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R.C. WALL CROSS - SECTION DESIGN FOR A GIVEN BEHAVIOUR FACTOR OF THE BUILDING

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SUMMARY

A practical method is presented, incorporating the ductility requirement directly into the strength verification of R.C. structural walls of buildings analysed with a given seismic behaviour factor q.

1. INTRODUCTION

The design method proposed in this paper, is an attempt to secure numerical interrelationship between the q-factor used for the design of the building, and the confinement conditions of the boundary areas of the wall (i.e. the required confined length "l_c" and the volumetric mechanical percentage "ω_{wd}" of the closed stirrups and or cross-ties available).

Essentially, this method follows the procedure proposed by Paulay, Uzumeri, 1975: The overall behaviour factor "q" is (roughly though) translated into displacement ductility factor "μ_d", which is subsequently expressed in terms of the local curvature ductility factor "$\mu_{1/r}$".

What then remains is to secure that this local ductility demand will be available in the critical region of the wall, thanks to its appropriate dimensions and detailing.

The nature of the related multiparametric and complex post-yield phenomena justifies a set of approximations introduced along the analysis. Thus, a rather low level of predictive precision is claimed. However, it is believed that the consequencial insight offered by such an approach is a positive step in rational aseismic design, as compared to the actual semi-empirical Code-provisions.

2. WORKING ASSUMPTIONS

To serve the purpose previously described, the following simplified assumptions are needed.

a) <u>How to relate "q" with "μ_d".</u>

Due to the reduced natural vibration periods of the structural systems considered, the equal energy principle is adopted in assessing their elastoplastic behaviour. Thus:

$$q = \sqrt{2\mu_d - 1} \qquad (1)$$

or $\mu_d = 0,5(q^2 + 1).$ (1a)

b) <u>How to relate "μ_d" with "$\mu_{1/r}$".</u>

In Fig. 1 the derivation of Equ. 2 is reminded. Thus:

$$\mu_{1/r} = 1 + \frac{1}{\beta} \cdot \frac{c(\mu_d - 1)}{\lambda_p(1-0,5\lambda_p)}, \tag{2}$$

where

β = coefficient reflecting the distribution of seismic forces along the height of the wall (Fig. 1a)

c = coefficient accounting for possible non-fixity foundations conditions, as introduced by Pristley, Park, 1984

μ_d = top displacement ductility factor

λ_p = $l_p:h_w$, normalised height of the critical region ("plastic hinge" length) of a wall (of total height h_w). Within this paper, based on suggestions of P. Steidle et al. 1986, R. Wohlfahrt et al. 1986 and Pristley, Park, 1984, it has been taken

$$\lambda_p = 0.6\, l_w : h_w. \tag{3}$$

c) <u>Approximate expression of base curvature at yield</u>

With the indications of Fig. 1c,

$$\left(\frac{1}{r}\right)_y = \frac{M_y}{(E_cJ)_y} = \frac{0,8\, M_R}{0,6E_cJ} = \frac{4}{3}\,\frac{M_R}{E_cJ}. \tag{4}$$

For a better approximation, see §3a, Equ. 10.

d) <u>Constitutive law of confined concrete</u>

The stress-strain relationship of concrete within the confined end-areas of the wall is approximated as shown in Fig. 2 and described by the following equations (based on Tassios, Lefas, 1984):

Strength of confined concrete,

$$f^*_{cc} = f_{cc}(1+2,50\cdot\alpha\omega_{wd}) \quad \text{for} \quad \alpha\omega_{wd} < 0,10 \tag{5a}$$

$$f^*_{cc} = f_{cc}(1,125+1,25\cdot\alpha\omega_{wd}) \quad \text{for} \quad \alpha\omega_{wd} > 0,10 \tag{5b}$$

Peak concrete strain,

$$\varepsilon^*_{co} = \varepsilon_{co}(f^*_{cc}:f_{cc})^2 \tag{6}$$

Concrete strain at $0,85f_{cc}$ level,

$$\varepsilon^*_{c,max} = 3,5\cdot10^{-3} + 0,1\cdot\alpha\omega_{wd} \tag{7}$$

where:

f_{cc} = unconfined concrete strength

α = $\alpha_s\alpha_n$, the effectiveness of lateral confinenment (s. Fig. 2b); for the simple configuration of lateral reinforcement feasible in walls, nd for stirrups or cross-ties provided every $s \cong 1/3b_c$, the factor "α" has been taken equal to $\alpha = 0,25$ along this paper.

