TIME-HISTORY RESPONSE OF BUILDINGS WITH UNUSUAL CONFIGURATIONS

by

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SYNOPSIS

Many buildings are of such size or slenderness, or have such unusual configurations, that they are different than the idealized models used in earthquake research and as a basis for code development. Moreover, many structures such as those in nuclear power plants present great risks in the event of earthquake failure. The building-code design approach in such cases is inapplicable and inadequate. However, it is feasible in today's state of experience and knowledge, and with the aid of the large capacity computer, to compute the response of any building configuration to the time-history of real or artificially generated earthquake motions. Data from such analyses are presented for school buildings, multistory setback buildings, and a nuclear power plant building.

GLOSSARY OF TERMS

C_B = base shear coefficient

C_T = tower-base shear coefficient

D = plan dimension of the building, L

E = dynamic modulus of elasticity, FL^{-2}

G = dynamic shear modulus, FL^{-2}

H = height of building, L

K_g = lateral foundation shear spring stiffness, FL^{-1}

K_s = vertical foundation spring stiffness, FL^{-3}

K_{ss} = lateral foundation side spring stiffness, FL^{-1}

L = longitudinal plan dimension of diaphragm, L

N = number of stories in building

P = story above which setback occurs

T_i = i^{th} mode natural period, T

V, V_B = base shear, F

V_T = tower-base shear, F

W = weight of entire building, F

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W_T	=	weight of tower, F
Y	=	transverse plan dimension of diaphragm
c	=	degree of setback, dimensionless
kx_i	=	stiffness in the i^{th} story, FL^{-1}
kx'_n	=	a constant stiffness, FL^{-1}
Δkx	=	incremental stiffness, FL^{-1}
m	=	a constant mass, $FL^{-1}T^2$
m_i	=	value of the i^{th} lumped mass, $FL^{-1}T^2$
p	=	P/N, level of setback, dimensionless
y_R, y_W, θ_W	=	modal deformations as shown in Figure 2

Superscripts

M = meter

T = ton, metric

Units

F = force

L = length

T = time

INTRODUCTION

There are many buildings with external configurations that are quite different from idealized rectangular shapes in plan and in elevation. Examples would include L or U-shaped floor plans, and buildings with setbacks in one or both elevations -- perhaps symmetrical and perhaps not symmetrical about the vertical axis. Many buildings have roof level appendages in the form of penthouses or towers. There are also rectangular shapes in plan or elevation that are so slender that traditional design concepts and normal code requirements may not be appropriate as a sole basis for design. Industrial installations such as nuclear power plant buildings may depart radically from the ordinary concept of symmetrical buildings with typical floor plans, and in addition may present such risks that ordinary design approaches to earthquake resistance are wholly inapplicable and inadequate. Unusual geometric shapes may have no resemblance to the shapes of buildings considered in research efforts and as a guide for earthquake code requirements.

Although it is desirable to avoid asymmetric shapes when possible, especially if these lead to coupled modes, this can not always be done. It is essential in such cases -- and in fact for all unusual shapes or configurations and all important structures from the risk viewpoint -- to employ dynamic analysis rather than depend upon code methods that may not treat the problem properly or realistically. Theory, with computer aid for labor and time saving, is adequate today to analyze models of any structure -- no matter how odd or complex -- for any real or designed ground motion, no matter how complex. The problem, however, is twofold-- to create a representative model of the structure, and to keep the overall operation on a practicable level of time, cost and refinement. These matters are not easily resolved, and require a proper balance of advanced theory, computer technology and a sound knowledge of real structures and of dynamics, together with sound engineering judgment. Results of the analyses of some structures with unusual configurations will be presented to illustrate the response of such structures. Detailed procedures are beyond the scope of this paper.

A SLENDER SCHOOL BUILDING

Even one-story buildings can present unusual configurations and problems in earthquake analysis. School buildings tend to be long and narrow. If permanent transverse walls are used between classrooms, or other rooms, the roof diaphragm spans are short and the diaphragm span-to-depth ratios are small. Such roof systems are generally strong and rigid and have short natural periods of vibration. (1)(2)(3) In such cases, the popular design assumption that the diaphragms are infinitely rigid in their own planes may be satisfactory. However, there is a tendency toward movable partitions rather than permanent walls, and also toward open spaces and a great deal of glass in the side walls. In some cases, interior partitions even though not movable, may be non-structural and may not be connected to the diaphragm system. The result is a structural system that may be termed an open staple, or a flat upside-down U consisting -- for lateral resistance -- of two end walls connected by a long, slender roof diaphragm. The small columns or posts in the side walls carry vertical loads but usually offer essentially no resistance to lateral forces. The diaphragms may have span-to-depth ratios of 3, 4 or more,and this system may have dangerous response to earthquake motion.

The top story of one such building designed to seismic standards suffered end wall damage in the 1952 Kern County earthquake. (4) Although some poor workmanship was blamed for the damage, dynamic phenomena -- not covered by the building codes -- seems to have been an even greater factor. Four models of this type of structure were subjected to three different earthquake excitations.

