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ADVANCES IN EARTHQUAKE-RESISTANT DESIGN OF CONCRETE STRUCTURES

T.P. Tassios

Department of Civil Engineering, National Technical University of Athens,
5, Iroon Polytechneiou str. 157 73 Athens, Greece

ABSTRACT

The philosophy of aseismic design of R.C. structures is briefly reminded; the significance of conceptual design is emphasised. Subsequently, some basic engineering models are reproduced, among those useful in analytical prediction of R.C. behaviour under post yield cyclic conditions. Emphasis is also given to shear resistance verifications especially for low shear-ratio elements, such as coupling beams and short columns. Several ductility related subjects are subsequently discussed, including definition of ductility factors, confinement requirements and relevant reliability considerations.

KEYWORDS

Conceptual design, confinement reliability, coupling beams, ductility verification, shear resistance, short columns.

1.- INTRODUCTION

1.1.-As a fundamental advance in earthquake-resistant design of concrete structures, the energy dissipation design-concept should be recalled once again: "The design should provide an adequate energy dissipation capacity to the structure, without substantial reduction of its overall resistance against horizontal and vertical loading. Adequate resistance of all structural elements shall be provided under the seismic combination of actions, whereas non-linear deformations in preselected critical regions should allow for the overall ductility assumed in the analysis". As it is known, this design-concept is implemented by means of design-criteria which will not be repeated here.

1.2.- The entire design-process includes three distinguished (but strongly articulated) stages, namely (i) conceptual design, (ii) analysis (calculation of action-effects of each structural element), and (iii) dimensioning-verification.
Of this rather extended process, this State-of-the-Art Report will not elaborate on seismic Analysis, which would deserve another very important Report; three-dimensional non linear dynamic analysis would in fact be a decisive tool, practising Engineers are expecting to be able to use as soon as it proves to be adequately reliable. It is however worth repeating here that the disproportionately large uncertainties related to the seismic input data (amplitude, frequency content, duration), coupled with the more or less aleatoric modifications of the stiffness matrix during real strong quakes, may drastically reduce the level of confidence to such analyses, be it the most sophisticated (Paulay, et. al., 1992)(*). If this is so, other components of the entire design process should be developed and applied in a way that the consequences of such uncertainties may be alleviated as much as possible: And those other components (conceptual design and dimensioning-verification), are precisely the topics of this Report.

2.- CONCEPTUAL DESIGN

Every analysis has to be carried out on a preconceived structural scheme; several "decisions" (temporary though) have to be taken a priori, prior to any analytical procedure. Thus, the following data should be preselected based on experience, oversimplified calculations and managerial thinking: General geometry, quality of materials, cross-sectional dimensions (of both concrete and steel).
In seismic design, more particularly, I will consider as an important advance the more or less formal recognition of the importance of this first design step.

2.1.-Selection of Materials

Materials are no more selected on the basis of their strength characteristics alone; ductility considerations may occasionally dictate higher concrete strengths (see §5.1), whereas several properties of steel (see i.a. Eurocode 8, Part 1-3, 1994) other than its nominal yield strength $f_{sy,nom}$ are seriously taken into account (Fig.1):

(*) Incidentally, among the factual advances in the aseismic design of concrete structures, the significance of the publication of this book cannot be overemphasised.

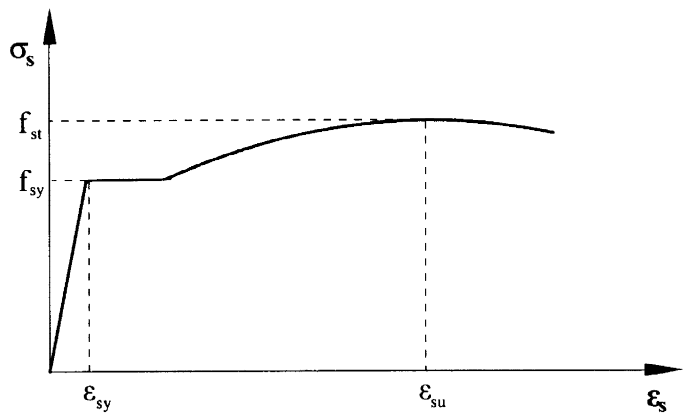


Fig. 1: Basic characteristics of steel

- Higher uniform elongation ε_{su} (at maximum load) and higher "tensile strength to yield strength ratio", f_{st}/f_{sy}, are sought to ensure adequate plastic hinge lengths, higher rotational capacities (see §3.3) and higher cross-sectional resistance after the spalling of concrete-cover.
-Nonetheless, the ratios f_{st}/f_{sy} and $f_{sy,act}/f_{sy,nom}$ should not be higher than certain specified values, in order to avoid uneconomical and doubtful control of the desired inelastic mechanism (see §2.2).

Recent developments in rebars' manufacturing processes (high-strength weldable steel) accompanied with a somehow reduced ductility, have encouraged specific research work on the subject (see i.a. Fardis M.N., 1995).

2.2.-Control of the Inelastic Mechanism

Although several analytical verification on this issue are to be made at a later design stage, it is important to remind here that at the stage of conceptual design appropriate care should be taken to enhance the "capacity design" criterion. As it is known, the following aspects of this philosophy are normally considered.

a) **Structural Regularity** should be sought, so that plastic hinges will not concentrate in only few areas (e.g. as in the case of a "soft storey mechanism"). To this end, code provisions regarding regularity of stiffnesses of non-cracked elements may not directly serve the purpose. Plastic hinge formation being a resistance-governed phenomenon, uniformity of "safety-margins" distribution seems to be a more direct criterion of regularity. In this respect, safety-margin is defined as the difference between expected action-effect and the corresponding available resistance of the region (cross-section) considered (e.g. M_R-M_S in case of prevailing flexural mode of failure). Useful examples of such distributions of safety-margins in elevation are reproduced here below.

