and measured (test) strains in both cases is satisfactory (Figs. B2 and B3). Particularly in the case of CUS the distribution of predicted strains along the longitudinal reinforcement has a parabolic shape, as expected in short elements after diagonal cracking (Pauley 1971, Tassios *et al.* 1996).

b) The strut-and-tie model is applied to two specimens (BRI 1978b): LE-8A-CL (V_{exp}=130 kN, ties 2∅9@40, f_{wy}=343.1 MPa, $\rho_w f_{wy}$=4.36) and LE-8B-CL (V_{exp}=127 kN, ties 2∅6@37, f_{wy}=454.6 MPa, $\rho_w f_{wy}$=2.77). The characteristics these specimens had in common were: cross section b=h=0.25 m, length L=1 m, longitudinal reinforcement 2×3∅16, f_{sy}=395 MPa, f_c=24 MPa, axial load N=164 kN, while they differed only in the $\rho_w f_{wy}$ value. In these specimens flexural failure is expected according to code calculations, for this reason they were not included in Tables 1 and 2. Specimen LE-8A-CL behaved in a more ductile manner than specimen LE-8B-CL with less response degradation and higher amount of energy dissipation (Fig. B4). The model depicts in fact this behavior (Fig. B5): LE-8B-CL ($f_c{}'$=30.2 MPa) is expected to fail in a brittle manner along an inclined concrete strut (safety factor of concrete strut γ=1, where γ =member resistance/member force in the strut-and-tie model). LE-8A-CL, on the other hand, fails due to yielding of the longitudinal reinforcement ($f_c{}'$=31.9 MPa because of the increased confinement).