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DESIGN OF SQUAT SHEAR WALLS USING CONTINUOUS STRUT-AND-TIE RESISTANT MODELS

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ABSTRACT

A new approach is presented for designing squat shear walls, subjected to seismic loading. In this approach, the strut-and-tie resistant models, proposed by Schlaich et al., are generalized in order to analyze continuous fields of compression and tension stresses. The application of the presented methodology is illustrated through numerical examples.

1. INTRODUCTION

In the stiff structures found in Nuclear Power Plants, the seismic forces are usually resisted by a structural system composed by horizontal (slabs) and vertical (walls) bidimensional elements. These elements do not possess, in general, one of their dimensions preponderant with respect to the other ones; standard criteria for the flexural and shear design of slender beams cannot be, therefore, directly applied. These elements present, also, in general, a great number of openings; then, their design becomes a very difficult task.

The application of Schlaich´s et al. (1987) strut-and-tie models to the design of bidimensional elements under seismic loading has been studied by the authors, Santos and Silva (1989, 1990, 1991), and Santos (1992). In this paper, a generalization of Schlaich´s models is proposed, for analyzing continuous fields of compression and tension stresses.

The justification for proposing this approach is that the current procedure for analyzing squat shear walls, as short corbels, do not consider the actual lay-out of reinforcement to be placed in the walls: continuous and uniform ortogonal grids of reinforcing bars. Usual short corbels design procedure leads to concentration of vertical reinforcement in the extremities of the walls; contribution of the horizontal reinforcement is neglected.

The analysis of the walls according to the proposed approach is helped through analytical expressions, displayed in adequately prepared figures.

2. ANALYSIS WITH STRUT-AND-TIE MODELS

The seismic-resistant structural elements in a NPP shall be designed for a combination of static and seismic forces. The internal forces in the elements are generally obtained in the elastic range, through finite element analyses. Obviously, relevant discrepancies can appear between the structural behaviour in the elastic range and the behaviour when designing for high seismic forces, near the ultimate state. The design can be done using new sophisticated computer codes for non-linear finite element analysis of reinforced concrete, considering accurate constitutive laws for the concrete and an adequate representation for the reinforcing bars. Unfortunately, due to the involved computer costs, complete non-linear analyses of reinforced concrete elements are justified only in very important and special cases.

An alternative approach for the design is the application of Schlaich's strut-and-tie models. These models are based on the analysis of pre-defined plane trusses, where the struts represent the compressive stress fields in the concrete and the ties, the reinforcing bars. The design is developed in the following steps: definition of the design truss; determination, through static equilibrium, of the forces in the truss elements; check of the compressive stresses in the concrete, from the determined values of the compressive forces; design of the reinforcement from the values of the tensile forces in the ties; detailing of the reinforcement, following directly the adopted resistant truss model. It is suggested that the geometry of the truss models shall follow, as closely as possible, the elastic stress fields.

3. CONTINUOUS STRUT-AND-TIE MODELS

Fig. 1 shows the theoretical scheme for the analysis of an elementary squat shear wall without flanges.

The proposed scheme is only valid for $l/h>1$. A distributed horizontal load f is applied on the top of the wall. A local system of axes (x, y) is defined from the top of the wall and from a distance h of its right face, respectively. The applied load is continuously resisted in the top of the wall by an uniform tension $tv = f$ in the vertical reinforcement, anchored in the base, and by a compression $-2f$ in the inclined (45°) compression strut. The compression struts in the region with $y<0$ find equilibrium directly in the base of the wall; the struts in the region with $y>0$ are equilibrated by compression vertical forces in the right edge of the wall and horizontal tension forces that shall be transferred again to the left side of the wall. These horizontal forces, for finding final equilibrium in the base of the wall, will cause an increase dtv in the tension in the vertical reinforcement.