Decanini et al. (1986) reported a reliable relationship between observed damage degree of buildings and their "regularity index" defined as

$$a_r = \min i_k / i_m \qquad (1)$$

where "i_k" denotes the ratio between seismic force acting on the k-th storey and the sum of shear resistance of all vertical elements of the same storey (infills included).

and i_m is the mean value of these ratios of all storeys

Fig. 2 illustrates schematically some relevant cases, together with corresponding consequences on the value of the overall behaviour factor ("reduction factor") "q'", as compared to normally recommended values "q".

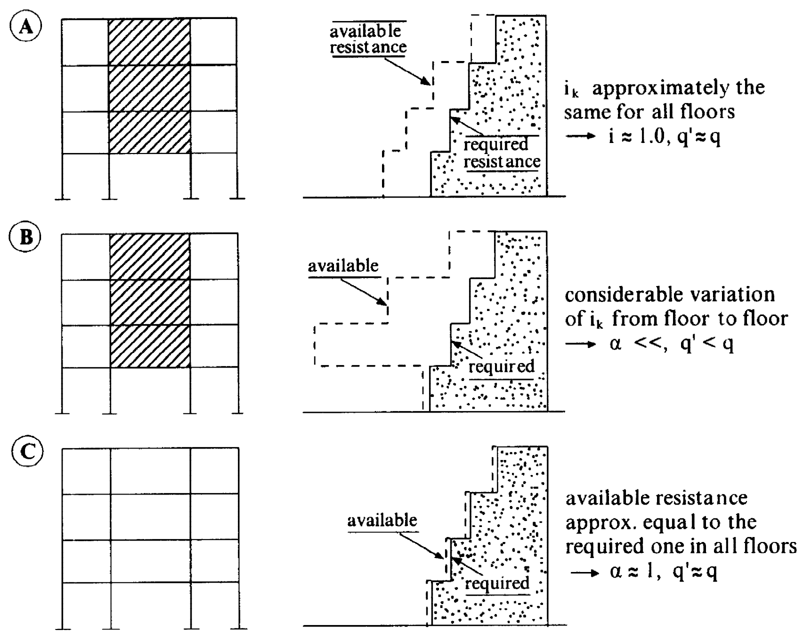
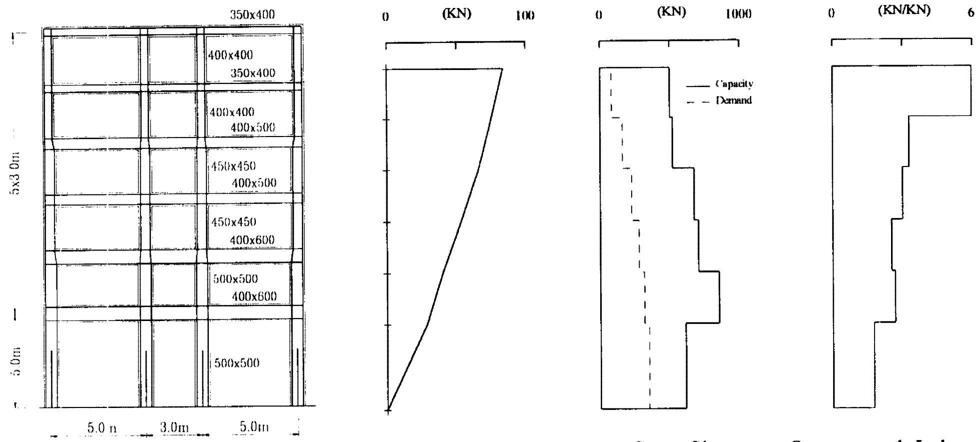


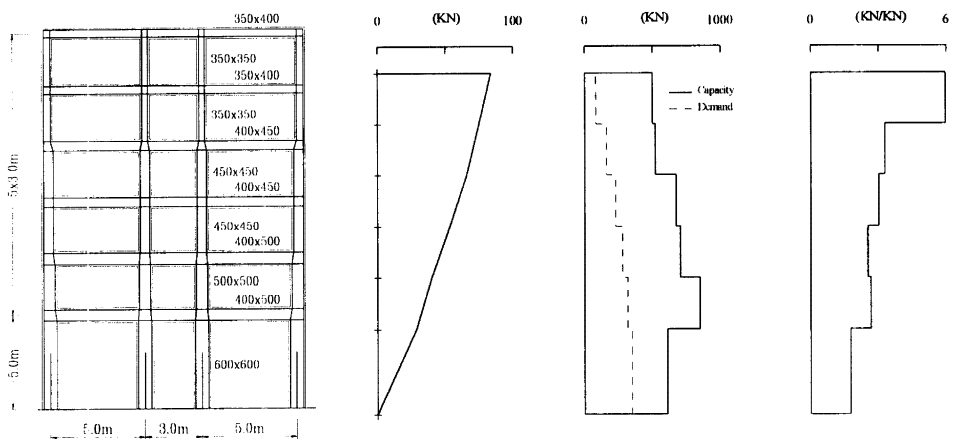
Fig. 2: Vertical irregularity of R.C. frames and its effects on the behaviour factor; a qualitative illustration (Decanini et al. 1986)

Such deficient overstrength distributions may also appear in the case of bare R.C. frames if a constant cross –sectional dimension or rebar diameters are adopted throughout the height of the buildings for the sake of simplicity in construction. This was for instance the case of Fig. 3a; shear-overstrengths were irregularly distributed. Subsequently, by means of an appropriate modification of column dimensions (see Fig. 3b), a considerably more uniform distribution of shear-overstrength was achieved. Nevertheless, it has to be recognised that such a (hidden) cause of irregularity is bound to the uncertainties of the shear demand as imposed by the particularities of actual quakes.



