ACI VERSUS CEB DESIGN OF R.C. COLUMNS

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SUMMARY

For r.c. columns governed by compression, ACI code allows in general higher concrete stresses than the European code, but requires more confining reinforcement. The behaviour of columns designed according to both codes was observed experimentally, on 5 + 5 specimens. They were subjected to cyclic compressive strains of increasing amplitude. Results are suitable for a direct comparison, and provide new insights as to the descending branch of the stress-strain diagram.

INTRODUCTION

Fig. 1 represents the M-P ultimate domain, according to ACI 318/77 code, of a r.c. column. A similar domain holds also for the CEB code apart from three differences: 1) If tension governs, the ϕ factor is 0.87 instead of 0.9 as for ACI code; 2) if compression governs, the ϕ factor is 0.63 instead of 0.75 as for ACI code -see section 9.1-; and 3) the maximum compression is limited to $0.5 f_{ck} A_g$, independent on reinforcing. The reference to characteristic values, f_{ck} and f_{yk}, instead of the expected values of f_c' and f_y is responsible for a further difference. It appears that the major disagreement regards the second and third items. Consequently for a column with a limited eccentricity, CEB requires wider cross section aerea than ACI code. Conversely higher concrete confinement, i.e., higher lateral reinforcing is prescribed by ACI code. In the present paper the different behaviour of columns designed according to both philosophies is discussed.

The reason for establishing an upper bound of P is twofold: 1) For columns subjected to severe bending deformation under earthquake conditions, the lower P is the higher the ductility shown in the M-rotation diagram, and, if $P > P_b$, the energy necessary to reach collapse in bending; and, 2) For columns loaded with a limited eccentricity in earthquake conditions, see Fig. 2 for instance, limiting P under the loading combinations including earthquake provides safety margins in the elastic range. It is in fact generally recognized that when compression governs, the margins offered by the ductility are limited. Margins are therefore to be searched for within the elastic range, precisely, by prescribing an upper limit of P. In the present paper the comparison will be drawn on the basis of this second item only. The comparison for the first item in fact is somewhat arbitrary in that the CEB code applies an amplification factor w to the seismic bending moment of columns, which have no correspondence in the US design nor in any other earthquake regulation. See for details the mentioned code, section 4.3.1.2.

To this purpose the structure of Fig. 2 has been designed according to ACI 318, Appendix A and, successively, according to CEB code, under the effects of dead load, and identical live load and earthquake conditions. Because CEB does not include seismic zoning, the maximum free field ground acceleration, required by CEB design, has been set equal to 0.30g and this was assumed equivalent to the US zone 4 and the subsequent derivation of lateral inertia

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forces established by UBC. In this example, and for the European design, the value 0.30 is amplified by a factor 2.5, corresponding to the "plateau" of the normalized response spectrum. Then it is divided by 4, the ductility factor to be assigned to the "dual system" of Fig. 2 when designed according to the higher ductility requirement of CEB.

The lateral forces which follow are, in general, identically distributed in the US(UBC) and the European approach but for a normalization factor. However, when the load combinations, with the pertaining load factors, are applied, the contribution of the earthquake load to the design axial load is quite similar in both codes. For instance at the base column of Fig. 2 it is 107 t for CEB and 112 t for ACI. Althougth small, it is essential to notice that the same difference is shown by two further quantities: 1) the load factor to be assigned to dead load in the main loading combination (1 for CEB, 0.75x1.4 for ACI), and 2) the design strength of steel in tension (0.87 f_{yk} for CEB, and 0.9 f_y for ACI). On this basis it can be assessed that the two codes can be made consistent each to the other, both as to the input load and to the strength of material, as far as steel tension is concerned.

However, a similar consistency cannot be established when the design is governed by concrete in compression. In the above example, for instance, the design of the base column is quite different, see Fig. 3 and Table II and III, explained in the next section. In particular the confinement of concrete and the minimum cross section aerea required by either code is different.

To exploit the performance of structures so designed, a research was undertaken on 10 r.c. columns, of the two types of confining reinforcement, acted on by cyclic axial strain of increasing amplitude, [9].

