SEISMIC DESIGN OF STRUCTURAL WALL BUILDINGS AND INELASTIC RESPONSE ANALYSIS

Boris Simeonov[I)]

ABSTRACT

The process of designing of buildings with reinforced concrete walls is shown on a nine storey building constructed in Skopje. The structural walls with openings are proportioned according to the method of allowable stress. The capacity of strength, stiffness and ductility is verified through the inelastic response analysis to actual earthquakes. The results of these analyses point to the necessity of the experimental investigation of the members and assemblages of the reinforced concrete walls and specially the necessity of high shear forces.

INTRODUCTION

In the course of the last few years, a great number of residential buildings with bearing reinforced concrete walls have been built in Skopje. The height of these buildings is from three to nineteen storeys and the thickness of the walls is from 15 to 25 cm. The designing has been carried out taking account of a seismic zone of nine degrees MCS and dynamic analysis of response to two types of earthquakes with maximum expected acceleration of 0.27 g has been carried out for buildings higher than those of eight storeys. The accelerograms of the earthquakes in El Centro and Parkfield have been used for the inelastic response analysis.

The process of the designing and the characteristics of the building with bearing walls will be shown in a nine storey building with symmetric distribution of the walls in the plane.

The necessary dynamic response analysis for $a_{max} = 0.34$ g has been carried out for the experimental investigation of three-storey models of structural walls.

STRUCTURAL SYSTEM AND DESIGN CRITERIA

The typical distribution of the walls is shown in (Fig.1) where not all of the front walls can be found. This is conditioned by technology of building by tunnel formwork. There are openings for doors and windows on the reinforced concrete walls so that equivalent frames with stiff zones are formed. The thickness of all the walls is 15 cm along the whole height.

The elastic analysis of the structural system has been carried out by using dead and life and seismic loadings according to the computer programme TABS(1). The seismic forces were defined on the basis of the nonlinear response of the system to the accelerograms of the El Centro and Parkfield earthquakes. The analysis has been repeated several times so that a satisfactory answer regarding the maximum displacements and ductility has been obtained. In this way, an optimal distribution of the seismic forces along the height of the building has been obtained. The seismic coefficient for this building is Cb=0.15.

The structural members (walls and coupling beams) were proportioned by applying the method of allowable stress. The characteristic cross sections of the walls and beams are shown in (Fig.2). At each end of the walls, the reinforcement is in the closed stirups, while at the web of the wall there are usually two fabric welded mashes. The main vertical reinforcement has a yield strength of 240 MPa. It is 0.25% from the whole cross section of the wall while the mashes contain equal procentage of vertical and horisontal reinforcement of 0.57% and a yield strength of 500 MPa.

The coupling beams in the lower floors of the building have symmetric upper and lower reinforcement, 0.77% of the concrete cross section and the vertical stirups, 0,38% of the horisontal cross section. The designed strength of the concrete is 20 MPa.

Only the reinforcement bears the shear force in the lower floors. It is assumed that the concrete can not carry shear forces because of the possibility of formation of plastic hinges in the lower floors of the building during an earthquake.

I) Associate Professor, Institute of Earthquake Engineering and Engineering Seismology (IZIIS), University "Kiril and Metodij", Skopje

INELASTIC ANALYSIS OF THE BUILDING

An inelastic response analysis of the building to the Parkfield earthquake in 1966, component NS, for the x-building direction, has been carried out according to the computer programme DRAIN-2D(2). The aim of this analysis is the estimation of the worst loading for some structural members.

In x-direction, the building has two identical walls with openings. So, the analysis can be performed by taking occount of only one wall taken as a plane structure with certain mass concentrated in the nodes of the frame with rigid zones. The plastic hinges can be formed on both ends of the columns and beams during the dynamic effect of the earthquake. The bearing capacities of the columns and beams are calculated by the computer programme and are given as interaction diagrams M-N.

The viscous damping is 4% of critical. Damping proportional to mass and stiffness has been calculated on the basis of the periods of two vibration modes.

The following values are to be considered as a dynamic response of the wall with openings to the time history of the earthquake.

- horisontal displacement in the nodes
- rotation of the nodes
- member forces
- plastic rotations at each end of the members

These values are obtained for every 0.02 sec of the modified duration of the earthquake. The maximum values of member forces as well as the plastic rotations within all members have been obtained at the end of the analysis.

The results of the inelastic analysis have shown that plastic hinges are formed at each end of the beams and at the base of the walls. This is acceptable mechanism for walls with openings (Fig.3).