Geometry and Reinforcement Design Lateral Force Story Shear Overstrength Index

(a). Frame designed with initial cross-section dimensions



Geometry and Reinforcement Design Lateral Force Story Shear Overstrength Index

(b). Frame designed with modified cross-section dimensions

Fig. 3: If only, ∅32 mm rebars were to be used throughout the height of this building, slight modification of column dimensions may improve the regularity of overstrength distribution in elevation

b) **The strong column/weak beam** approach is almost everywhere adopted nowadays. However, a rather probabilistic approach in applying this capacity design criterion is followed in Europe, as opposed to the genuine development of the criterion in New Zealand and USA. The idea is to recognise that if the actual moments M_{bS} (acting on beam-ends under the seismic combination of loads) are considerably small as compared to the available resisting moments M_{bR} (which may be mainly dictated by vertical loading), it is reasonable to expect that under seismic conditions the moments $M_{bS,CD}$ which will probably act on these beam-ends may not reach the corresponding values M_{bR}. This may lead to a "milder" application of the capacity design criterion, using a "moment reversal factor", δ, as defined in EC8, P 1.3 §2.8.1.1(4). In Fig. 4 the interdependency of the relevant factors is roughly illustrated. With the notation of Fig. 4, the values of design moments of the column can be expressed as follows: Uncompromised application of the capacity design criterion

$$M_{cS,CD} = \alpha_{CD}.M_{cS} \triangleright q.M_{cS} \tag{2}$$

"Milder" version of the same criterion

$$M'_{cS,CD} = \alpha'_{cD}.M_{cD} = [1+(\alpha_{cD} - 1).\delta].M_{cS} \triangleright q.M_{cS} \tag{3}$$

It appears that, in the case of small ä-values, a substantial economy may be made.

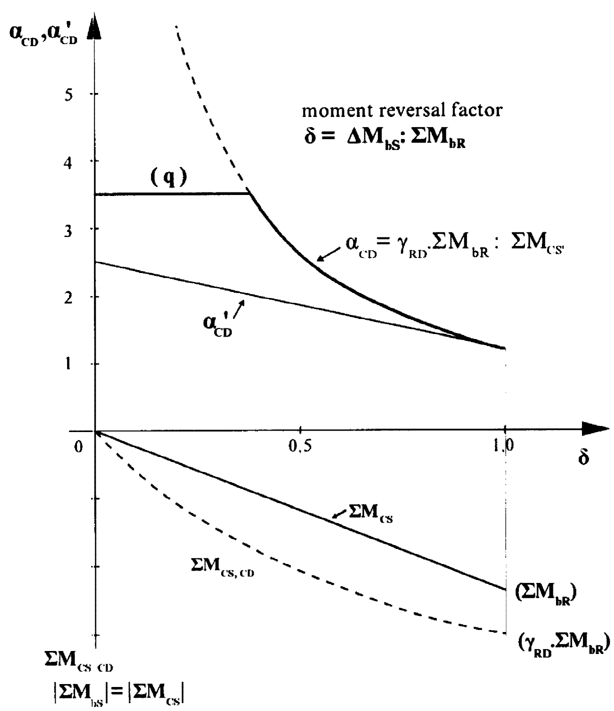


Fig. 4:
A "milder" application of the capacity design in columns is allowed in EC8 if the moments acting on beam-ends under the seismic combibation are considerably lower than the corresponding resisting moments

More generally, in the opinion of this writer, the philosophy of any capacity design criterion needs to be further clarified by means of appropriate probabilistic simulations. In fact, we do not avail of a quantitative insight on the causes and effects producing a complete exhaustion of every (nominal) safety margin at all critical regions simultaneously. And, since only mathematical testing is conceivable on this matter, it is advisable to carry out parametric studies with pragmatic randomness simulation of both the seismic action and the structural behaviour of critical regions.

3.- BASIC MODELLING

With the advances in computerised analysis, we have also witnessed a tendency to underestimate the realities of the structural behaviour of critical regions. It was rare to see bond-slip or buckling of corner-bars under cyclic loading to be taken into account in softwares, even the most promising among them. Progress in design however is hard to be achieved without a full recognition of physical phenomena. The fundamental importance of modelling of the mechanisms of "local" structural behaviour was also recognised in the case of non-seismic environments. The recent CEB-FIP Model Code '90, does include in one of its first chapters a set of such engineering models.
The importance of these models appears to be more practical in the case of seismic actions, under which a dramatic degradation of constitutive laws takes place, leading to drastic modifications both of stiffness and force-response characteristics. That is why it was thought that in this S.O.A. report some consideration should be given to the matter.

3.1.-Confined Concrete

A practical model we introduced both in CEB-FIP Model Code 90 and the background documents of Eurocode No 8, is illustrated in Fig. 5. The average confining stress laterally acting on concrete, may be approximated by

$$\frac{\sigma_2}{f_c} = \frac{\sigma_3}{f_c} = \frac{1}{2}\alpha\omega_w \qquad (4)$$

where ω_w defines the volumetric mechanical ratio of confining steel
and $\alpha = \alpha_n.\alpha_s$, is a reduction factor expressing the effective concrete area in cross-section and in elevation (see Fig. 5).