ω_{wd} = design value of the volumetric mechanical ratio of confining reinforcement

$$\omega_{wd} = \frac{\text{volume of conf. steel}}{\text{volume of core concrete}} \cdot \frac{f_{yd}}{f_{cd}}.$$

3. DUCTILITY DESIGN OF WALL'S CRITICAL REGION

Following the conventional definition of the curvature ductility factor (Fig. 1a),

$$\mu_{1/r} = (\frac{l_w}{r})_u : (\frac{l_w}{r})_y = (\varepsilon_{su} + \varepsilon_{cu}) : (\frac{l_w}{r})_y \qquad (8)$$

and on the basis of Equ. 2 and Equ. 4, the following expression is found:

$$\underline{\varepsilon_{su} + \varepsilon_{cu} = \left| 1 + \frac{1}{2\beta} \cdot \frac{c(q^2-1)}{\lambda_p(1-0{,}5\lambda_p)} \right| \cdot \frac{4}{3} \frac{M_R}{E_c J}} \cdot \qquad (9)$$

The following design steps are suggested:

a) The critical region of the wall is first dimensioned under the minimum axial force N_{min} and the corresponding acting bending moment M_{S1} under the seismic load combination. Vertical steel bars against axial actions may be provisionally considered as being spread at legnths $l_c' = \max(0{,}2l_w,\ 2b_w)$ on both ends. Thus, longitudinal reinforcements are determined.

b) The "final" dimensioning (taking also into account the ductility requirement of Equ. 9) should now be carried out under N_{max} and the corresponding acting moment M_{S2}. Shear dimensioning and verification should be carried out first under the N_{max}; it shall be confirmed that flexurul failure precedes shear failure. Now, regarding axial effects, among all possible parallel lines of the rotated base cross-section (i.e. among the several x_u-values, Fig. 3) as dictated by Equ. 9, that line will be retained which satisfies equilibrium conditions:

$$N_{max} = F_c - F_s = (F_{cc} + \sum_i A_{si} \cdot \sigma_{si}) - \sum_j A_{sj} \cdot \sigma_{sj} \qquad (10)$$

$$N_{max} \cdot \frac{l_w}{2} + M_{S2} = F_c y_c - F_s y_s \qquad (11)$$

with notation as explained on Fig. 4.

On the other hand, under the assumption that plane sections remain plane after bending,

$$x_u = (l_w - 2c) \cdot \varepsilon_{cu} : (\varepsilon_{su} + \varepsilon_{cu}) \qquad (12)$$

$$l_c = x_u \cdot (\varepsilon_{cu} - \varepsilon_{c,lim}) : \varepsilon_{cu} \qquad (13)$$

where

$\varepsilon_{c,lim} = 1{,}5 \cdot 10^{-3}$, the concrete strain under which the material may be considered as elastically stable and does not need confinement.

The five unknowns of the problem (i.e. $\varepsilon_{su}, \varepsilon_{cu}, x_u, l_c, \omega_{wd}$) may now be determinted by means of the five available Equations (9),(10),(11),(12),(13). Of course, a trial and error method may be followed, taking initial arbitrary ω_{wd}-values (possibly guided by Equ. 15).

Obviously, the new l_c-values (step "b") differ from the initial l_c'-values (step "a"); consequently, a slight modification of bearing capacity is expected and a convergence is needed.

4. PARAMETRIC STUDIES

A limited series of numerical studies has been carried out for R.C. buildings, accounting for the following intercombined parameters:

. Three q-factor values (2.5, 3.5, and, to a limited extent 4.5)
. Three number of storeys (3, 5 and 7)
. Two total floor-areas were considered (250 m^2 and 500 m^2).

The walls were consideredas fully fixed on soil. In all cases, an effective bedrock acceleration equal to $\alpha_g = 0{,}30g$ was considered, leading to a base shear coefficient equal to $\varepsilon = 2{,}5\alpha_g/g : q$. (14)

A static analysis was carried-out (with inverted triangular distributions of seismic loads); appropriate magnifications were introduced, accounting both for static torsion and for higher modes effects, as suggested by aseismic codes.
In each of the fifty cases examined, the length of the walls was selected with an engineering judgement out of the following dimensions: $1{,}50^m$, $2{,}00^m$, $3{,}00^m$ and $4{,}00^m$. Their thickness were always equal to 250 mm; materials were C20/S400.

Because of the numerical procedure followed, the degree of precision achieved was not uniform through the entire number of cases examined.