Since Sinha [11], a number of researches are reported in literature on concrete acted on by cyclic compressive stress or strain, but only a few concern reinforced concrete elements [2,7,8], and in practice only three deal with r.c. elements of dimensions ratios representing columns, [5,10,12]. Unfortunately none of them is directly appliable to the present subject, because the strains are there read across the most damaged region (10 - 20 cm) and the behaviour of the entire column is not directly modelable from this. The present research on the other hand tries to represent the behaviour of the entire column, and, moreover, it provides identical test conditions for the two kinds of columns, so it offers stress-strain diagrams suitable for a fair comparison.

The main results of this comparison are the following.

1) The two series of columns show till $\epsilon = \epsilon_{max}$ a similar behaviour, see Table IV.

2) As to ACI design, the average value of the quantity P_{max} found experimentally is 1.14 P_{th} , while the maximum design load is 0.8x0.75 P_{th} Therefore P_{max} is nearly 1.9 times the maximum design load. As to CEB design this ratio is 2.04, (reference to the cross section of Fig. 3 is made). Assuming a linear elastic behaviour till $P = P_{max}$, and a response spectrum amplification around the natural frequencies of the building of 2.5, the value P_{max} at the base column is reached for a ground acceleration of approximately 0.28 g, for the columns of minimum dimension

designed according to both codes, as represented in Fig. 3. It can therefore be assessed that during an earthquake of maximum ground acceleration up to ·.28 g, the two design philosophies deserve a similar safety margin, as far as the other structural elements too remain in the linear elastic range.

3) Above ϵ_{max} the relevant behaviour diverges. See Figs 4, and 5 showing higher ductility margins as to ACI design.

DESIGN OF THE BASE COLUMN

In order to compare the two codes, the design of the minimal cross section according to both codes is shown in fig. 3. The ratio of the design compression P_u to A_c f_{ck} governs the CEB design. By this code one may take benefit of column widths larger than the minimal: the required ρ_s in fact decreases, see Table II, so some congestion can be softened. In general the quantity A_{sh} does not increase with the increasing of the dimension of the cross section.

By working with ACI code on the other hand, there is no benefit in column widths larger than the minimum required: in fact the lower limit $\rho_s = 0.12\ f_c'/f_y$ is independent on b, see Table II. By this code the governing quantity is the ratio of the vertical reinforcement, see Table III. A reasonable upper bound has been set up to 4%, in the present paper.

By comparing the two tables, it results that the minimum column width according to CEB is 0.53 m, but one may take benefit of larger dimensions to reduce ρ_s , while the minimum column width according to ACI is 0.4 but there is no practical advantage in $b > b_{min}$ to reduce ρ_s .

The New Zealand code is generally in between the two previous ones but biased toward ACI position.

Further differences between CEB and ACI codes as to ρ_s are:

1) The amount of lateral reinforcing prescribed for the critical regions holds for the entire column according to ACI, but for large columns, (with the cover dimension choosen, for b>.60 m). On the other hand, CEB allows a reduction to 1/3 of ρ_s in non-critical regions, for the choosen example.

2) For a given ρ_s both codes consider a reduced efficiency of rectangular hoops with reference to spirals. However according to ACI some reduction in the required steel area through the section is allowed when supplementary crossties are used, see section A 6.53 and 4 of the code.

Table I Source data. Strength are in kg/cm², D = 144 t, L = 36 t.

	CEB	ACI
Design axial load	D + L' + E 287 t	0.75 (1.4D + 1.7L + 1.7.1.1 E) 309 t
Earth. axial load	107 t	80 t
Concrete Reinforcing	f_{ck} = 207 f_{yk} = 4050	f_c' = 222.5 f_y = 4200

Table II Ratio of volume of spiral reinforcement to total volume of core, ρ_s ,in function of the columns width, within the critical regions of the colums. Concrete cover 0.035 m. The value b = 0.53 is the lower bound of b to respect the limit 0.5 $f_{ck}A_g$ prescribed by CEB.