The resulting compressive strength and deformability increase are approximately described by the following expressions.

$$f_C^* = f_c(1{,}000+2{,}50.\alpha\omega_w) \quad \text{if } \sigma_2/f_c < 0{,}05 \qquad (5a)$$

$$f_C^* = f_c(1{,}125+1{,}25.\ \alpha\omega_w) \quad \text{if } \sigma_2/f_c > 0{,}05 \qquad (5b)$$

$$\varepsilon^*_{cl} = \varepsilon_{cl}.(f^*_c / f_c)^2 \quad (6)$$

$$\varepsilon^*_{c,85} = \varepsilon_{c,85} + 0{,}1.\alpha\omega_w \quad (7)$$

In spite of existing experimental evidence supporting this modelling (see i.a. Pilakoutas et al., 1994), further research is needed to assess its applicability under large number of full deformation reversals.

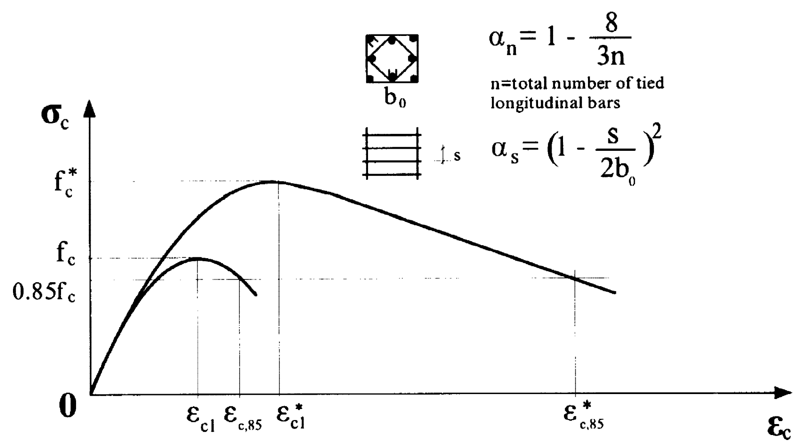


Fig. 5: A practical model for confined concrete [see Equs. (5a) to (7)]

3.2.-Bond under Cyclic Loading

Among other more precise models (Eligehausen et al., 1983) the bond-slip prediction illustrated in Fig. 6 (Tassios, 1979 and 1983) may be useful for practical applications. Among the analytical formulations proposed that of Yankelevsky et al. (1992) should be mentioned here.

3.3.-Plastic Rotational Capacity

The significance of prevailing flexural failure-modes cannot be overemphasised. The corresponding favourable hysteretic behaviour is due to relatively large rotational capacities of critical regions. Moreover, in the displacement based design philosophy (see i.a. Kowalsky et al., 1994, Calvi et al., 1995) ϑ_{pl} or total ϑ_u-values are directly used in design. Yet, technical literature is not very rich in investigating this basic property of R.C. elements; as a substitute, the issue is normally addressed by means of curvature ductility considerations or by means of valuable empirical relationships (see i.a. Fardis, 1995).

That is why it would be worth reminding here the parameters affecting plastic rotational capacity (be it within an oversimplified model).

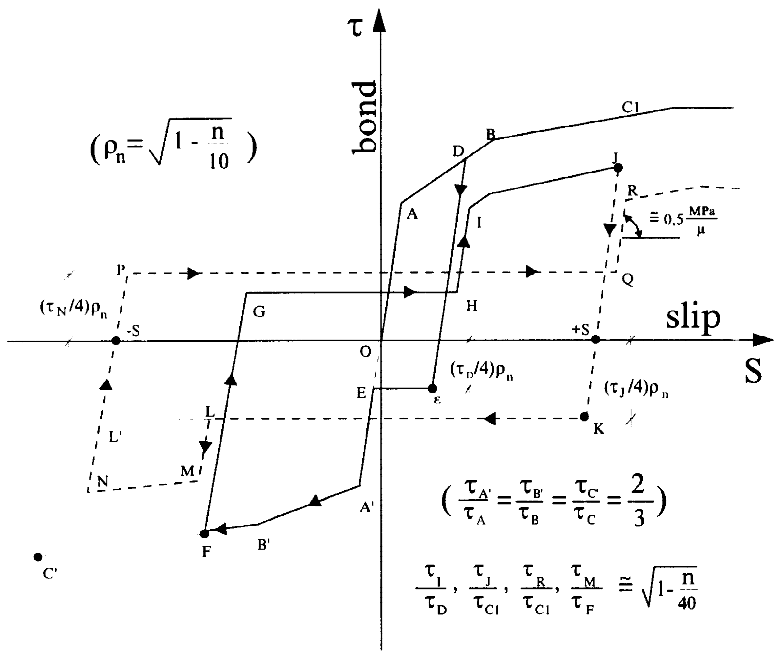


Fig. 6: A practical constitutive low for local bond vs. local slip of deformed bars under cyclic conditions ("n" full cycles)

Let us consider the case of a beam end (Fig. 7); the distance a_0 of the contraflexure point to the column face is eventually increased to $\beta.a_0$ because of the shear action. From simple geometric relationships on Fig. 7, it may be found that

$$\beta = 1 + \frac{1}{2}\frac{z}{\tan\varphi}\frac{V_0}{M_{max}} \cong 1 + \frac{0,4}{\tan\varphi}\frac{V_0 d}{M_{max}} \tag{8}$$

where

V_0, the shear force acting at the contraflexure point

φ, the angle of the inclined compressive force in the web ($30°<\varphi<45°$) due to shear

d, the structural depth of the beam cross-section

a) Now, if **concrete** is critical, a rough estimation of the plastic rotation due to the "plastification" of concrete alone can be made if we assume that in the region where $\varepsilon_s>\varepsilon_{sy}$ (Fig. 7), concrete strain ε_c^* is uniformly distributed. Thus

$$\vartheta_{pl} \sim \frac{a_{pl}}{d}\frac{\varepsilon_c^*}{\xi_{cu}} = \frac{\beta a_0}{d}(1-\frac{f_{sy}}{f_{st}})\frac{\varepsilon_c^*}{\xi_{su}} \qquad (9)$$

(Since in the remaining length a_0 post-yield contributions to ϑ_{pl} were not taken into account here, the elastic rotation is not subtracted from the value given by Equ. 9).