Maximum tension forces in the vertical reinforcement occur in the left side of the wall ($h-l<y<0$), being constant in this region. Maximum forces appear in the base, and are obtained by summing up the value already present in the top with the increases, throughout the height of the wall:

$$tmax = tv + \int_0^h dtv \qquad (1)$$

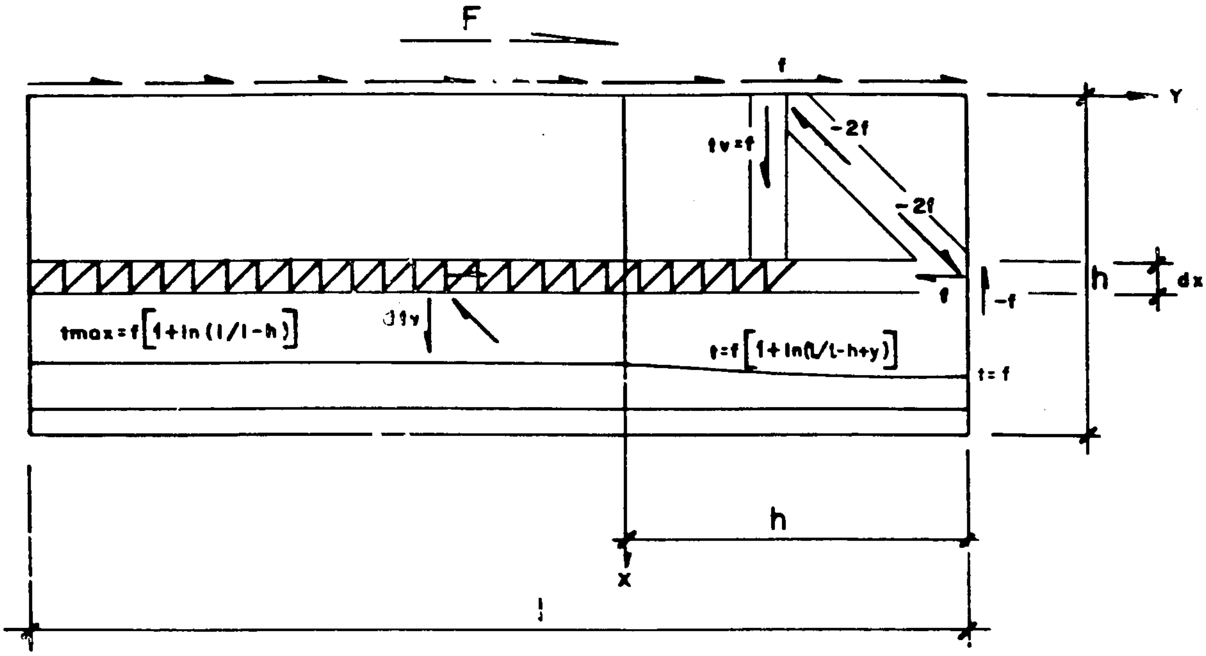
F
f
Y
tv=f
-2f
-2f
h
dx
f
-f
tmax=f [1+ln (l/l-h)]
dtv
t=f [1+ln(l/l-h+y)]
t=f
h
X
l