Idealized Models. Figure 1 illustrates idealized models of one-story buildings, or of the top stories of 2 or 3-story school buildings. The columns between the end walls are assumed to offer no lateral resistance and are therefore not shown. However, the diaphragms have adequate vertical support. The two end walls provide the entire vertical plane transverse resistance. Model No. 1 has the same roof dimensions as the school building that suffered end wall damage in the 1952 earthquake. (4) The roof weights in kips are 742, 558, 371 and 186 for Models 1 to 4 respectively. The following properties are common to all four models:

End wall gross area	3130 in^2
" " effective shear area	2610 in^2
" " moment of inertia	$31.5 \times 10^6 in^4$
" " polar moment of inertia	$31.52 \times 10^6 in^4$

Diaphragm gross area	1970 in^2
" effective shear area	1640 in^2
" moment of inertia	$66.2 \times 10^6 in^4$
Tributary weight of each end wall	34.5 kips
Modulus of elasticity, wall and roof	2×10^3 kip/in^2

The models were assumed to have full continuity between the walls and the roof system, and the base of each wall was assumed to be fixed. No other restraints were imposed. Shear, flexure and torsion were considered in the analysis. There must be geometric compatibility between the end rotation of the diaphragm in lateral flexure and the torsional rotation at the top of each wall. It is to be noted that the gross translation of the diaphragm is a result of its translational shear and flexure, torsional freedom in the end walls, and shear and flexure in the walls. Earthquake motion was applied parallel to the end walls. Figure 2 shows the fundamental mode shape and the lumped-mass model. This composite system was analyzed as a series of lumped masses for natural periods of vibration and mode shapes and was then subjected to the recorded earthquake acceleration time history of three earthquakes. Damping was taken at 5% of critical and the response was assumed to be elastic.

Table I shows the natural periods of vibration and the ratio of the translation of the midspan of the roof to the translation of the top of an end wall in the first mode. For Model No. 1, the maximum deformation in the first mode (at the center of the roof) is 14.7 times the translational deformation of the wall tops. This indicates that the diaphragm, often assumed to be rigid in design, is actually a dominant part of the dynamic system. The 0.175 sec fundamental mode period for this model is less than the measured period of 0.25 sec for the school building referred to above. The second mode period of 0.084 sec compares to a measured period of 0.10 sec. The differences are due to two basic factors -- the real building has two stories rather than one and therefore more wall deformation, and the assumed modulus of elasticity and isotropic material behavior for the model are no doubt in error. Moreover, the long roof overhang may not be fully effective as assumed for the model.

Earthquake Response. All four models, with assumed damping at 5% of critical, were subjected to the recorded (digitized) earthquake motion of three earthquakes: the first 15 seconds of Taft 1952, N69°W component; the first 5 seconds of El Centro 1940, N-S component; the first 6 seconds of San Francisco 1957, Golden Gate Park record, S89°E component. It is believed that the portions of the records used account for most of the severe motion. However, slightly greater response would be expected if the entire records had been employed, especially for El Centro.

Figure 3 shows the elastic response acceleration for the lumped mass at the midspan of the diaphragm under the three different earthquakes and for the four models with diaphragm span ratios, L/Y, of 1.04, 2.11, 3.18 and 4.22. The tremendous increase in response acceleration with increasing diaphragm span is obvious for ratios greater than 2. Although normal code design coefficients might be adequate for span ratios of 1 or 2, they are inadequate for all three earthquakes in the L/Y range of 3 to 4 and more, unless much ductility can be mobilized. However, because a great deal of shear response is involved it is doubtful if much ductility is generally available. These midspan response accelerations are the maxima for the models; thus the midspan corresponds, in this regard, to the top of a tall building where maximum accelerations usually occur.

Figure 4 shows the base shear coefficients, applicable at the base of each end wall to all tributary weight above that point, to meet the earthquake demands. Torsional shear effects are not included in these coefficients. The El Centro values are all well above code coefficients, and increase rapidly above an L/Y ratio of 3. The Taft and San Francisco earthquake demands are considerably less than for El Centro.

Figure 5 shows the maximum unit shear stresses at the top of each end wall under the actual earthquake exposure, for the four diaphragm slenderness ratios. The total gross area of the wall was used. The effect of the rotation of the walls due to the end slopes of the diaphragm spans has been included in all cases, although shown separately only for the El Centro earthquake. Unit stresses in earthquakes obviously increase with the diaphragm span ratios.

It appears not only that real earthquake forces are greater than code requirements but also that normal static design procedures are inadequate for building configurations such as represented by Models 1 and 2 with diaphragm ratios in the order of 3 and 4, or greater. The use of interior structural walls or frames would reduce these ratios and materially decrease seismic risk in school buildings. If these walls or frames can not be provided for any reason, dynamic design is indicated or else improved code provisions are needed for buildings of this type and configuration. It is understood that based upon this study improved design practices and code provisions are in process for California school buildings.

