ESTIMATION OF FRAMEWORK - INFILL WALL INTERACTION FOR MULTISTOREY BUILDINGS SUBJECTED TO CODE SEISMIC LOADINGS

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ABSTRACT

This paper presents the main results of an extensive analytical and experimental research program performed with the objective of elaborating a method and a computing program for the analysis of framed structure buildings in interaction with infill walls under conventional seismic actions.The algorithm emphasized the characteristic working moments of the building as long as the lateral seismic loadings are increased and takes into consideration the wall stiffness variation along building height in terms of the seismic shear force distributed to the walls.

1. EXPERIMENTAL PROGRAM

The significant influence of interaction between framework and infill walls on seismic response of buildings was emphasized by the last earthquakes,including that occured in Romania during 1977.Consequently an analytical and experimental research program regarding the multistorey framed buildings was undertaken within the Polytechnical Institute of Iassy.

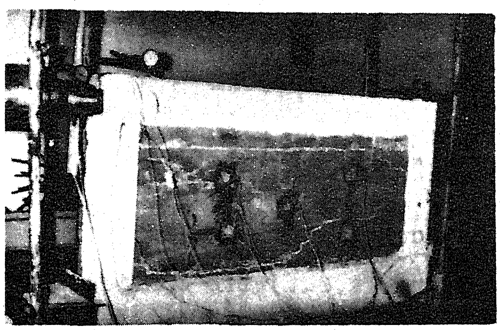
The general program included the following phases:

1. Tests on wall models bounded with reinforced concrete columns and girders under reversed static loadings, Fig.1;

2. Tests on scale models of actual structures on seismic shaking tables,Fig.2;

3. Establishing the analytical models corresponding to the parameters determined experimentally on reduced scale models;

4. Establishing the analytical models corresponding to actual structures:



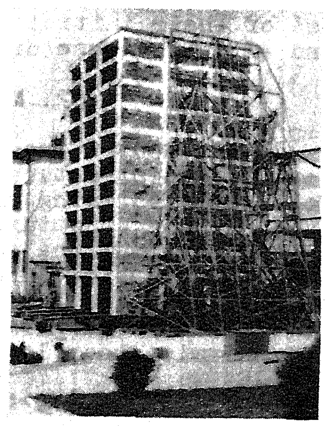


Fig.1,2

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The main results of the experimental program are:

1. The existence of the main differences among the deformation characteristics of materials from which the structure and the infill walls are made up;

2. The pronounced non-linear character of the load displacement behaviour curves of infill element.The infill of brick or lightweight concrete masonry have a significant deformability pattern even at relatively small loadings and this is why they are named "soft" panels;

3. The lateral stiffness of an infill panel Kw,from a certain level,decreased non-linearly with the increase of the seismic shear force,Hw,at the building level under consideration.One can notice that in the terminal part of the curve,the stiffness decrease is almoust straight;

4. The following behavioural characteristic moments during the seismic action are pointed out:

a. The structure and the infill panels behave within the linear-elastic range with a full coöperation between the two systems.

b. the structure continues to behave elastically,then the first concrete cracking takes place into the most stressed panel,this moment indicating the initiation of pronounced non-linear behaviour for the whole system of infill panels;

c. The structure is just before formation of the first plastic hinge,the panels being in various cracking stages, depending on their corresponding shear force.

The design of buildings under conventional seismic actions should take into consideration the structural behaviour from stages b to c,which implies that framework behaves in elastic range,while the infill walls are into non-linear range posessing a reduced stiffness in comparison with stage a due to cracking process and to detachment between panel and adjacent (perimetral) bars.This decrease in wall stiffness may be imposed by designer in accordance with the type and intensity of lateral actions (wind,earthquake) based on experimental data or conventional behaviour curves.For brick or lightweight concrete masonry walls it was found experimentaly that the most reduced stiffness was 10% of their initial elastic stiffness for the whole walls and 4% in the case of walls with openings.

2. METHODOLOGY AND ALGORITHM

A computer program was drown up by adapting an existent framed structural program which enables to perform the following designing phases:

1. The structure analysis to the static gravitational actions without the consideration of infill walls;

2. The structure analysis to the code seismic loadings without co-operation of infill walls,but with consideration of their afferent mass.The maximal values of stresses and the seismic shear load withstood by framework,are thus obtained;

3. The analysis of stresses into the framework and the walls in linear-elastic stage (stage a).In this stage the amount of total seismic load withstood by walls reaches

its maximal value. The infill wall are considered by a diagonal strut (2). The sectional area of the equivalent diagonal strut has the expression:

$$A_{d,el} = \frac{\gamma}{1,2} \cdot \frac{\beta}{\cos^3\alpha} \cdot a \cdot t \qquad (1)$$

in which:

a,b and t - wall dimensions

$\beta = a/b$

1,2 - shape coefficient

$\gamma = G_w/E_w$ - the ratio of transversal and longitudinal Young's modulus for the wall material

α- the angle between diagonal strut and the lenght of panel.

