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Plastic design of steel EBFs connected to R.C. walls

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ABSTRACT: We analyze the behaviour of steel buildings resisting seismic horizontal forces by means of reinforced concrete shear walls (RCW) and eccentrically braced steel frames (EBF). We propose a plastic design method for EBF members based on a distribution law of the total shear to each structure linearly changing along the height and depending on the inertia properties of the base sections. Furthermore, we recommend a new arrangement of the braces at the first level of EBFs and test the proposed design procedure by an elastic analysis of a ten story-building.

1 INTRODUCTION

The European recommendations for steel structures in seismic zones (EC8) consider the following two types: non-dissipative structures and dissipative structures.

The first should be designed to behave elastically, even in a major earthquake, in accordance with the standard code (EC3), assuming a behaviour factor q=1.

The second must behave elastically only against minor earthquake ground shaking, but must not collapse in a major earthquake. In this case, significant damage to the structure and non-structural components is acceptable. In order for a structure not to collapse it must have a large dissipation capacity during large inelastic deformations. In general, structural systems wich exhibit stable hysteretic loops perform well under the large inelastic cyclic loadings characteristic of a major earthquakes. Such stable hysteretic characteristics of a structure can be obtained provided that the structural members and joints are designed to possess sufficient ductility. The recommendations consider seven types of dissipative structures, but the dual systems that we studied are not included. two structures will be able to satisfy both the seismic standards and the fire rules about stairs and lifts.

In previous papers we proposed a simplified plastic design method, wich considers the following steps: i) choosing the rate of shear carried by each system; ii) designing RC walls and EBFs as independent structures; iii) developing a linear elastic analysis of the coupled system and checking the previously assumed distribution; iv) going to the first step and assuming a new shear distribution on the basis of the previous results.

We have shown (Scibilia 1990) that the procedure convergence can be improved by assuming rates of total shear changeable along the height of the building.

In the present paper we propose a distribution law of the total shear linearly changing along the height and depending on the inertia properties of the base sections and we develop the plastic design of the structures. Furthermore, we recommend a new arrangement of the braces at the first level of EBFs in order to translate the plastic hinges of the column bases from the first to the second level and we present some consideration about

2 PLASTIC DESIGN OF R.C. WALLS

The fundamental requirement in the design of ductile structural walls is to avoid brittle failure mechanisms. The principal source of energy dissipation in a cantilever wall should be the yielding of the flexural reinforcement in the plastic hinge region at the base. Failure modes to be prevented are those due to diagonal stresses caused by shear, instability of a thin walled section and bond failure.

In rectangular-shaped sections it is necessary to confine the extreme portions with transversal reinforcement, as shown in Fig.1. In this way we obtain two results:
i) an increase in plastic concrete strain;
ii) prevention of buckling of the principal wall reinforcement.

We evaluate the first effect using the Kent and Park curve for concrete confined by rectangular hoops (Fig.2).

For steel reinforcements we use a trilinear stress-strain curve, where the increase in stress due to strain hardening is considered linear with slope equal to E/50 (Fig.3).

The theoretical moment-curvature relationship (m-ϕ) for a given axial load level is determined by a computer programme supposing that the cross section remains plane and using the above stress-strain curves.

The curvature ductility μ_ϕ expressed by the ratio between the ultimate and the yielding values (ϕ_u / ϕ_y) depends mainly on the axial load.

We suppose that the collapse mechanism is due to the flexural plastic hinge at the wall base and we assume that the displacement ductility μ_d is equal to δ_u / δ_y, where δ_u is the top building lateral deflection at the end of the post-elastic range, and δ_y is the lateral deflection when yield is first reached.

Neglecting the effects of migration of the neutral axis and of the loss of bond resistance we can calculate the displacement δ_u at the top of the building using the following expression:

$$\delta_u = \delta_e + \theta_f H + (\phi_u - \phi_y) l_p (H - 0.5 l_p) \quad (1)$$

where δ_e is the elastic deformation of the cantilever and θ_f is the foundation rotation.