SHEAR BUILDINGS WITH SETBACKS

A highrise building with one or more setbacks (Figure 6) is another configuration presenting special problems. American codes (5) (11) have special provisions for computing seismic design forces for such buildings. If the setback at a given level in a building is significant, one may expect stress concentration at the setback level when the building is subjected to ground motion. The codes try to deal with this problem by specifying the alternative of treating the tower portion of the building as a separate structure. The separate tower concept, however, does not take into account the fact that the ground motion is modified greatly by the base portion of the building before it affects the tower. The tower is subjected to an essentially harmonic forced vibration instead of the nearly random motion of the ground. Another problem may arise if the setback is asymmetric. The torsional and translational vibrations of a building with unsymmetric setback will, in general, be coupled. This may cause severe torsional vibrations in the building.

A study of the effect of setbacks, both symmetric and unsymmetric, on the response of tall buildings to earthquake ground motion has been made and presented elsewhere. (6) Some of the results obtained in that study for the case of buildings with symmetric setback are presented herein. The following assumptions are made in the analysis: the masses are lumped at floor levels; the stiffness is linear elastic; and the damping is linear viscous. Besides these usual assumptions, it is assumed that the stiffness of the building is of the close coupled type; i.e., the shear force acting in a story is independent of the deformations in other stories. This is a shear building with rigid floor systems. (7)

An N-story building with a setback above the P^{th} story is shown in Figure 6. The setback is symmetric about the x-axis, and it is assumed that the motion of the building along this axis is uncoupled from torsion or translation in the y-direction. The following distribution of the mass, m_i, and the story stiffness, kx_i, over the height of the building is assumed:

In the base portion $(1 \leq i \leq P)$:

$$\begin{aligned} m_i &= m \\ kx_i &= kx_n' + \Delta kx \, . \, (P - i + 1) + c . \Delta kx \, . \, (N - P + 1) \end{aligned} \quad (1)$$

In the tower portion $(P + 1 \leq i \leq N)$:

$$\begin{aligned} m_i &= cm \\ kx_i &= c . kx_n' + c . \Delta kx \, . \, (N - i) \end{aligned} \quad (2)$$

In equations (1) and (2) kx_n', Δkx and m are constants. These distributions are shown graphically in Figure 7. The masses have a constant value in each part of the structure: m_b in the base portion and m_t in the tower portion. Then from the above equations

$$c = m_t/m_b \, ; \quad (3)$$

c is termed the degree of setback. One may also define $p = P/N$ as the level of setback. These two quantities are then the parameters of the setback. The range of both parameters is given by $1 \geq c,p > 0$. However, it should be noted that p can take on only a few discrete values within this range.

In equations (1) and (2) kx_n' $(c.kx_n')$ is the constant part in the story stiffness and Δkx $(c.\Delta kx)$ is the increment in stiffness per story in the base (tower) portion of the structure. The degree of setback, c, is assumed to have roughly the same effect on story stiffness as on the mass. The ratio $\Delta kx/kx_n'$ was taken equal to 1/3; the ratio kx_n'/m was chosen equal to 580 sec^{-2} to obtain a period of approximately 1 second for a 10-story uniform (i.e., c = 1) building to satisfy the code formula $T = 0.1N$.

Results of the modal analysis and response computations for 15-story buildings with varying levels and degrees of setback having the idealized properties described above were obtained. Due to lack of space the equations of motion and the procedure for their solution will not be discussed here. Reference may be made to structural dynamic textbooks, e.g., reference (8). The effect of setback on the natural periods is shown in Figure 8. Note that the values $c = 1$, $p = 0$ and $p = 1$ each represent a uniform building. The fundamental period is seen to be decreasing with decreasing c for all levels of setback. The decrease seems to increase as the setback level is shifted from either end (i.e., $p = 0$ or 1) towards midheight ($p = 0.5$).

Time-histories of various responses of the same 15-story buildings to the N-S component of El Centro 1940 records and the N69°W component of Taft 1952 records were computed and, among other things, the maximum values of base shears, V_B , and shears at the base of tower, V_T , were obtained. Four modes were considered in the response computations, with damping values of 4% of critical damping in the first two modes and 6% in the third and fourth modes. Only the first 15 seconds of the ground motion records were used as input since computations for a few selected cases using the entire records showed all the maximum responses occurring well within the first 15 seconds.

The ratios of maximum base shear coefficient C_B to the maximum base shear coefficient of the uniform building, C_B ($c = 1$), are plotted in Figure 9. Similarly the ratios of the tower-base shear coefficients ($C_T = V_T W_T$, where W_T is the weight of the tower) to the shear coefficient at the corresponding level in the uniform building, C_T ($c = 1$), are plotted in Figure 10. For the purpose of comparison similar ratios computed from the values obtained from the code specifications (5) are also plotted in these figures. It should be noted that the reason for comparing the ratios of the shear coefficients, instead of their actual values, is to measure the relative effectiveness of the code provisions for buildings with setback. The actual values of the code shear coefficients are much smaller than the corresponding computed response values, as would be expected.

The striking feature of Figure 9 is the difference between the Taft and the El Centro curves. The El Centro responses indicate, in general, an increase in the base shear coefficient as the tower size decreases relative to the size of the base portion (i.e., as c decreases), for all levels of setback. The Taft responses indicate, in general, very little change and even a slight decrease. The code curve falls somewhere between the Taft and the El Centro curves. The effect of setback on the base shear coefficient is thus apparently strongly dependent on the characteristics of the ground motion which may be roughly represented by its response spectrum. These characteristics may be significantly influenced by the local geologic conditions. However, until such relations can be well established and incorporated into code provisions, the proper alternative seems to be more accurate computations of seismic design forces instead of a straightforward application of code specifications. More than one earthquake is often desirable in analysis to provide for various possible ground motions.