A summary presentation of the results is shown on Fig. 5; the ductility-related values of "l_c" (final length of confined end-areas) and "ω_{wd}" (design volumetric mechanical ratio of confining reinforcement provided at distances $s=1/3b_c$) are illustrated against a practical e s t i m a t o r of the axial actions i.e. the sum $(\bar{\nu}_d+\bar{\mu}_d)$ of

- the normalised axial compressive force $\bar{\nu}_d = N_{Smax} : b_w l_w f_{cd}$ and
- the corresponding normalised flexural moment $\bar{\mu}_d = M_{S2} : b_w l_w^2 f_{cd}$.

On the basis of this, rather limited, numerical investigation, the following empirical (and somehow conservative) expressions may also be retained, which could be useful for a pre-estimation of the relevant quantities before the final analytical verification of R.C. walls:

$$\omega_{wd} = \left(\frac{q}{2{,}5}\right)^2 \cdot \left|(\bar{\nu}_d + \bar{\mu}_d) - 0{,}05(4{,}0-q)\right| \qquad (15)$$

$$l_c = 0{,}10 + 0{,}45\,(\bar{\nu}_d + \bar{\mu}_d)\ (\not> 0{,}50). \qquad (16)$$

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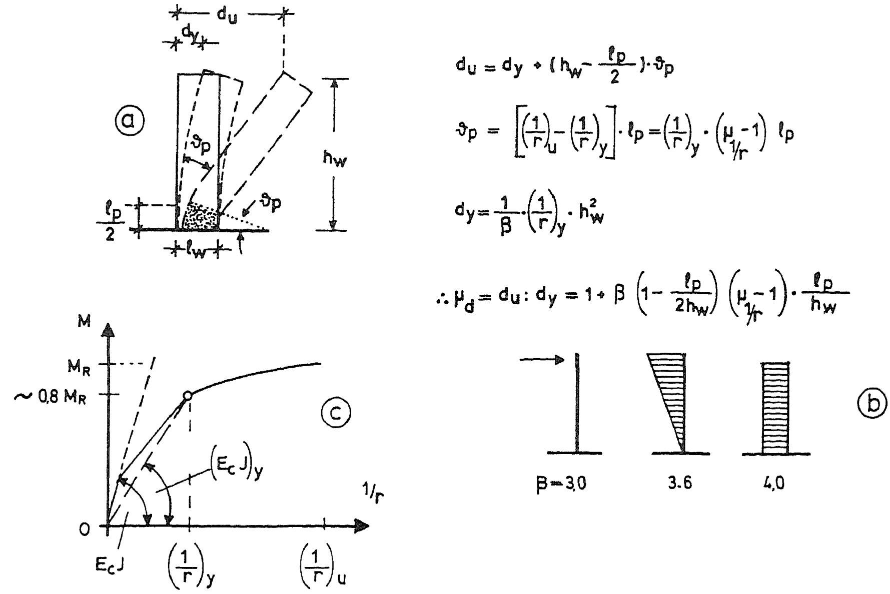

Fig. 1: Relationship between curvature and displacement ductility factors ($\mu_{1/r}$ and μ_d)

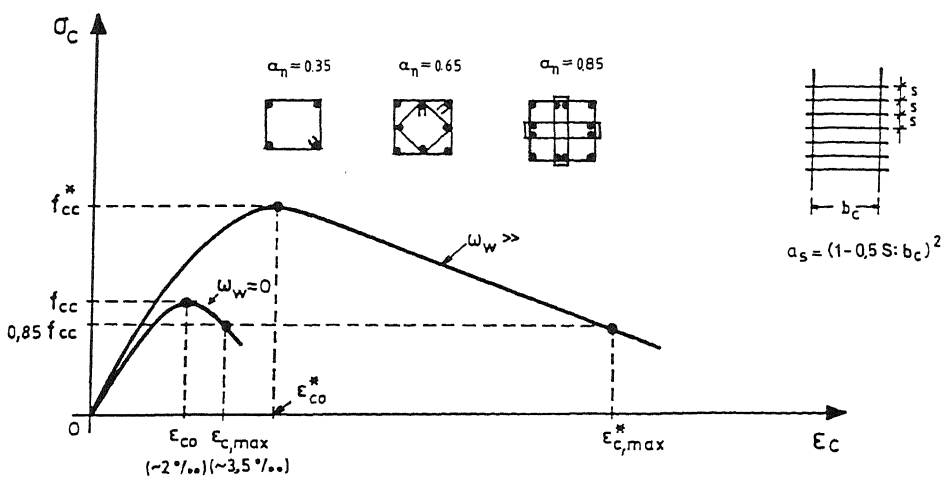

Fig. 2: Practical constitutive law of confined concrete

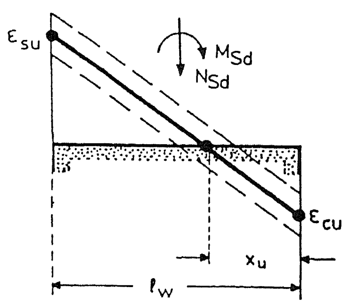

Fig. 3: All these parallel positions of the rotated base, observe the overall ductility requirement of Equ. 9