$P_u/A_c f_{ck}$	0.3	0.4	0.5	0.63	
b (m)	0.75	0.65	0.60	0.53	
ρ_s (%)	0.333	0.381	0.429	0.44	CEB
	0.525	0.587	0.646	0.72	SANZ
	0.643	0.643	0.643	0.643	ACI

Table III ρ_s in function of the ratio of the total reinforcement to the gross area of the section, according to ACI, along the entire column. CEB or SANZ do not allow the values of b here shown.

ρ	0.02	0.03	0.04
b_{min}	0.45	0.42	0.40
ρ_s (%)	0.796	0.87	0.94

EXPERIMENTAL CONDITIONS

Columns were cast vertically. After ageing, both ends were capped with hydrostone, and the surfaces, -concrete and reinforcement- were smoothed by a diamond drill to ensure good contact between the loading heads and the specimen. Two further ties were placed near the ends. This resulted to be enough to ensure that the failures would occur in the instrumented region of the column.

Deformed wires were used both for vertical and for horizontal steel, assembled in two different configurations. All the specimens were cast from the same concrete mix. However three different times of casting were necessary and this gave origin to some differences in the reference f_c'. This quantity was determined from three samples for each specimen. During ageing, columns and concrete samples were placed in the same temperature and humidity controlled room.

Local axial strains and hoop strains were monitored through electrical strain gauges welded over reinforcing. Vertical elongations of the entire column were measured througth linear variable differential transducers, directly applied to the loading heads.Further information will be given in [9].

The loading apparatus was manufactured by MTS. It is a fairly rigid loading apparatus, able to provide cyclic strain of a given amplitude.

As to excitation history, only compressive strains were considered. In fact, due to the inavoidable presence of construction joints along the column, the likelihood of existence of meaningful tension stresses is hardly debatable. Besides, in the case of pier foundations, at the pier-soil boundary, no tension stresses can be transmitted.

Cyclic strains of constantly increasing amplitude were applied, each cycle starting from the state $\epsilon = 0$, $P = 0$. This kind of excitation provides the wider range of comparison with the previous researches reports, [2], being

somehow representative of the seismic excitation.

The aim of the present research is quite limited, so it does not cover the analysis of the effect of the strain gradient, the one of the concrete strength or composition, the effect of the load eccentricity and that of the strain rate. In this regards the test condition were: maximum strain increase from cycle to cycle, $\Delta\epsilon = 2\ 10^{-4}$; main frequency = 0.05 cps; f' = 307 - 362 kg/cm^2; e = 0.. This last conditions was assured by the presence of mechanical hinges at the heads of the loading apparatus.

MAIN EXPERIMENTAL RESULTS

Figs. 4 and 5 are two typical load-deformation diagrams obtained experimentally .

In the range $P < P_{max}$ it can be noted: 1) the strain ϵ_{max} corresponding to P_{max} is approximately the same in the two series of tests. The more confined ACI specimen show a sligthly better performance in this respect; 2) the quantity P_{max} is sensibly affected by ρ_s , see Table IV. Notice however that the chosen values of ρ_s follow from the requirements of both codes within "non-critical regions" of columns of the present dimensions, and the resulting ACI ratio is a limit value. See Table III and IV for a comparison.

As to the remaining portion of the cycles, a qualitatively different behaviour is shown by the two series of tests, see again Figs. 4 and 5. It can be noted that the CEB column suffer a more rapid stiffness degradation after cycling, the strength being practically null for $\epsilon > 1.3\epsilon_{max}$.

Table IV Main results over 5 + 5 columns. The variance is shown as ratio to the average value.

