PREDICTION OF THE TORSIONAL RESPONSE OF A MULTI-STORY REINFORCED CONCRETE MASONRY BUILDING BY A THREE DIMENSIONAL DYNAMIC ANALYSES

by

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SYNOPSIS

Because of its economy, reinforced concrete masonry is being used in the construction of multi-story buildings in many areas of the United States. These structures have relatively high natural frequencies and, in the absence of a detailed dynamic analysis, present potentially serious problems in seismically active regions[1,2,3]. This paper presents the results of a three-dimensional finite element analysis of a thirteen story building recently designed and constructed of concrete masonry, using a configuration that indicates considerable torsional response would result if strong earthquake ground motions were experienced. The study demonstrates the inherent threat to public safety through construction of such structures without a dynamic analysis and, the economic feasibility and practicality of providing such analyses with large scale digital computing systems.

INTRODUCTION

There are four primary reasons for concern with present practice in the design of multi-story concrete masonry buildings which can be enumerated as follows: First, these structures are basically brittle. In contrast to ductile structures where a trade-off can be made between yield strength and energy absorption capacity, ultimate strength must be the primary design criterion. Little margin for analysis errors exists for such structures since significant energy absorption in the inelastic range can take place only by brittle fracture. Catastrophic failures may result as the walls must resist both shear and bearing stresses. Second, the usual alternative to a detailed dynamic analysis is to provide the minimum design requirements of the Uniform Building Code. Substantial analysis errors may consequently result since the latter is known to significantly underestimate the seismic response of brittle structures having high natural frequencies[4]. Third, innovative architectural features may result in complex dynamic response and consequent structural overstress not predictable by a static analysis, which is illustrated in this paper. Fourth, the problem is compounded by the fact that the number and quality of material tests on concrete masonry are insufficient to accurately define material failure under the cyclic stresses produced by seismic loads.

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ANALYSIS OF A TYPICAL MULTI-STORY BUILDING

The thirteen story building shown in Figure 1, having a length of 154 feet, width of 64 feet and height of 124 feet was selected for the study. All walls with minor exception were constructed of 8 in. reinforced concrete masonry, generally fully grouted up to the ninth floor, and grouted at intervals of 48 in. from the ninth through the thirteenth floors. The walls were supported upon grade beams which were supported upon concrete piles. Floor slabs at the first floor level were poured on grade. Precast, pretensioned, hollow concrete plank, 30-1/2 feet by 4 feet by 8 in. were used for all other floors. Joints between the planks were grouted but no reinforcing steel was provided across the joint except in corridors. Planks have a minimum bearing width of 2-1/2 in. at the wall line. For architectural reasons, the longitudinal shear wall was omitted on the west side of the building.

A search was made of the technical literature to establish a failure criterion for concrete masonry walls subjected to combined seismic and dead loads. No test results directly applicable to these combined stresses could be found. Therefore, the criterion had to be based largely on an interpretation of test results for statically loaded isolated wall panels of single-story construction not subjected to high bearing loads. These static tests indicate that a failure criterion for the mortar joint in shear might be based upon the coulomb expression $S_{xy}(av) = 100 + 2S_y$ where $S_{xy}(av)$ is the average shearing stress on the mortar joint in psi and S_y is the average normal compressive stress on the joint, also in psi.

The structure is located in close proximity to a fault for which a maximum earthquake potential of magnitude 6.5 has been associated, and approximately 60 km from major faults on which earthquakes of magnitudes of 7.5, or greater, have been experienced. The former could produce peak horizontal ground accelerations of 0.3 to 0.5 g at the construction site, while the latter could yield 0.3 g or greater. The scaled (1.5) Glendale (1971) and El Centro (1940) earthquake ground motions were considered typical of the above conditions, and the three components from each record were used as base input in separate analyses.

The structure was modeled with 498 nodal points consisting of 726 plate elements and 81 beam elements. The three dimensional dynamic elastic GENSAP code was used for the first analysis. In order to demonstrate the feasibility of using large digital computers for dynamic analyses, the problem was also run on four different computing systems to provide a comparison of computing time for the different systems.

ANALYSIS RESULTS

Examination of the analysis results indicates that the first three modes of response are torsional modes, having centers of rotation outside the perimeter of the structure (Figure 2). The first three natural frequencies were found to be 2.64, 3.09, and 4.50 cps, which indicate that this is a stiff structure and that the peak ground accelerations will be greatly amplified, producing high structural stresses. A summary of some of the critical element stresses for the south wall (shown in Figure 1) is given in Table 1 in the following order: scaled Glendale at t = 1.6 secs., and El Centro at t = 2.6 secs., respectively. The mesh arrangement for the south wall is shown in Figure 3. An examination of the design and the stress levels in various elements indicates that the structure is lightly reinforced and does not have sufficient vertical steel to resist the tensile stresses computed, nor sufficient horizontal steel to control vertical cracks in the walls.

Computational times for the four computing systems were found to be as follows: 10,100 seconds on Burroughs 6700, 4,020 seconds on UNIVAC 1108, 771 seconds, CDC 6600, and 131 seconds on CDC 7600. Thus, it is evident that the cost of computing time on the larger systems is not prohibitive and even three-dimensional dynamic analyses are economically feasible.

RECOMMENDATIONS

The result of this study indicates that the building analyzed, even though it satisfies the Uniform Building Code, will be severely damaged if subjected to the ground motions considered. The following recommendations result from this study:

(1) If reinforced concrete masonry is to be used for combined shear and bearing wall construction of multi-story buildings having a high factor of human use, or to house important facilities, an extensive program of experimental and analytical investigation should be provided to define the behavior of these structures under repeated cycles of dynamic loading. At present a reliable failure or design criteria is not available to the engineering profession for such structures. The criteria used in this study to define failure may ultimately be proven unconservative.