The Taft and El Centro curves in Figure 10 show a more mutually consistent effect of decreasing c on the tower-base shear coefficient, viz., they increase as the tower size decreases relative to the base size. A comparable effect is seen in the code values when the setback level is below the midheight ($p < 0.5$) of the building. However, when the setback is above the midheight level, the increase provided by the code specifications is significantly less than that observed in the computed response values. This situation can possibly be improved by introducing proper scale factors in the code formula for the computation of tower-base shear coefficients.

The setback building results reported herein were obtained by one of the authors (6) at the University of Michigan, Ann Arbor, partially under a research project sponsored by the National Science Foundation. The author gratefully acknowledges the guidance and encouragement provided by Professor G. V. Berg during the course of that work.

NUCLEAR POWER PLANT BUILDING

The type of structure represented by a reactor building is, generally speaking, not adequately covered in any building code. This type of building is often a massive reinforced concrete structure consisting of six or seven floors with a steel braced structure at the top. Such structures are built on a single concrete foundation mat and are so rigid that the effect of soil-structure interaction cannot be neglected nor can other dynamic influences.

Figure 11 is a section through a typical reactor building. This building is roughly square in plan. For the dynamic analysis of this building an equivalent multi-mass mathematical model, shown in Figure 12, is constructed to approximate the system. Masses are lumped at each floor level except that the top story steel frame is represented by an equivalent multi-mass system. Each story level mass represents the mass of concrete and equipment at each floor and the tributary mass of the concrete and equipment between adjacent floors. The top story masses are similarly developed but include the tributary mass of the walls, frame, bridge crane, and the mechanical equipment of the top story. The average area and moment of inertia of the concrete between floors is used to determine the stiffness characteristics between masses. The stiffness of the top steel story is determined by calculating the force-deflection relationship of the steel bracing.

The soil-structure interaction of the reactor building, whether founded on rock or soil, may have a considerable effect on the response of the structure. For this reason it is necessary to include in the model the effect of the foundation material. Equivalent springs may be introduced as indicated in Figure 12. The stiffness of these springs is determined from equations developed for the case of a rigid plate on a semi-infinite elastic half-space. (9)(10) The stiffness of the springs using these equations has been checked using elastic finite element methods of analysis. Although excellent agreement was found in the cases checked, the finite element method offers advantages; in fact, it is expected that it will soon be feasible to model the entire soil-structure system using finite element techniques. In order to apply the elastic half-space equations, or use the finite element methods, it is first necessary to obtain the elastic properties of the foundation material. This is done by appropriate field and laboratory tests.

The effect of rocking and translation of the foundation is coupled with and greatly influences the response of the significant modes of vibration. Thus the effect of the flexibility of the foundation can not be treated as an independent mode, as has been done in many cases.

When the mathematical model, the building properties and the foundation springs are determined, computer programs based on known procedures can be used to determine the stiffness and mass matrices, natural periods and mode shapes, and various responses (time-histories and maximum values). The details of the procedures will not be discussed here due to lack of space and since these are available elsewhere.

The first three natural mode periods of the model of Figure 12 are given in Table II for two different sets of foundation springs. One set (case 1) of foundation springs represents the actual soil conditions and are shown in Figure 12. The other set (case 2) represents a much stiffer soil, resulting in foundation spring stiffnesses that are roughly twelve times greater than those shown in Figure 12. The results show that the soil conditions have a significant effect on the natural mode periods and thus also affect the building responses to earthquake ground motion.

The responses of the mathematical model of the reactor building to the N69°W component of the Taft 1952 earthquake records were computed using five percent of critical damping in each of the lowest three modes. The maximum values of the shear, overturning moment, and acceleration responses are shown in Figures 13 through 15. The maximum value of the various responses at different levels do not necessarily occur at the same time. For the purpose of comparison, the shear forces and overturning moments obtained by the application of Uniform Building Code (9) are also shown in two of the figures for $K = 1.33$ and $Z = 1.0$. The rigorously computed fundamental mode period of 0.34 sec was used in these computations. The code formula would lead to a fundamental period of about 0.8 sec for this unusual structure, as compared to the 0.34-sec value which includes the effects of ground flexibility. Had the 0.8-sec value been used in the code shear equations, the design values would have been further reduced below those computed for response to the real earthquake motion.

There are some excellent references (8)(12)(13) on various phases of the computation procedures involved in multi-mass analyses. However, the detailed refinements and the coordination of the many disciplines required for massive time-history analyses are complex and important. In the nuclear power plant analysis outlined herein, many participated including Messrs. Haupt, Husain, Jain, Solanki, and Soleymani, with E. J. Keith supervising.