The maximum plastic rotations at each end of the beams are 0.00423 and at the base of the walls 0.00012 for maximum ground acceleration of 0.27 g. If the ground acceleration is 0.34 g, the rotations will be 0.00753 and 0.00075 respectively. The ability of the members to perform plastic rotations will be verified by experiments on models 1:3. These experiments will be carried out by IZIIS. In case of adequate reinforcement of the plastic zones, the required plastic rotations are expected not to exceed their capacities.

The relationship between the base moment and shear base for the external and internal member is shown in (Fig. 4). The sequence of simultaneous appearance of these values is shown numerically. The internal member-wall is designed considering the moment M=2080 kNm and shear force Q=824 kN. Thus, the ratio M/Q=2.52 is obtained. This ratio is almost the most critical as it can be seen from (Fig. 4a), but the high values of the shear force during the inelastic response point to the possibility of premature destruction of the member due to shear forces. The maximun shear force in the internal member is Qmax=1820 kN for the time t=0.78 sec. The value of the moment for the same time is M=6570 kNm and the value of the axial force is N=2325 kN. The external member-wall is designed considering the moment M=5530 kNm and shear force Q=1150 kN. Thus, ratio M/Q=481 is obtained. The maximum shear force during the inelastic response to the earthquake in Parkfield 1966 (0.34 g) is Qmax=2970 kN, for time t=1.22 sec. The value of the moment for the same time is M=16870 kNm and the axial force is N=7240 kN.

The banding capacity of the walls is sufficient for the moments of the response. The capacity of the shear force is 980 kN for the internal member-wall and 3650 kN for the external member. In both cases the contribution of reinforcement in the web and flanges is taken in account, while it is not the same with the concrete. There is possibility of destruction of the wall with rectangular cross section which can be caused by shear forces. Beside this, there is also possibility of local instability of this member.

REFERENCES

1. Wilson, E. L., and Dovey, H. H., "Three Dimensional Analysis of Building Systems", Report No. EERC 72-8, University of California, Berkeley, California.
2. Kanaan, A. E., and Powell, G. H., "General Purpose Computer Programme for Inelastic Response of Plane Structures", Report No. EERC 73-6, University of California, Berkeley, California.