	ACI (ρ_s = 1.18%)		CEB (ρ_s =0.24%)	
	average value	variance	average value	variance (%)
P_{max}	170.60 t	7.8 %	130 t	17.5
ϵ_{max}	3.69 ‰	10.3 %	3.21 ‰	19.0
P_{max}/P_{th}	1.14	12.0 %	0.84	18.0

LIST OF SYMBOLS (in agreement with Appendix C of ACI 318)

A_g = gross area of section;
A_s = total area of longitudinal reinforcement;
A_{sh} = area of transverse hoop bar. The area refers to one leg, according to ACI;
A_c = aerea of rectangular core of column measured out-to-out of hoop;
f'_c = specified compressive strength of concrete - standard cylinder test;
f_{ck} = characteristic value of the compressive strength (0.86 0.95)f'_c , depending on the dispersion of the strengths around the average value;
f_y = specified stress of reinforcement;
f_{yk} = characteristic value of the yield stress of reinforcement;
P = axial load in column;
P_u = design axial compression;
P_b = axial load strength at balanced strain condition;
P_{th} = $[0.85f'_c\ (1 - \rho\ ) + f_y\ \rho\ ]\ A_g$;

P_{max} = the maximum value of the axial load observed experimentally, in the $P - \epsilon$ diagram;
ϵ = axial strain, average value over the column length;
w = an amplification factor applied by CEB code to bending moments of columns;
ρ = ratio of total reinforcement;
ϕ = strength reduction factor, see ref. [1] chapter 9.

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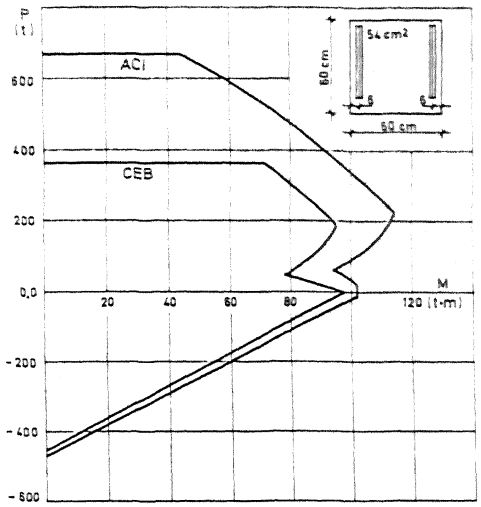

Fig.1 Ultimate domain according to ACI and CEB codes. f'_c = 222.5 kg/cm^2; f_y = 42000 kg/cm^2.

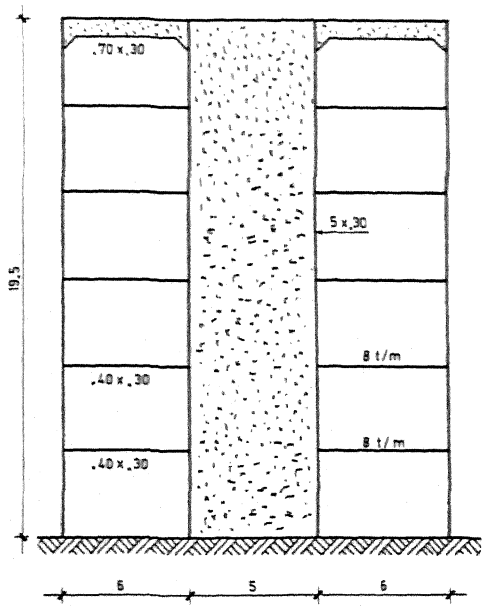

Fig.2 Dual system under examination. Measures in meters. The lower portion of the column is governed by simple compression in earthquake conditions.

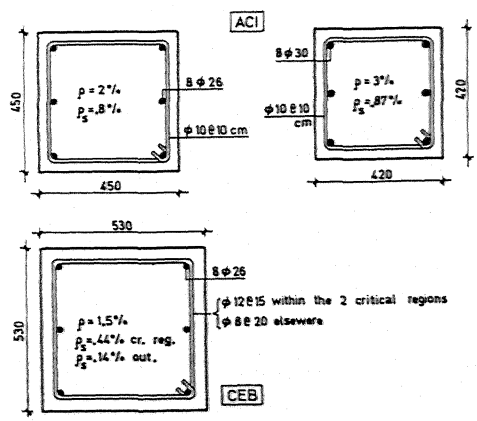

Fig.3 Measures in mm. Cover width 30 mm. The critical regions extend along 150 mm. A single hoop is designed for the sake of comparison.