CONCLUSION

The earthquake response analyses of unusual buildings -- whether "unusual" refers to geometry, to size, to shape, to asymmetry, and/or to risk and the consequences of failure -- can now be, and should be, accomplished by time-history or other realistic methods. The degree of reliability is limited mainly by the ability to model properly real structures and soil-structure systems, and probable ground motion at any given site. Generally, the design values obtained from normal seismic code provisions applied to unusual structures, are much less than those obtained from specific response analyses. The record of great differences between ordinary code values and apparent real values in severe earthquakes (based upon time-history analysis) continues to grow. At the same time, the list of damages to, or failures of, buildings designed to ordinary code values by ordinary static procedures is growing. It is essential for unusual configurations -- and desirable for all configurations -- to reconcile analysis and design procedures with the real earthquake problem and its probabilistic aspects.

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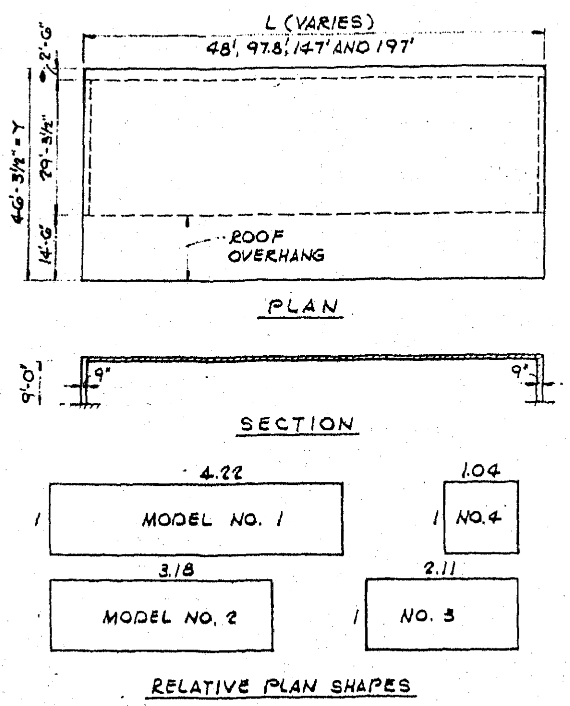