In this expression,

ξ_{cu}, is the normalised depth of neutral axis at ultimate load

$\varepsilon_c^* = \varepsilon_{c,85}^*$ predicted by Equ. 7

$\triangleleft$ 7.10^{-3} nominal maximum ε_{cu}-value for a zero-response of the extreme compressive fibre.

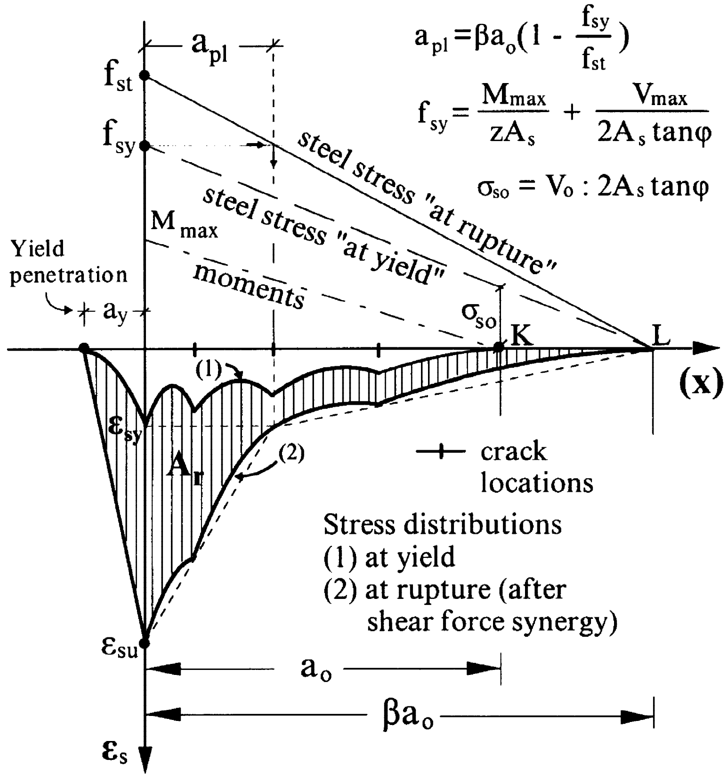


Fig. 7:Post-yield steel-strains contributing to the total "plastic" rotational capacity of the critical region, in case steel is critical ($\varepsilon_s = \varepsilon_{su}$, Fig. 1); a bilinear σ_s-ε_s diagramme was taken into account

b) If, on the other hand, **steel** is critical (i.e. if its uniform plastic elongation ε_{su}, Fig. 1, is reached), Fig. 7 offers a qualitative representation of the post-yield steel-strains contributing to the plastic rotational capacity of the critical region; a "yield penetration" length a_y beyond the column face does also contribute. The area A_r included between the strain distributions at rupture and at yield may roughly be estimated

$$A_r \approx k\left\{\frac{1}{2}(\varepsilon_{su}-\varepsilon_{sy})a_{pl}+\frac{1}{2}\varepsilon_{sy}a_{pl}+\frac{1}{2}\varepsilon_{sy}(\beta-1)a_0+\frac{1}{8}\varepsilon_{sy}a_0\right\}\eta+\frac{1}{2}(\varepsilon_{su}-\varepsilon_{sy})a_y \quad (10)$$

where

k, a factor reflecting bond and scale effects

a_y, the yield penetration (roughly taken equal to five rebar diameters "d_b").

η = a factor reflecting the shape of the diagram of bending moments; if linear, η =1

Now, the concept of the "plastic hinge" length l_{pl} may be used, defined as

$$l_{pl} = A_r : \varepsilon_{su} \quad (11)$$

Finally, the plastic rotational capacity in the case that steel is critical, may be approximated as

$$\vartheta_{pl} = l_{pl}\left(\frac{1}{r}\right)_u = A_r : d(1-\xi_{su}) \quad (12)$$

where
$$\left(\frac{1}{r}\right)_u = \frac{\varepsilon_{su}}{1-\xi_{su}}\frac{1}{d} \quad (13)$$

Or, by virtue of Equ. 12 and Fig. 7,

$$\vartheta_{pl} = \{\frac{k\eta}{2}\beta\varepsilon_{su}\frac{a_0}{d}[1+(1-\frac{0{,}75}{\beta})\frac{\varepsilon_{sy}}{\varepsilon_{su}}-\frac{f_{sy}}{f_{st}}]+2{,}5\frac{d_b}{d}(\varepsilon_{su}-\varepsilon_{sy})\}:(1-\xi_{su}) \quad (14)$$

Both Equs 9 and 14, demonstrate the significance of the following parameters enhancing the plastic rotational capacity:

- large ε_{cu}-values (confinement)
- large distance α_0 between the contraflexure point and the critical cross-section
- small values of the normalised neutral axis depth ξ_{cu}, i.e. small normalised axial force values, or enlarged ends (e.g. double flanged sections or barrelled walls)
- large ε_{su} -values (ductile steel)
- large f_{st}/f_{sy} ratios contributing to longer plastic hinges
- smaller shear-ratio values (M/Vd) may be favourable, under the condition however that they are sufficiently large to avoid "short column" effects.