(2) Building codes should be revised to insure that new construction of this type will meet adequate standards in order to prevent the loss of life and property. The seismic risk should be related to the geology and seismicity of the local area. Either a two or three dimensional dynamic computer analysis, dictated by the nature and type of construction, with adequate ground motions should be required. The results should be reviewed by competent engineers in seismic design before construction is authorized.

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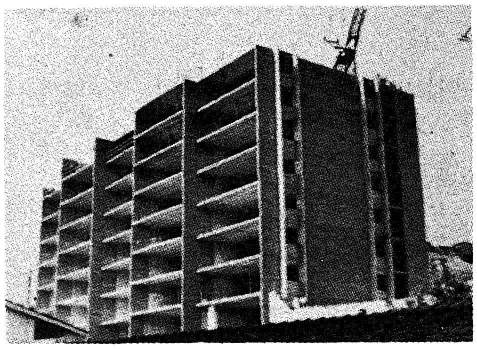

FIGURE 1. REINFORCED CONCRETE MASONRY BUILDING

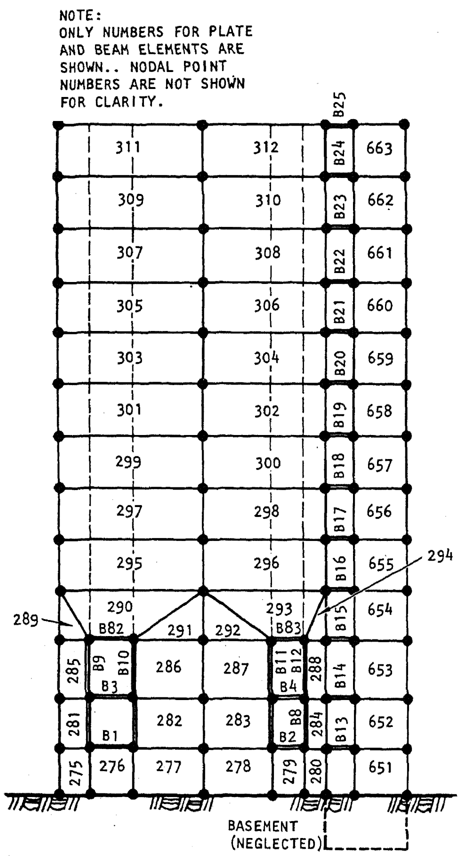

FIGURE 3. MESH ARRANGEMENT FOR SOUTH WALL (WALL 1)

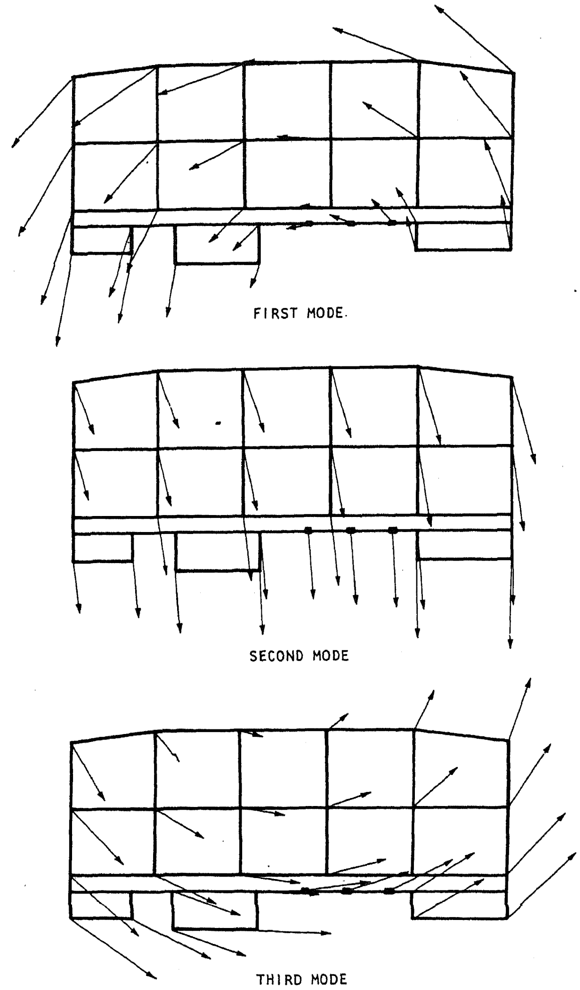

FIGURE 2. VECTOR DISPLACEMENTS OF ROOF NODAL POINTS

TABLE 1. SUMMARY OF ELEMENT STRESSES (PSI)

Element	Computed Average Stresses			S_{xy} Vertical Joints	Remarks
	σ_x	σ_y	τ_{xy}		
275	+28	+423	+ 49	44	1, 4
	+16	+350	+ 32	68	1
276	+58	+281	+141	-	1,2,3,4
	+45	+226	+ 95	10	1, 4
277	+29	+128	+ 22	42	1
	+18	+ 82	+ 64	64	1, 4
278	-31	-154	+101	162	4(a)
	-32	-163	+ 68	164	-
279	-30	-261	+173	160	4
	-52	-271	+120	204	-

1 and 2 denotes horizontal joint failure in tension and shear, respectively. 3 and 4 denotes vertical joint failure in tension and shear, respectively. (a) denotes vertical joint failure in shear after horizontal cracks develop in Elements 275, 276 and 277.