FIG. 1 - DIAPHRAGM - WALL MODELS

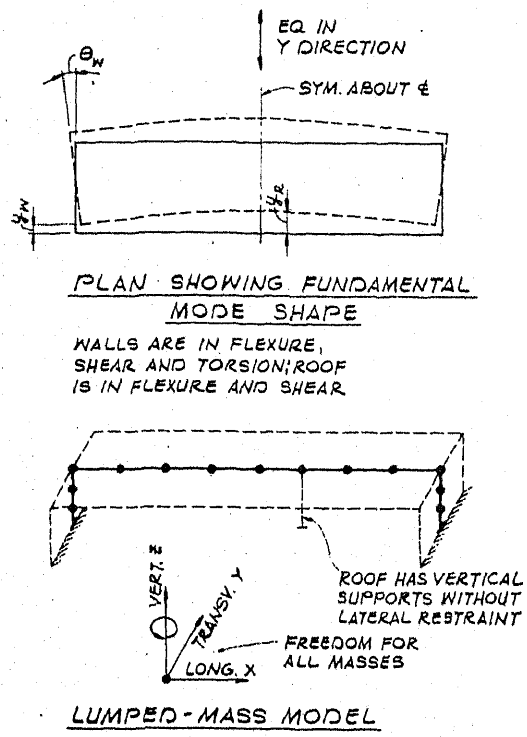

FIG. 2 - MODE SHAPE AND MODEL

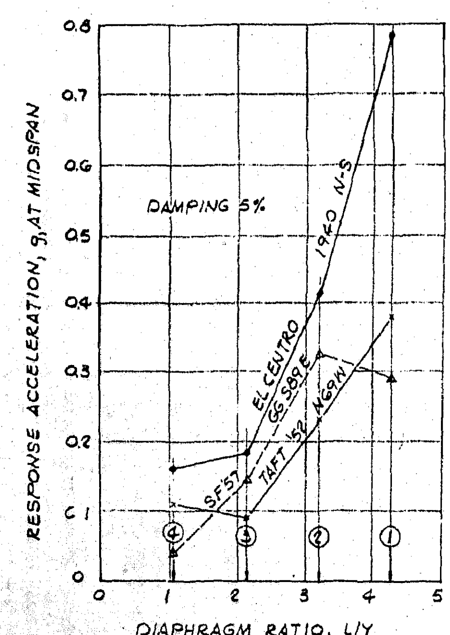

FIG. 3 - DIAPHRAGM RESPONSE

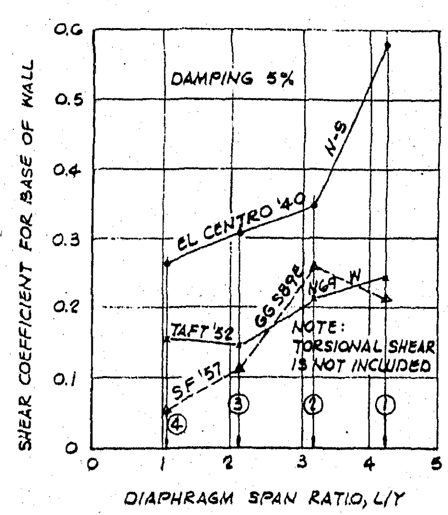

FIG. 4 - WALL BASE SHEAR COEFFICIENT VS. DIAPHRAGM SPAN RATIO

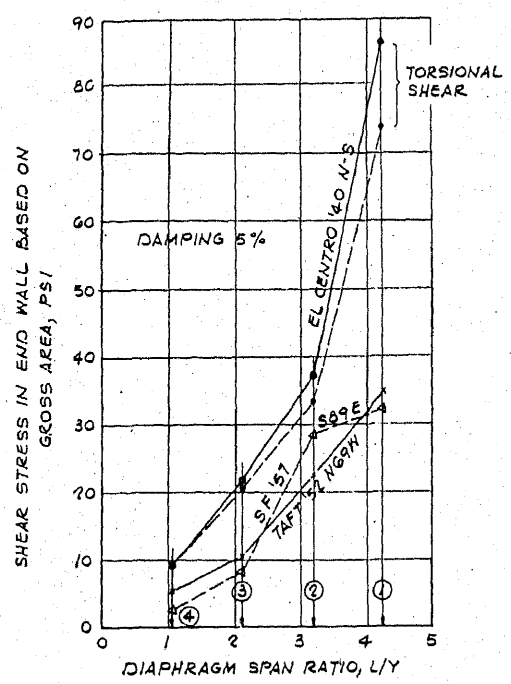

FIG. 5-SHEAR STRESS IN END WALL

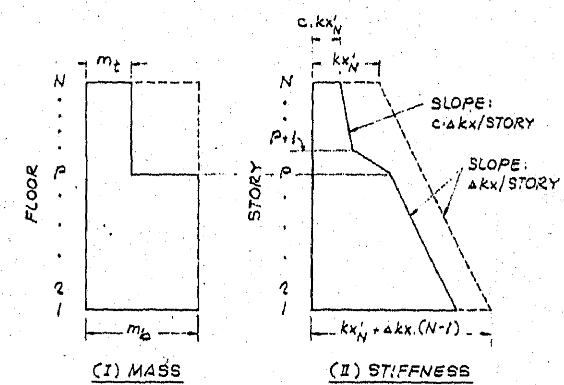

FIGURE 7-ASSUMED MASS AND STIFFNESS DISTRIBUTION

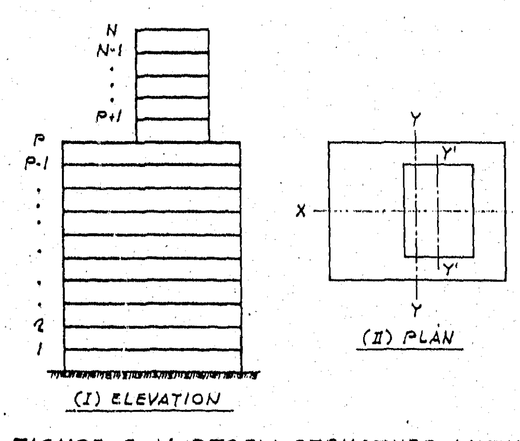

FIGURE 6-N-STORY STRUCTURE WITH SETBACK AT P^{TH} STORY

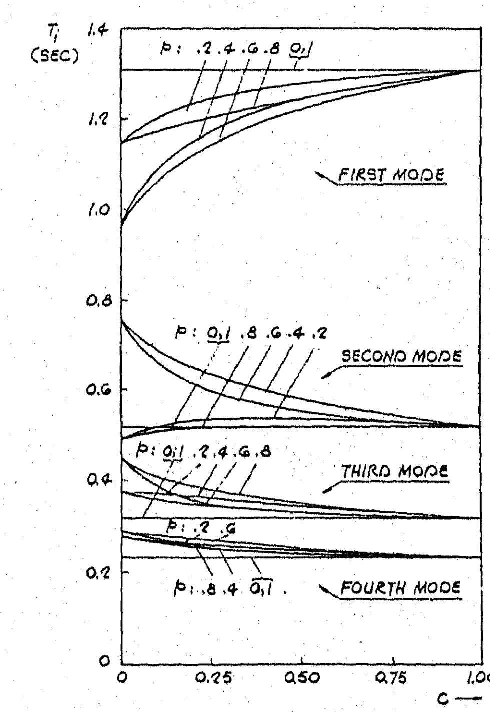