Moreover, **scale effects** appear to be rather pronounced, not only because ϑ_{pl} directly depends on the a_o/d ratio, but also because ε_{cu}-values and bond behaviour are scale sensitive. Experimental and analytical evidence related to these conclusions, may be found in previous work; among the best collections of relevant papers, the CEB Bulletin 218, (1993) and Longlei Li (1995) are commendable. However, the state of the art is not mature enough, particularly if cyclic loading conditions are considered.

4.- SHEAR RESISTANCE VERIFICATION

Only a limited selection of developments in this field is presented in what follows; other issues may be equally important.

4.1.- Low Shear Ratio

To quote Fardis (1994), "partly due to its complexity and partly because it has not received enough attention from the research community, the problem of modelling the load-deformation behaviour and strength of low shear-span-ratio elements (in cyclic shear and normal force) does not have yet a satisfactory answer".

a) More particularly, regarding **coupling beams**, what really matters is not their resistance under monotonic loading, such resistance being acceptably defined by modern Codes. What really matters is their force-response degradation ΔV_n after a certain number of post-elastic excursions at an imposed ductility level "μ". In Fig. 8 experimental results (Tassios et al., 1996) are reproduced on such degradations for different shear-ratio values a_s = M:Vd and several patterns of steel reinforcement. The detrimental effect of small a_s-values is apparent, even in the

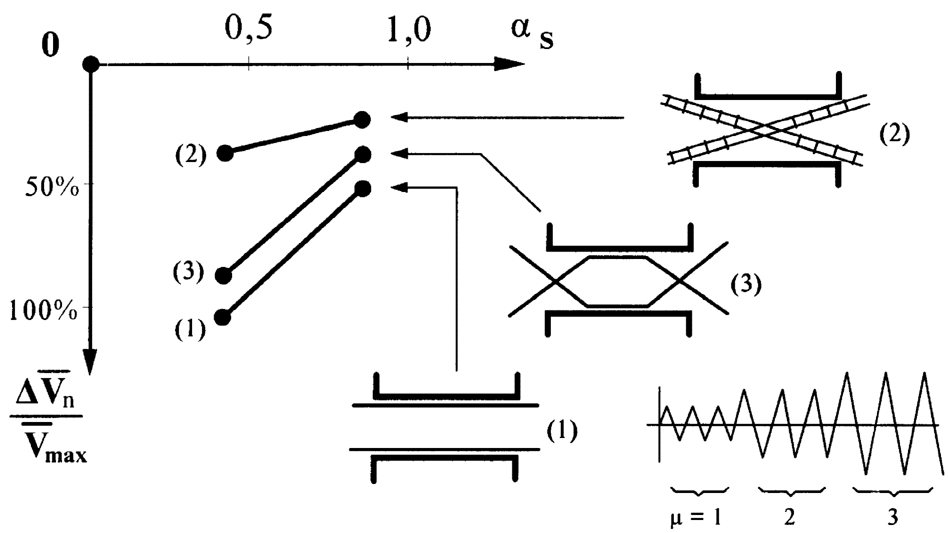


Fig. 8: Force-response degradation (average values) of coupling beams, with various shear ratios and different patterns of reinforcement, at imposed ductility μ=3 and after a total number of cycles n=9

case of bidiagonal "hidden columns" (specimens "2"). It should be noted that the calculated initial "static" resistance of all specimens (for each a_s-value) were equal. Since their respective "seismic" resistance (at, say, 20% degradation) were much different, the need for a pragmatic analytical prediction of such a "seismic" resistance (appropriate notation $V_{R,\mu,0.20}$) becomes apparent. Thus, except for the steel pattern of bidiagonal "hidden columns", the calculation of the shear resistance of coupling beams cannot be made by means of models provided in the Codes under static conditions. One of the possible solutions could be the application of an "understrength" factor ($\gamma' = V_{R,code} : V_{R,act}$) empirically estimated, which roughly speaking may take the following form

$$\gamma' \approx k_1(1-k_2.\alpha_s).\mu_d \lhd 1, \qquad (\text{for } 0,5 < a_s < 1,5) \tag{15}$$

in order to counterbalance resistance degradation. However, most preferable rational analytic models are now expected.

b) Similarly, in the case of **short columns**, Code-predicted shear resistance (web compression models) are valid only under monotonic conditions. From Moretti (1996), Fig. 9(i) illustrates this fact. Small shear ratio columns $a_s < 2$, correctly designed versus monotonic loading may exhibit disproportionately large force-response degradations when submitted to cyclic post yield displacements. This becomes worse in case of large percentages of longitudinal steel (Fig. 9, ii), and higher normalised axial force values ("ν"). Here again, despite extensive research, a generally accepted model for shear strength prediction under cyclic post-yield conditions does not seem to be available.

The following suggestions may be made for design.