FIG. 1

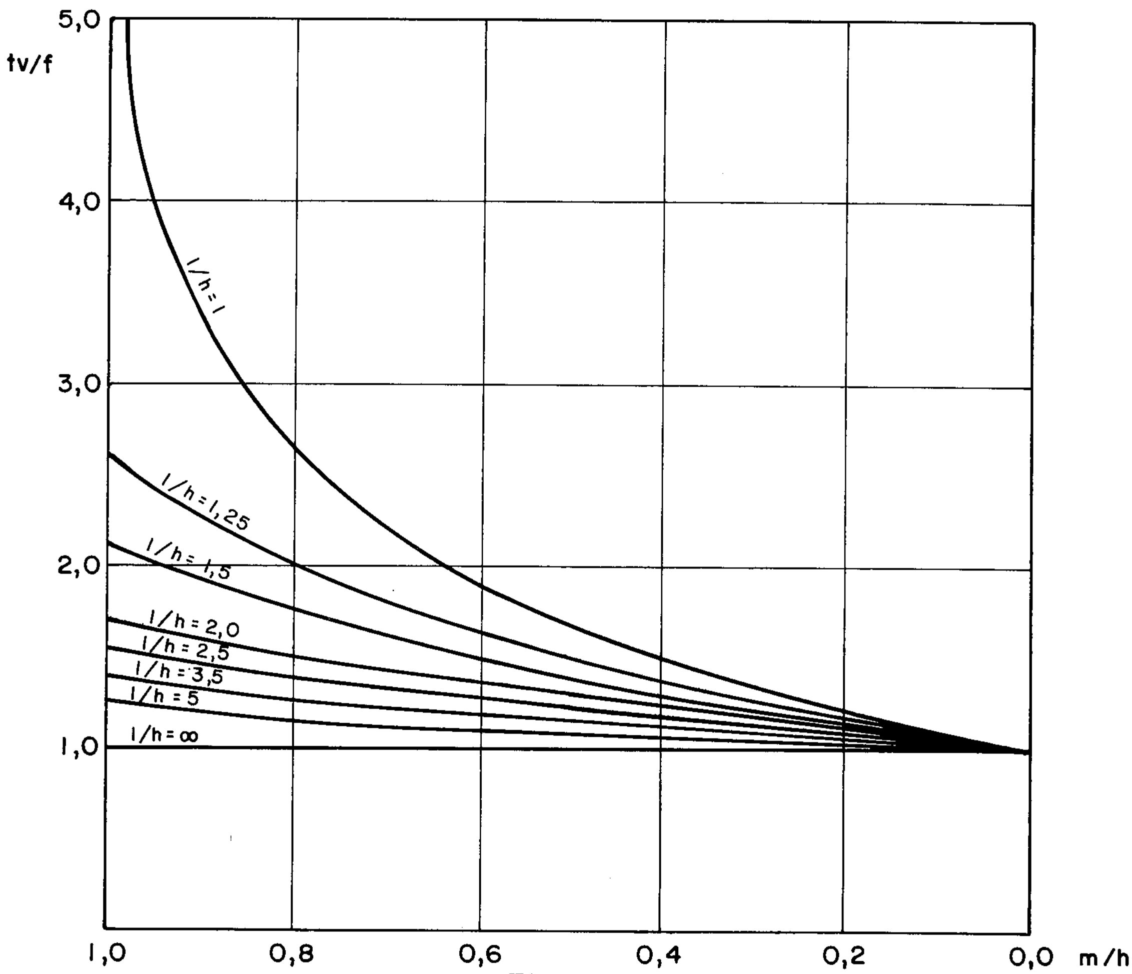
tv/f
5,0
4,0
3,0
2,0
1,0
l/h=1
l/h=1,25
l/h=1,5
l/h=2,0
l/h=2,5
l/h=3,5
l/h=5
l/h=∞
1,0
0,8
0,6
0,4
0,2
0,0
m/h

FIG.2

The elementary increase dtv applied in an elementary strip dx is equal to the total horizontal force suspended in the strip (f.dx), distributed in the length where the vertical reinforcement is effective (l - h + x), in the corresponding x coordinate:

$$dtv = \frac{f\,dx}{l-h+x} \tag{2}$$

$$tmax = f + \int_0^h \frac{f.dx}{l-h+x} = f+f[\ln\,(x+l-h)] =$$

$$= f+f\,[\ln\,(h+l-h) - \ln\,(l-h)] = f+f\,[\ln\,(l)-\ln\,(l-h)]$$

$$tmax = f\,[1+\ln \frac{l}{l-h}] \tag{3}$$

It can be similarly shown that the maximum tension force in the base, varies in the right side of the wall ($0<y<h$) according to the formula:

$$t = f\,[1+\ln \frac{l}{l-h+y}] \tag{4}$$

The design with this approach can be helped using graphs, like the one presented in the Fig. 2. This figure displays graphically Eq.(4) in this adimensional form ($m=h-y$):

$$\frac{t}{f} = [1+\ln \frac{l/h}{l/h-m/h}] \tag{5}$$

The numerical example given in the following shows the application of the presented methodology.