FIGURE 8-FUNDAMENTAL AND HIGHER MODE PERIODS, SHEAR BUILDING

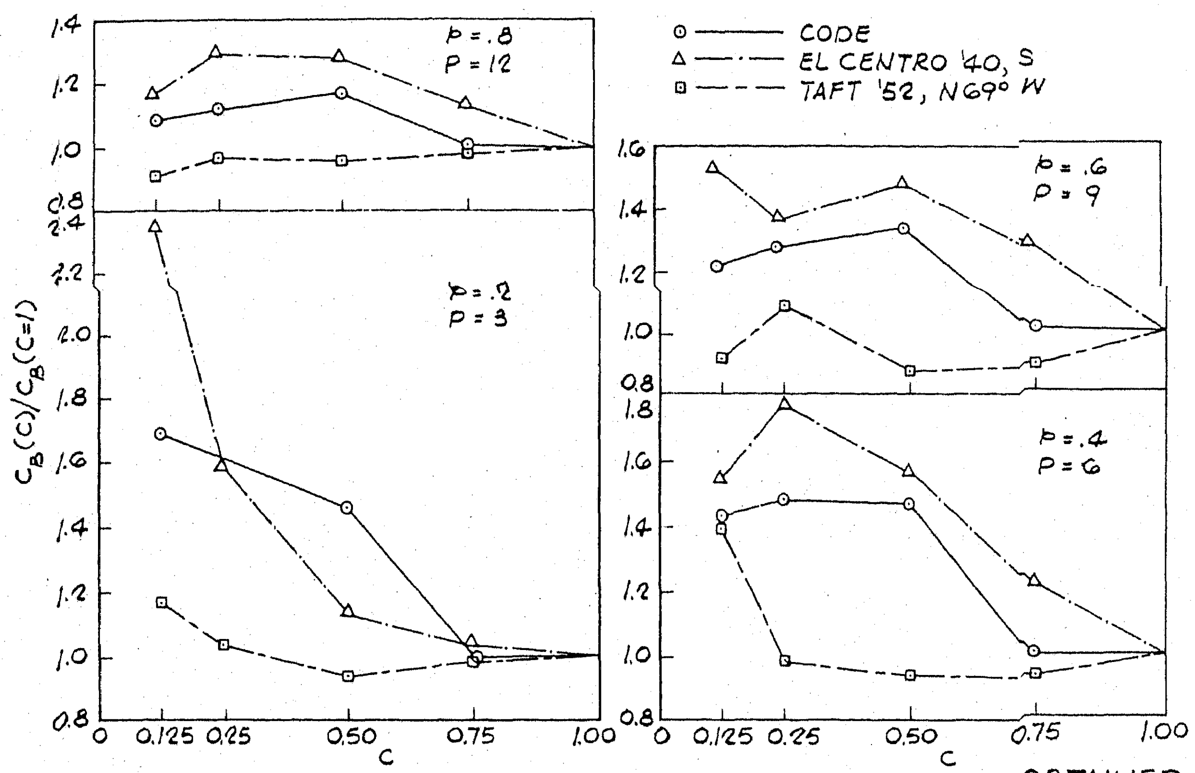

FIG. 9 - COMPARISON OF EFFECT OF SETBACK ON C_B AS OBTAINED FROM RESPONSES AND CODE SPECIFICATIONS

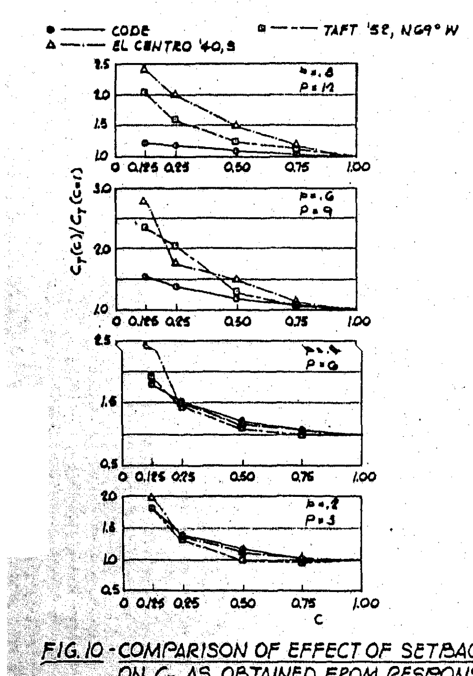

FIG. 10 - COMPARISON OF EFFECT OF SETBACK ON C_T AS OBTAINED FROM RESPONSES AND CODE SPECIFICATIONS

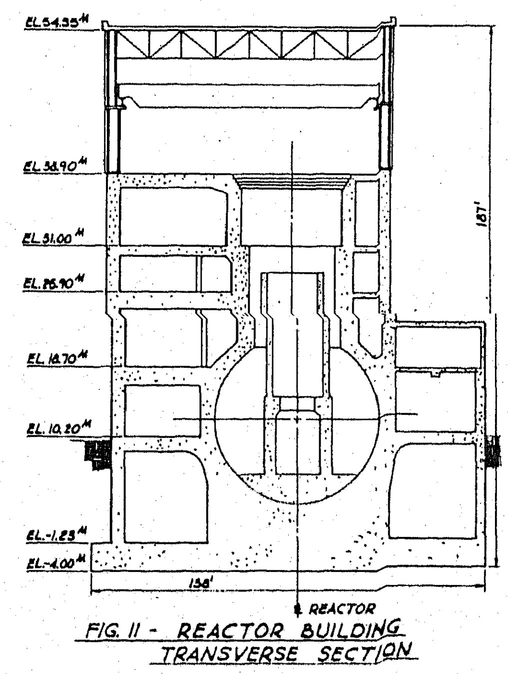

FIG. 11 - REACTOR BUILDING TRANSVERSE SECTION

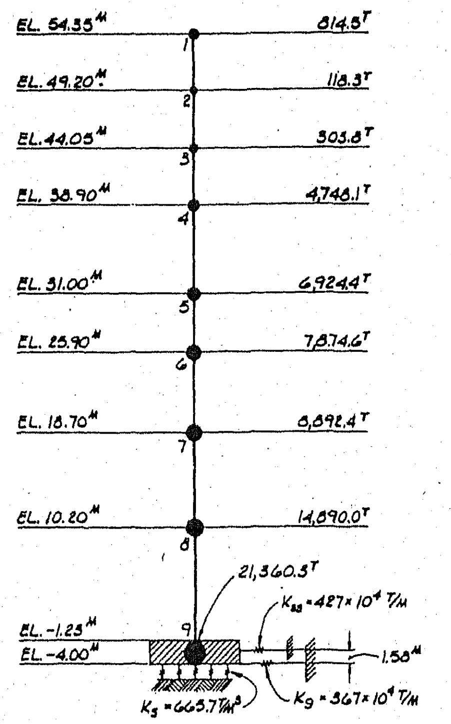

FIG. 12 - REACTOR BUILDING MODEL

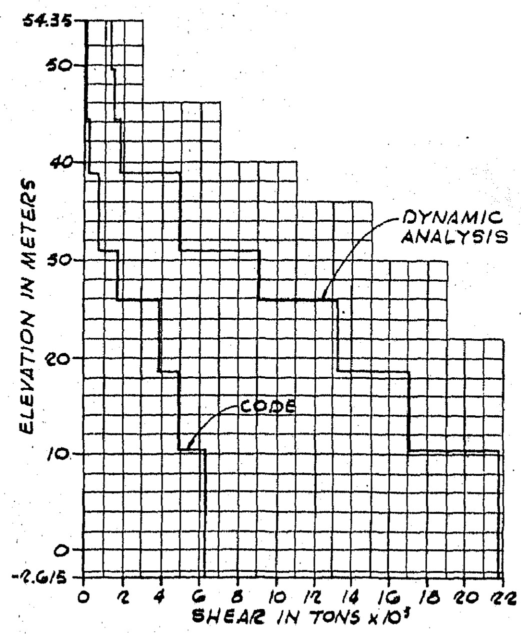

FIG. 13 - MAXIMUM STORY SHEARS, REACTOR BUILDING