4. The limit bearing capacity of walls is determined by:

$$H_{w,lim} = c \cdot c_{op} \cdot a \cdot t \cdot R_{sh} \qquad (2)$$

and the limit axial stress in the equivalent strut:

$$N_{d,lim} = H_{w,lim} / \cos\alpha \qquad (3)$$

in which:

c - experimental coefficient, depending upon β and on brick to brick friction coefficient (1),(2),(3).

c_{op} - experimental coefficient considering the influence of wall openings and depending to the ratio a_{op}/b_{op}, Fig.3, (1),(3).

R_{sh} - design shear strength of masonry through continue gaps.

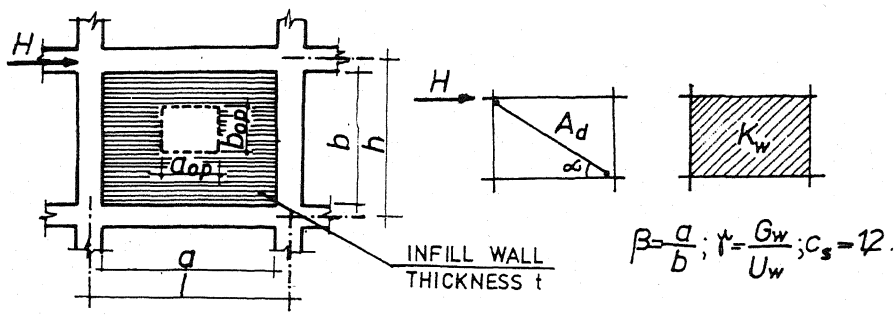

Fig.3

5. Limit axial stress is compared, relation (3), with the stress corresponding to the equivalent strut. In case that $N_{d,lim}$ is smaller, the analysis is repeated without taking into consideration the corresponding walls;

6. In case of the most stressed wall, or the strut with the highest axial stress, a minimum stiffness or a minimum sectional area is required, namely:

$$A_{d,lim} = c_s \cdot A_{d,el} \qquad (4)$$

in which c_s is o,1 for the whole walls and o,o4 for the walls with openings (according to chapter 1,point c). The sectional areas of equivalent struts along the building height will be greater than minimal sectional area,in an inverse proportional ratio with their corresponding axial stress.For a certain wall,i,it may be written:

$$A_{di} = A_{d,lim}\left(\frac{N_{d,lim}}{N_d} \right)_i / \left(\frac{N_{d,lim}}{N_d} \right)_{min} \quad (5)$$

The relation (5) is accepted in the virtue of observation made at chapter 1,point 3;

7. The first distribution of wall stiffness by equivalent strut areas along the building height corresponds to the seismic shear force distribution for initial elastic stage.For this reason the reanalysis of the structure become necessarily up to a point when the reduced area distribution is in aggreement with the axial stress distribution in equivalent struts.It was found that 3 to 4 succesive iterations are enough for design purposes;

8. Within the structure computation model to seismic actions,the interaction effect is considered by means of the equivalent strut reduced to its own axis.Indeed this "embending" effect is evident upon a relatively large width on both sides of strut axis so that the checking of shear forces is required in girder and column sections just in vecinity of joints.

3. CONCLUSIONS

The program was applied to the analysis of large number of multistorey buildings.The favorable effects of the framework - wall interaction was emphasized by a reduction of framework stress due to seismic action up to 3o per cent, with the mention that the stress reduction is not uniform, being more significant into the bounding bars of walls.

A large non- symmetrical disposition of infill walls in the structure,increases the effects of general torsion. There are also cases specialy in the initial stages of the interaction and to the upper building levels when the stresses in some bars with wall interaction,were greater than the stresses determined without considering the cooperation of infillings.

REFERENCES

1. Ciongradi C., Contributions regarding the interaction between the framework and infill walls in buildings subjected to seismic loads,Thesis,Iassy,1979.

2. Poliakov S.V., Kamenaia kladka v karkasnîh zdaniah, TNIISK,Moskva,1956.

3. Sachanski S.,Brankov G., Influence of the Nonelastic Deformations of the Structures on the changing of Dynamic Characteristics and Response,5th.WCEE,Roma,1973.