- Bidiagonal arrangement of reinforcement may be helpful in this case too.
- Whenever it is possible, adequate shear overstrength should be provided so that the ratio $M_{V_R} : M_R$ is kept sufficiently higher than unity (e.g. $\gamma_{Rd} = 1,2$):

$$M_{V_R} = a_s V_R d \tag{16a}$$

$$M_{V_R} : M_R \lhd \gamma_{Rd} \tag{16b}$$

$$(V_R)_{requ.} = \frac{\gamma_{Rd}}{a_s} \frac{(M_R)_{av}}{d} \qquad (\text{for } a_s \leq 1,5) \tag{16c}$$

where a_s, shear ratio

$(V_R)_{requ.}$, required value of shear resistance

$(M_R)_{av}$, available flexural resistance

- In any event, for an acceptable force-response degradation (say 20%), the "feasible" ductility level should be estimated and used in design, instead of overoptimistic values of behaviour factors "q" (reduction factors "R") conventionally used. As an example, column "D1" of Fig. 9,i could hardly be designed with a displacement ductility factor equal to 2.

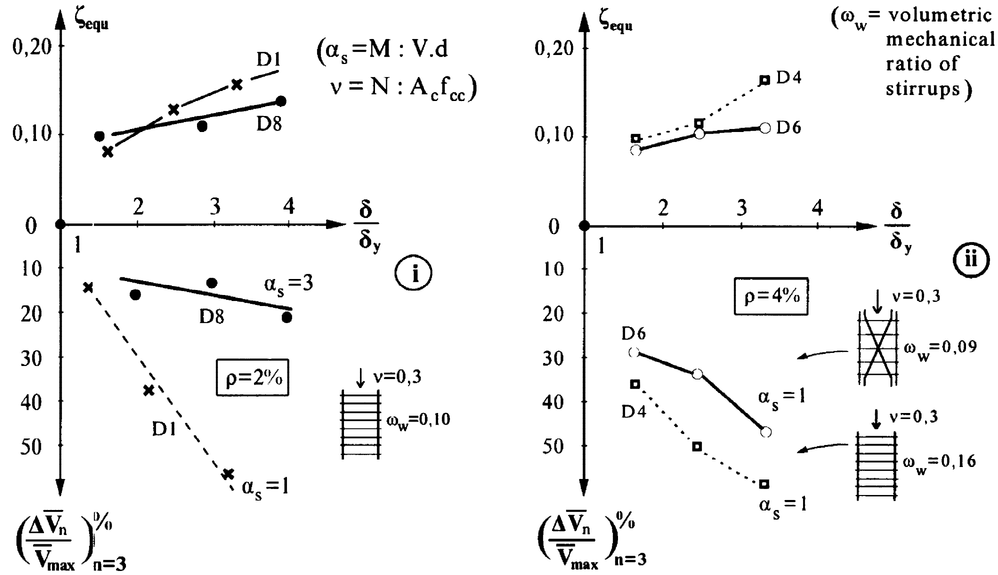


Fig. 9: Short columns under seismic conditions: (i) the role of shear ratio "α_s" and (ii) of longitudinal steel ratio "ρ", on force response degradation (mean values) and equivalent damping, as a function of imposed ductilities (after three full cycles at each ductility level)

4.2.-Sliding shear failure

Ductile structural walls being recognised as a highly reliable element in aseismic design, their energy dissipation capacity should not be reduced by potential sliding displacements. In fact, as it is known, in plastic regions, large tensile cracks may not be able to close completely during the subsequent load reversal; thus, sliding displacements may be observed.

Several measures can avert such a danger

a) Vertical bars along the web of the wall insure a better crack control

b) Limitation of the diagonal compression forces to avoid web crushing along the critical cross section. To this end, it has to be said that a more conservative limit than specified in EC 8, (P.1-3, §2.11.2.1.2) should be adopted, so that larger post-yield excursions be taken into account. Thus,

$$\text{if } q > 3,\ \ V_{Rd2} = k(0{,}7\text{-}f_{ck}/200)f_{cd}.b_{w0}.z \qquad (17)$$

$$\text{with } k = \frac{2}{3}(1 - q/8) \ \lhd 0{,}4$$

where

q = the actually used behaviour factor value (if higher than 3)

f_{ck} = characteristic value of concrete compressive strength (MPa), ($f_{ck} \triangleright 40$)
$f_{cd} = f_{ck} : 1{,}5$, design value of concrete strength
b_{wo}, z the width of the web and the level-arm length of the wall

c) Verification of potential sliding:

$$V_{Sd} \triangleright V_{dd} + V_{id} + V_{fd} \tag{18}$$

where $$V_{dd} = \min\begin{cases} 1{,}3\sum A_{sj}\sqrt{f_{cd}f_{yd}} \\ 0{,}25\, f_{yd}\sum A_{sj} \end{cases} \tag{19}$$

the dowel resistance of vertical bars

$$V_{id} = \sum A_{si} f_{yd} \cos\varphi \tag{20}$$

the shear resistance of inclined bars (angle φ as in Fig. 10)

$$V_{fd} = \min\begin{cases} \mu_f[\xi(f_{yd}.\sum A_{sj} + N_{Sd}) + M_{Sd}/z] \\ 0{,}25 f_{cd}\xi.l_w.b_{w0} \end{cases} \tag{21}$$

the friction resistance

$\sum A_{sj}$ = sum of the areas of vertical bars of the web
$\sum A_{si}$ = sum of the areas of all inclined bars (in both directions)
$\mu = 1$, friction coefficient
ξ = the mormalised depth of neutral axis

If inclined bars are provided, the corresponding increase of flexural resistance should be taken into account when the acting shear V_{Sd} is computed by capacity design criteria.