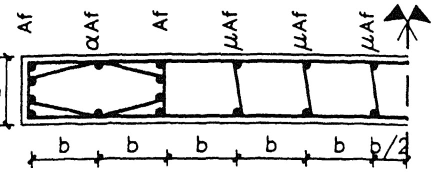

Fig. 1 Transversal wall section

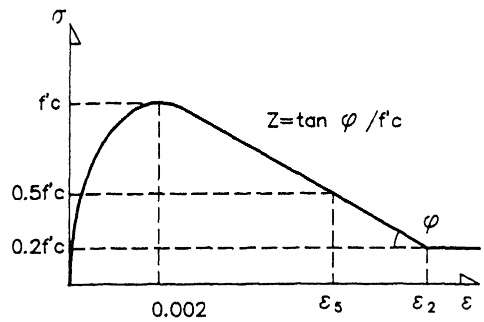

Fig. 2 Kent and Park σ-ε curve

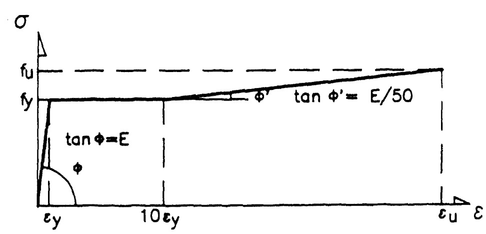

Fig. 3 Steel trilinear σ-ε curve

We can evaluate with easily δ_e by introducing for the stiffness of the cracked wall a fictious value I* of the moment of inertia I ($I^*=\psi_1 I$).

To evaluate the equivalent length of the plastic hinge l_p, Paulay (1987) assumes :

$$l_p = 0.5 \quad \beta b + 0.05 \quad H \qquad (2)$$

where βb is the height of the cross section and H the building height. On the basis of EC8 we assume that l_p is equal to the greatest of the following values:

- length βb of the wall;
- inter-story height h;
- 1/6 of the total height H.

By the EC8 rules the regular building braced with RC walls can have a structural factor q equal to 4.5 and the q-μ_d law is:

$$(2 \mu_d-1)^{1/2} < q < (q^2+1)/2 \qquad (3)$$

wich is satisfied for μ_d changeable from q to $(q^2+1)/2$. From here on we assume that μ_d is equal to q and that we can evaluate q using computer programmes, as shown in (Scibilia 1991). In any case is advisable to limit the axial load below 0.15 of the axial strength value and to reduce the hoop distance p (p<=b/2).

3 PLASTIC DESIGN OF EBFs

The basic charateristic of EBFs is the eccentricity of the braces to beam connections, which allows energy dissipation by means of plastic cyclic shearing deformations concentrated in the active links.

With reference to the part of V-EBF in Fig.4, we consider the dissipation mechanism which has the links yielded in shear. When the EBFs are isolated the mechanism involves the yielding of the first level column bases, whereas in the coupled systems it is better to translate the plastic hinge to the same level as the hinge to the RC wall.

The plastic design methods for EBFs select the active links for the required plastic capacity and the other members must be so designed that they remain elastic. They are based on the limit analysis approach and use the lower and upper bound theorems for ideally plastic structures. However, it was very difficult to obtain a reasonable estimate of member forces and satisfactory design employing these methods without other rules.

The current method was presented by Kasai and Popov (1984). They neglected the small brace moments, and they assigned the location of the column inflexion moment points, and the link length, with respect to the span L, included in the range 0.1 L to 0.3 L. With few iterations an optimized solution is obtained.

The numerical tests developed with the rigid-plastic method previously indicated appear to agree well with the elasto-plastic solutions.

To avoid local web buckling we must use stiffeners proportioned to link deformations.

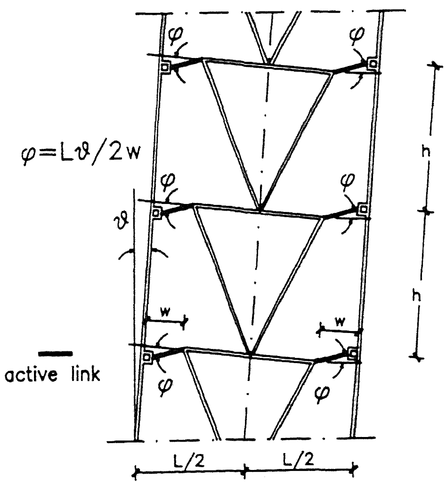

Fig. 4 VEBF dissipation mechanism

4 ANALYSIS OF THE COUPLED SYSTEMS

If the wall geometrical dimensions and the EBF length L are known we can design the EBF members and the steel reinforcement bars of the wall to withstand seismic forces with