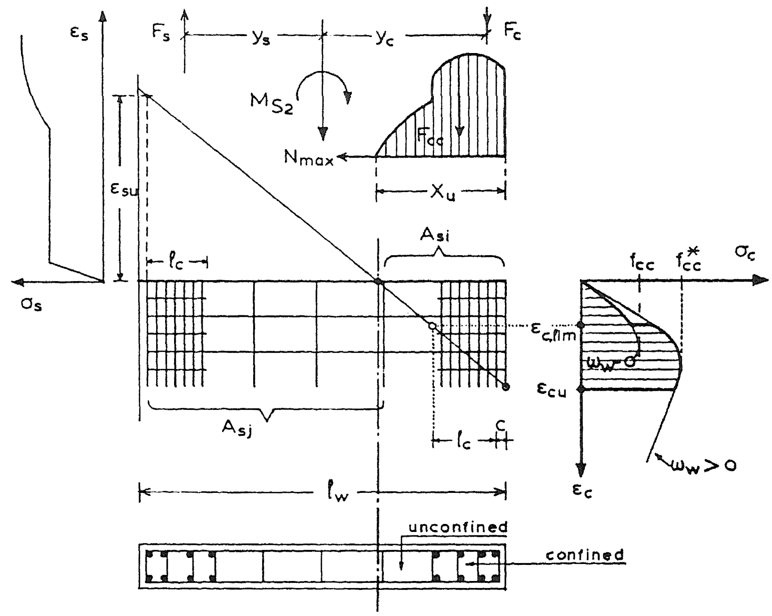

Fig. 4: Stress and strain distributions in base cross-section under the final seismic load combination

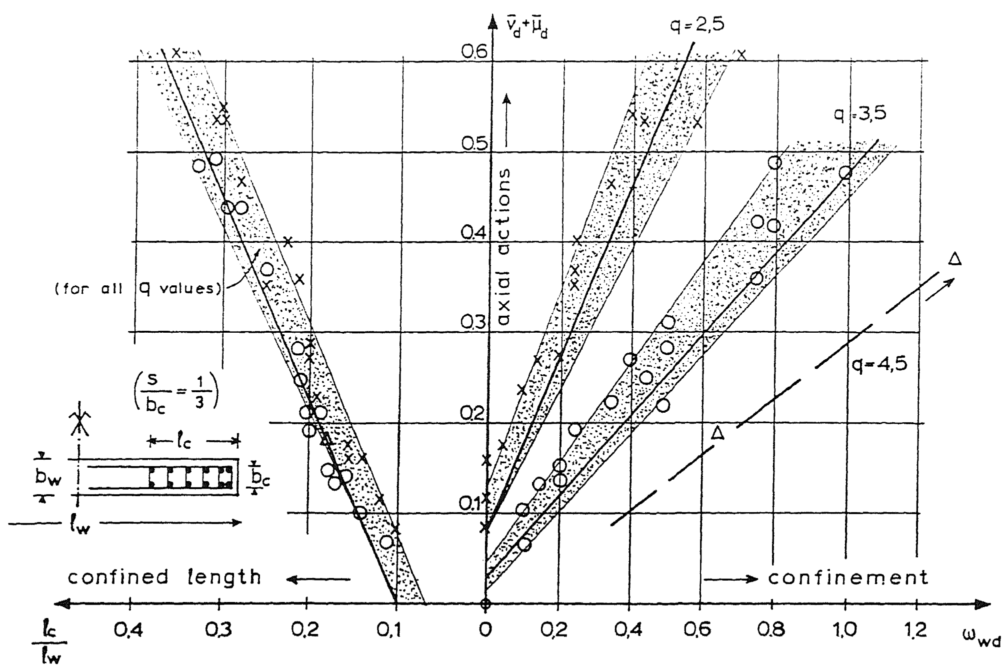

Fig. 5: Volumetric mechanical confining steel ratio ω_{wd} and normalised confinement length $l_c:l_w$ of the end-areas of R.C. walls, as functions of the sum of normalised axial force and flexural moments