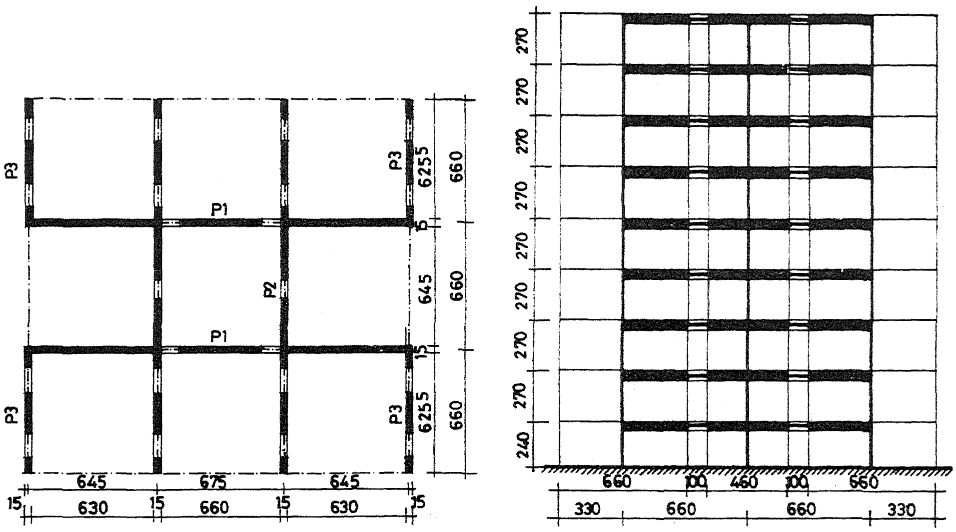

Fig. 1 Typical floor plan and Elevation

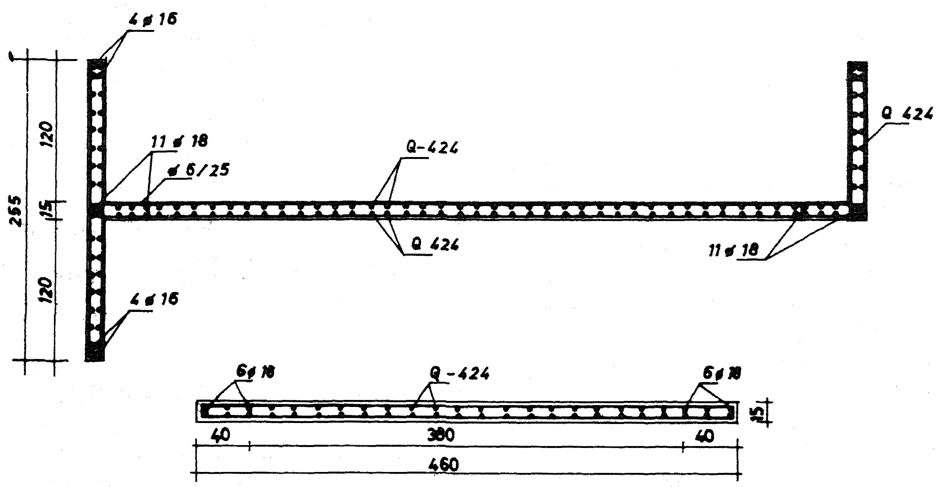

Fig. 2 Cross sections of walls

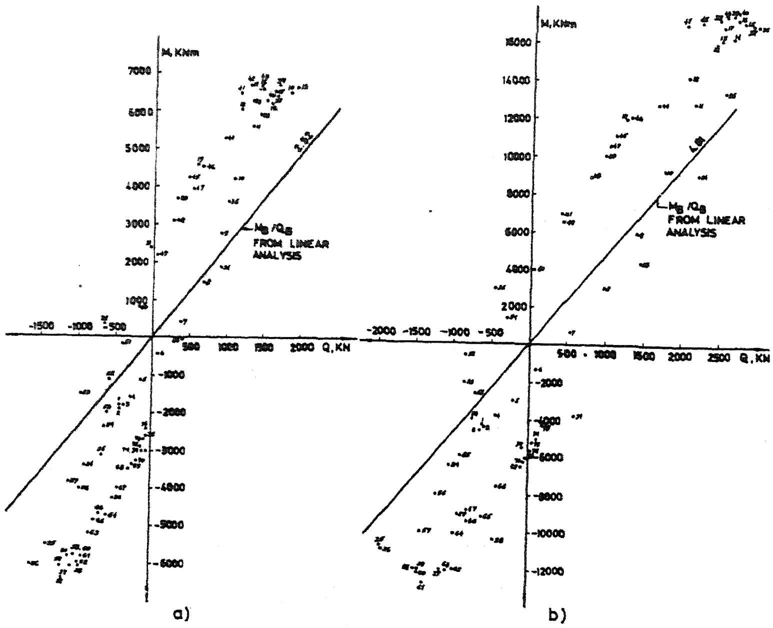

Fig. 4 Relationship between M_B and Q_B under Parkfield Earthquake (0,34g)

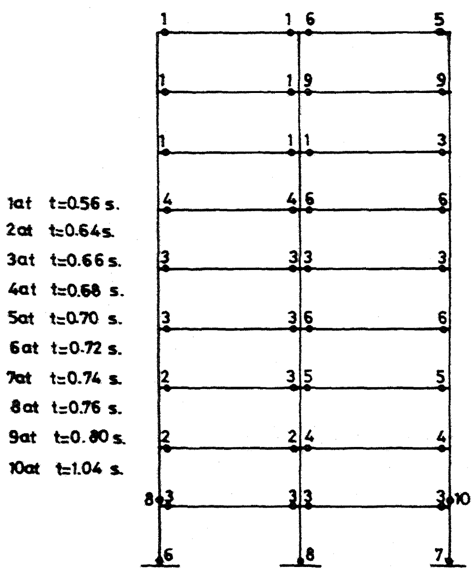

Fig. 3 Location of plastic hinges

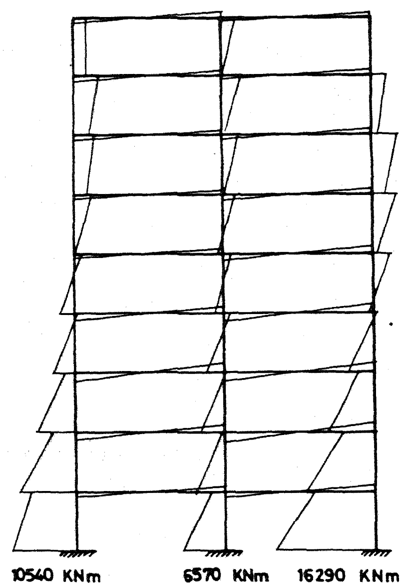

Fig. 5 Moment diagram from inelastic response analysis, t = 0,78 sec.