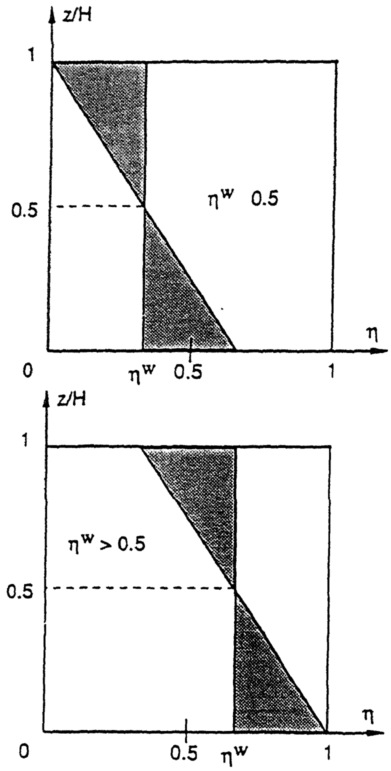

Fig. 5 TOTAL SHEAR RATE COEFFICIENT

assigned ductility and with controlled deformations.

We calculate the total shear forces T^T at half height, we choose the rate η^w to attribute to the wall ($T^w = \eta^w T^T$) and we suppose that the values η are linearly changeable with the height, inside the fields shown in Fig. 5 bounded by the following lines:

$$\eta = \eta^w; \ \eta = 2\eta^w\left(1-\frac{z}{H}\right) \quad (\eta \le 0.5) \quad (4.a)$$

$$\eta = \eta^w; \ \eta = 2\left(\eta^w - 1\right)\frac{z}{H}+1 \quad (\eta > 0.5) \quad (4.b)$$

An approximate criterion useful to determine the value of η^w can be obtained using the following expression:

$$\eta^w = (A^w/6)/(E_s/E_c\ A^s + A^w/6) \quad (5)$$

where A^w is the area of the RC wall, A^s the area of the frame column bases, and E_s and E_c the Young's modulus of the steel and of the concrete.

The above relation comes from the semplifying hypothesis that the rate coefficient η^w depends on the values of the flexural inertia of the base sections of the two structures:

$$I^w = b\ L^3 \psi_1/12 = A^w\ L^2\ \psi_1/12 \quad (6.a)$$

$$I^{ebf} = 2\ A^s\ L^2\ \psi_2/4 = A^s\ L^2\ \psi_2/2 \quad (6.b)$$

and assuming that the coefficents ψ_1 and ψ_2 have the same value.

Imposing the η^w value from (5), it is possible to evaluate A^s and, if necessary, to change the previous value.

Choosing the line inside the dashed field we can evaluate the bending moment at the wall base and the steel reinforcement using the m-n curves. Then we can check the available ductility by the q-n relations .

By this test we have a first valuation about the assumed function η .

The EBF design can be developed by the plastic method mentioned before with reference to the total shear $(1-\eta)$ T.

Furthermore the behaviour of the structures suggests changing the EBF layout, arranging the braces at the first level as concentric K frames. Thus the plastic flexural hinge in the steel coloumn develops at the second level base ,at the same height as the centre of the wall plastic hinge, so that when all links are yielded the plastic deformations are congruent.

Then we develop the linear elastic analisys of the coupled system, checking the hypothesis about the assumed shear distribution and yield of the links.

5 ANALYSIS PERFORMED

In order to evaluate numerically the coupled system behaviour, we develop the design for a tipical steel building.

The structure investigated has the rectangular plan (12*26 m) shown in Fig.6 and is ten stories high.