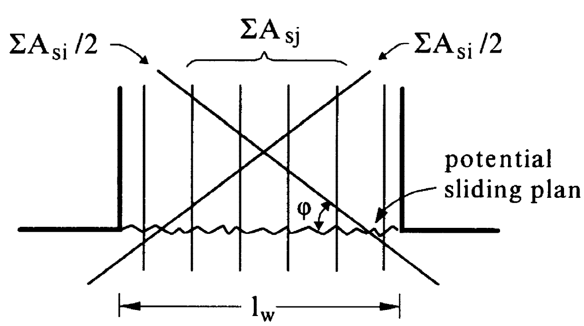


Fig. 10: Potential sliding of a R.C. wall and related reinforcements

5.- DUCTILITY VERIFICATION

In spite of important advances towards a more quantified expression of the global ductility of buildings, most of the "energy dissipation" philosophy remains to be qualitative and fragmentary. Some of ductility related issues will be discussed in this section.

5.1.-First, an oversimplified expression of the **curvature ductility factor** $\mu_{1/r}$ should be reminded (as derived in Fig. 11) in order to reiterate the importance of the parameters entering the ductility game:

$$\mu_{1/r} \cong \frac{\varepsilon_{cu}}{\varepsilon_{sy}+0{,}0015}:[\nu+\frac{f_{sy}}{f_c}(\rho-\rho')] \tag{22}$$

where

$\varepsilon_{cu} = \varepsilon^*_{c.85}$ (Equ.7) ultimate strain of confined concrete at a stress response level of $0{,}85f_c$

f_{sy},ε_{sy}, denote yield strength and yield strain of steel (i.e. steel response ignoring hardening effects)

ρ, ρ' tensile and compressive reinforcement ratios

λ, the stress of compressive reinforcement normalised to yield strength (λ~0,7 to 1,0)

$\nu = N:A_c.f_c$, the normalised axial force value

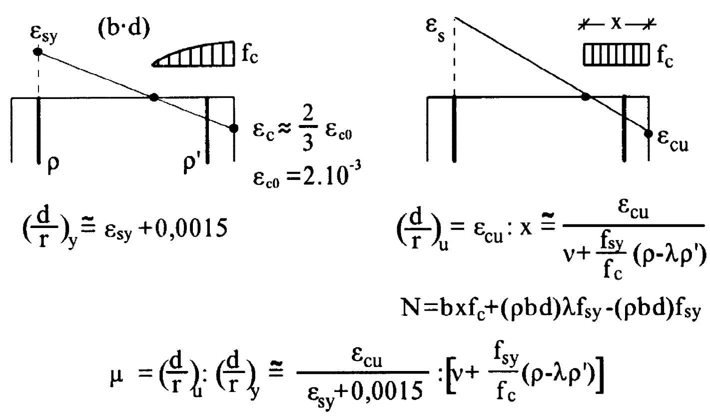


Fig. 11: Derivation of an oversimplified expression for curvature ductility factor

Thus, the importance of the following conditions favouring ductility comes apparent, once more, without sophisticated modelling:

- substantial confinement
- use of low-yield reinforcement

- generous cross-sectional dimensions of columns, so that normalised axial load takes low values
- provision of compressive reinforcement ratio ρ' as close as possible to the tensile ratio ρ.
- use of concrete exhibiting relatively high strength.

5.2.-However, in reality, the precise **definition** and the suitability of ductility factors to adequately describe the energy dissipation capacity of a critical region continue to be a rather unsettled issue. A ductility definition which takes into account both the plastic deformation capacity and the resistance degradation observed, as proposed by Maruyama et al. (1989) (as illustrated in Fig. 12), was found to be a better estimator of the overall performance of several systems of coupling beams (Tassios, et al., 1996).

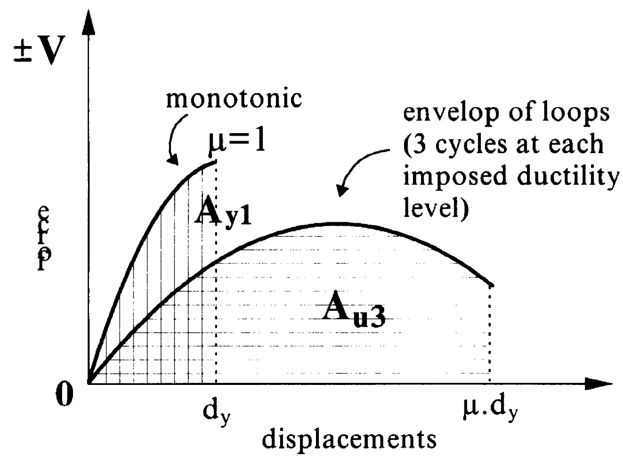


Fig. 12: Definition of a "pseudo stored"energy ductility factor $\mu_E = A_{u3} : A_{y1}$

5.3.-In spite of the improvements observed in the formulation of confinement requirements, it seems that the New Zealand Code (1995) specifies considerably lower confinement than Eurocode EC8 (1994); yet, according to Eibl et al. (1995), these Eurocode requirements are considered as non-conservative!

To the knowledge of the author of this S.O.A. report, the provisions of EC8 for confinement were experimentally found (Chronopoulos et al., 1995) to lead to an average bending moments' degradation of 20% for the range of ductilities and normalised axial force values used in EC8; nevertheless, a very large scattering of results was observed.

On the basis of this contradictory information, the issue of code-provisions for the design of confinement can hardly be considered as finalised.

Regarding the unusually large scattering of results reported by Chronopoulos et al., 1995, it is worth noting that, up to now, this fundamental design aspect of confinement has not been treated probabilistically. However, in a study on this topic, Trezos (1996) concludes that upper 5% fractile values of required confinement could be as larger as 50% above the mean values specified in Codes; and only highly redundant structures can tolerate it. Thus, further conservatism seems to be needed.

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