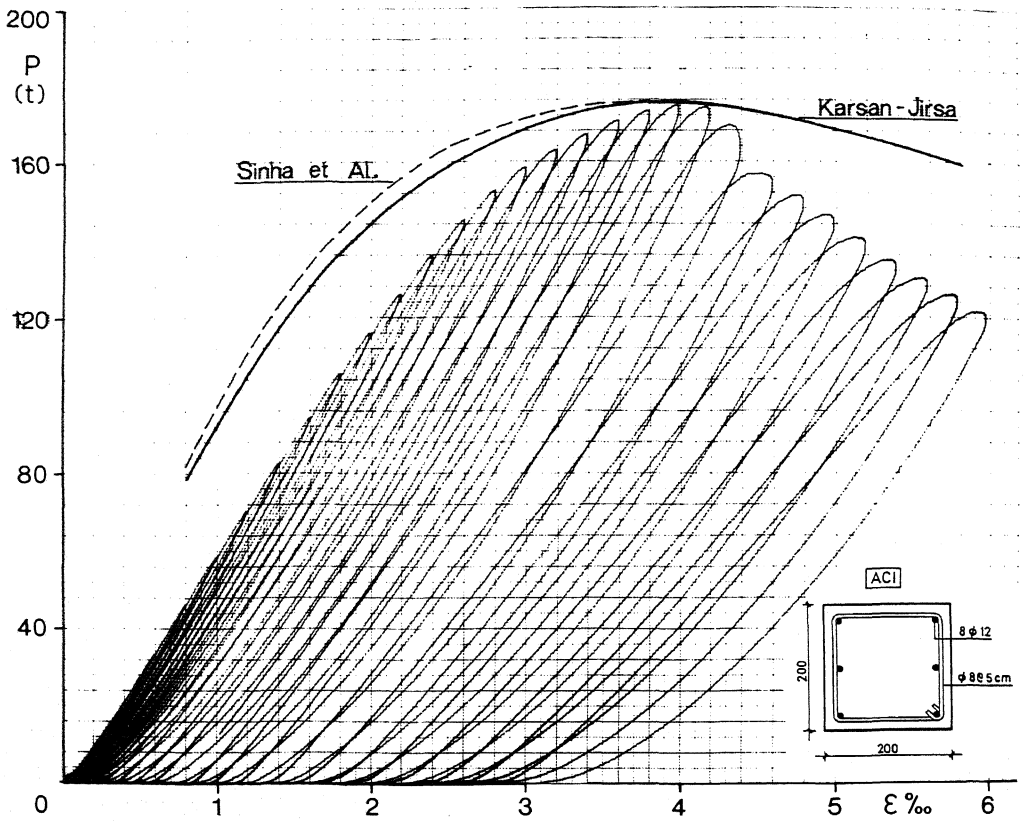

Fig.4 Experimental results of an ACI specimen, 1 m long. Two envelopes curves according to [5] and [11] are represented for comparison.

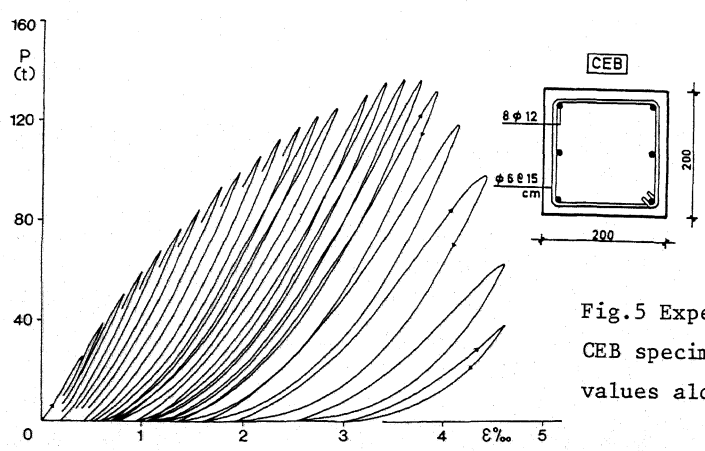

Fig.5 Experimental results of a CEB specimen. Strains are average values along the entire column.