We consider two different EBF

Table 1. HEA size for the links

Floor	1	2	3	4	5	6
V-EBF	160	160	200	200	220	220
MEBF	200	160	200	200	220	220

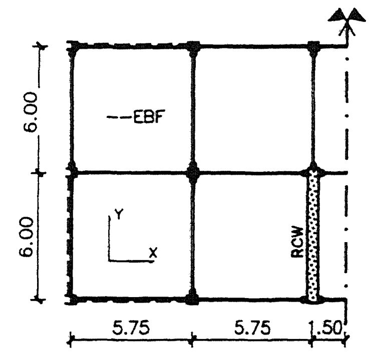

Fig. 6 Structural plan of building analysed along Y direction

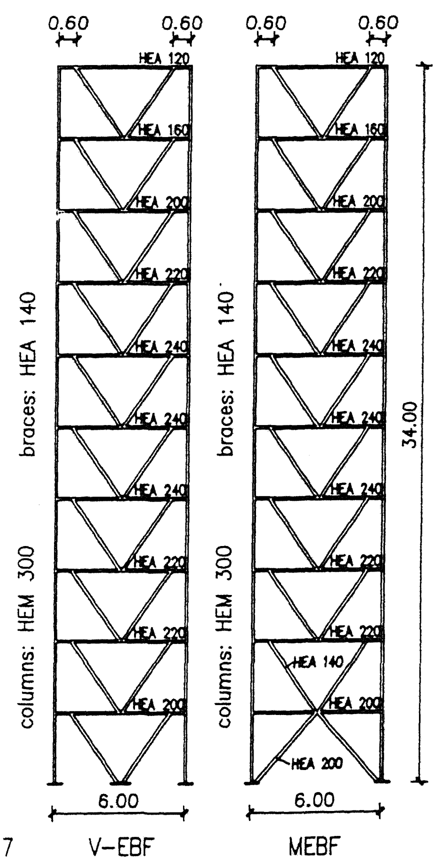

Fig. 7 V-EBF MEBF

layouts shown in Fig.7:
i) the typical arrangement (V-EBF);
ii)the modifited structures (MEBF)
and we perform the analysis.

We evaluate the seismic forces F_i by an equivalent static analysis using the well-know relationship :

$$F_i = W_i \gamma_i R(T) A / q \qquad (7)$$

We assume for all floors the same value of W_i equal to 2700 kN and the ground acceleration A equal to 0.4 g.

We select the following geometric dimensions for the walls:
b= 0.30 m, a= 6.00 m, H= 34 m and we design the reinforcement and the EBF beams following the European standards for H rolled shapes (HEA, HEB, HEM).

We neglect the floor stiffness and connect the two structures by truss elements. By means of (5) we deduce that we must choose values of η^w greater than 0.6 in order to limit the column steel area. We set η^w equal to 0.6 and $\eta(z=0)$ equal to 0.80 and we apply the plastic design method for the EBF.

In the designs proposed the column cross sections are the same along the height (HEM 300), as are the braces cross sections (HEA 140), while the beam size are designed following two different criteria:
i) choosing sections having yield shear greather than the value required by the plastic balance of the EBF;
ii)using lighter steel shapes at the lower levels in order for all the links to yield simultaneously .

The beam sections designed with the first criterion are shown in Fig.7, while the other different sections , in the second design, are reported in Tab. 1

Elastic analysis results,referred to ψ_i=0.6, are shown in Figs. 8-9-10.

The total shear distribution is presented in Figs.8-9 referring to the first design criterion for the links, as there are no significant variations due to the lighter beam sections.Following the second design criterion it is possible for all the links to yield simultaneously, as is shown in Fig.10.

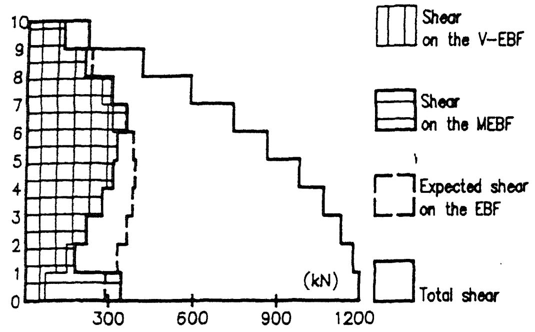

Fig. 8 Total shear distribution

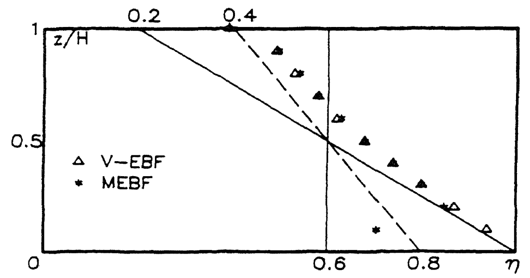

Fig. 9 Total shear distribution law η

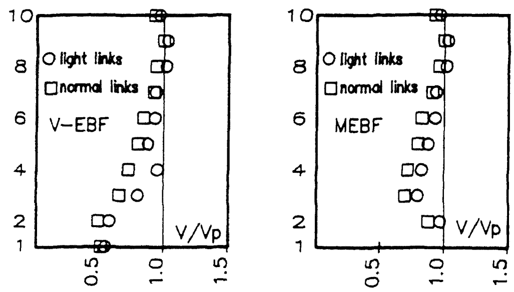

Fig. 10 Shears in the links

6 CONCLUSIONS

The analysis of the previously illustrated coupled systems shows the possibility of using RC walls together with EBFs to resist seismic forces.
From our results we can make the following considerations:
i) the total shear ratio to attribute to each system can be evaluated by means of (5);
ii) it is better to change the brace layout at the first level in order to permit the development of the assumed plastic mechanism;
iii) it is opportune to reduce the beam sections at the lower levels in order for all the links to yield simoultaneously. However, we are now searching for a different stuctural arrangement.

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