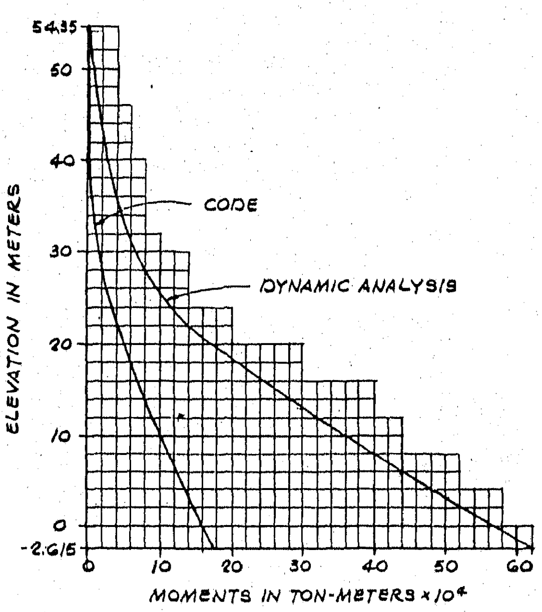

FIG. 14 - MAXIMUM STORY OVERTURNING MOMENTS, REACTOR BUILDING

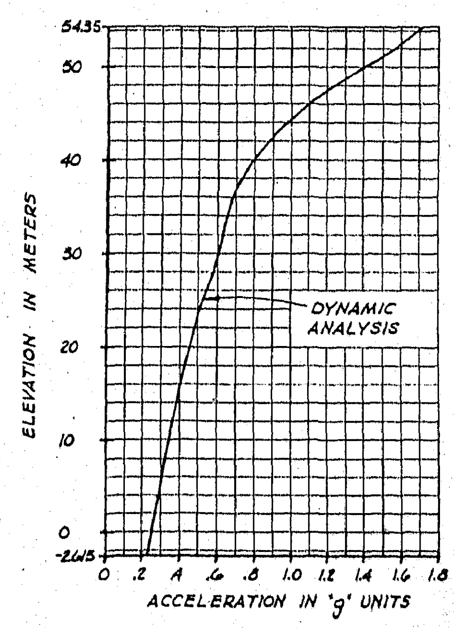

FIG. 15 - MAXIMUM FLOOR ACCELERATION REACTOR BUILDING

Table I - Natural Periods and Relative Model Deformations

Model number	L/Y	T_1, sec	T_2, sec	T_3, sec	y_R/y_W
1	4.22	0.175	0.084	0.054	14.7
2	3.18	0.119	0.060	0.039	8.5
3	2.11	0.076	0.040	0.026	4.7
4	1.04	0.042	0.022	0.014	2.4

Table II - Natural Mode Periods, Nuclear Power Plant

Mode	Natural Periods, sec	
	Case 1	Case 2
1	0.344	0.220
2	0.191	0.160
3	0.108	0.058