4. NUMERICAL EXAMPLE

A typical NPP shear wall without flanges is shown in Fig. 3. An uniformly distributed horizontal force of 600 kN/m is applied in the top floor, from the left to the right.

The development of the maximum vertical tensile forces in the base of the wall, according to Eq. 4, is shown in Fig. 4. This figure shows also average tensile forces (t_1 to t_5), to be used in the design. These average forces and the corresponding necessary reinforcements (with fyd = 500 MPa) are determined below:

t_1 = 1566 kN/m; As_1 = 31.3 cm^2/m

t_2 = 1320 kN/m; As_2 = 26.4 cm^2/m

t_3 = 1014 kN/m; As_3 = 20.3 cm^2/m

t_4 = 798 kN/m; As_4 = 16.0 cm^2/m

t_5 = 660 kN/m; As_5 = 13.2 cm^2/m.

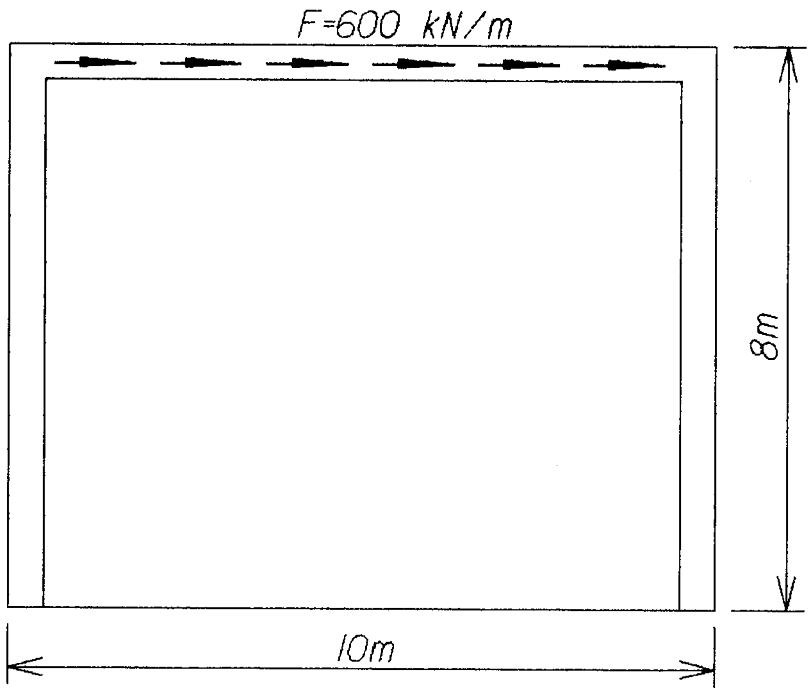

Fig.3 - SHEAR WALL I - ELEVATION

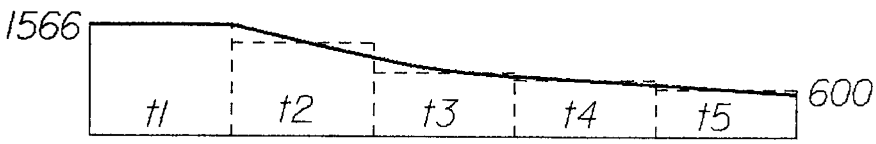

Fig.4 - AVERAGE VERTICAL FORCES (kn)

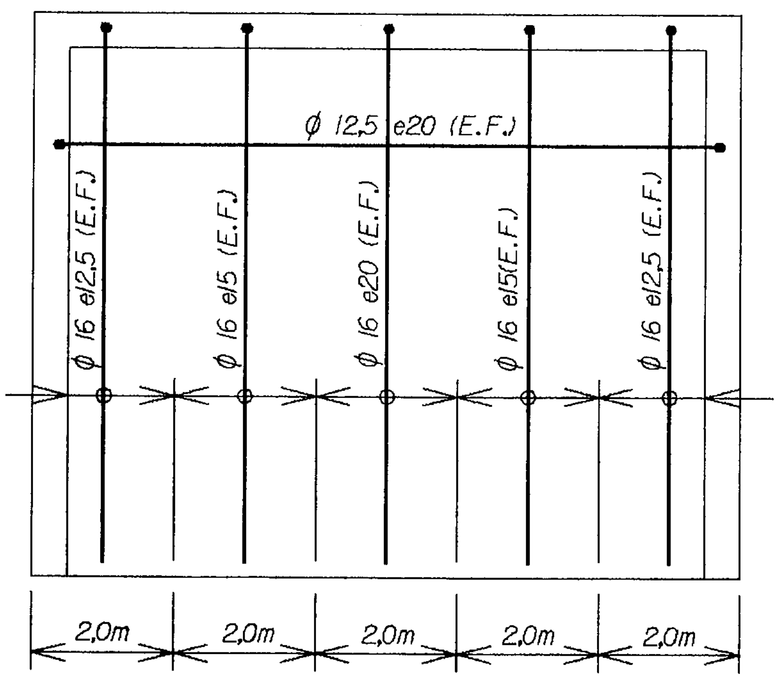

Fig.5 - SHEAR WALL I - REINFORCEMENT

Determination of the horizontal reinforcement:

th = f = 600 kN/m; Ash = 12.0 cm^2/m

Fig. 5 shows the final adopted vertical and horizontal reinforcements for the wall. It is to be noticed that the adopted vertical reinforcement is symmetric, considering that the horizontal force can change its direction.

Maximum compressive stress in the wall (thickness = 0.20 m):

$$\sigma c = \frac{1566}{0.20} = 7830 \text{ kPa}$$

Allowable concrete compressive stress according to CEB-FIP Model Code (1991), under cracked conditions (fck = 20 MPa):

$$fcd_2 = 0.60\ [1-\frac{fck}{250}]\ fcd = 11 \text{ MPa} > \sigma c \text{ (OK)}$$

5. CONCLUSIONS

A new approach was presented for analyzing squat shear walls using a generalization of the strut-and-tie resistant models. The proposed methodology was illustrated through a simple numerical example. The proposed idea can be generalized, in future research, for much more general situations, like slender shear walls and shear walls with flanges.

REFERENCES

Comité Euro-International du Béton (1991). CEB-FIP Model Code 1990, Final Draft. Bulletin d´Information nº 204. Paris.

Santos, S.H.C. and Silva, R.E. (1989). Seismic Analysis and Design of NPP Structures: An Integrated Approach. 10th SMIRT, Anaheim, USA.

Santos, S.H.C. and Silva, R.E. (1990). Design of Nuclear Structures Under Seismic Loading Using Strut-and-Tie Resistant Models. Fourth U.S.National Conference on Earthquake Engineering. Palm Springs.

Santos, S.H.C. and Silva, R.E. (1991). Design of Shear Walls Under Seismic Loading Using Strut-and-Tie Resistant Models. 11th SMIRT. Tokio, Japan.

Santos, S.H.C. (1992). Aplicação dos Modelos de Biela-Tirante ao Projeto de Estruturas Sismo-Resistentes. Ph.D Thesis. Rio de Janeiro: COPPE/UFRJ. (In Portuguese).

Schlaich, J., Schäfer, K. and Jennewein, M. (1987). Toward a Consistent Design of Structural Concrete. PCI Journal, Vol. 32, nº 3 (May/June): pp. 74-150.