# RESPONSE OF THE OLIVE VIEW MEDICAL CENTER MAIN BUILDING DURING THE SAN FERNANDO EARTHQUAKE

by

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## SYNOPSIS

Dynamic response of the main building of the Olive View Medical Center, damaged beyond repair during the 1971 San Fernando earthquake, is analyzed and the principal cause of the severe damage identified.

## INTRODUCTION

The intensity of ground shaking near the causative fault during the San Fernando earthquake was rather strong resulting in damage to many structures(1). The severe damage to the buildings of the Olive View Medical Center, designed in accordance with modern codes, was of special concern from the point of view of modern structural engineering. The main building of the center, a six-story reinforced concrete structure, was damaged beyond repair. The performance of the main building during the earthquake is analyzed with the aim of identifying the reasons for its damage and their implications to present code requirements.

## STRUCTURAL SYSTEM

The plan of the upper five stories of this reinforced concrete building is composed of four wings intersecting each other at right angles, having an open courtyard in the center (Fig. 1). The ground story extends in plan beyond the upper stories, is underground on the north and west sides, and separated from the retaining wall by a 2-4 in. space (seismic joint). The structure is made up primarily of a flat slab-column system with drop panels at the columns. The columns used were of two types: tied and spirally reinforced. Shear walls were provided in the upper four stories but not in the ground and first stories (Fig. 1). The stiffness and strength of the lower stories were therefore smaller than the upper four stories resulting in a system with two 'soft' stories.

## PRINCIPAL DAMAGE

The severe structural damage to the building was concentrated in the ground and first stories; the upper stories were only lightly damaged. The earthquake caused large permanent deformations which consisted of translation towards the northeast combined with clockwise rotation of the structure above the first floor. Permanent drifts up to 10 in. in the ground story and 30 in. in the first story were measured after the earthquake. The displacements of the first floor exceeded the 4 in. separation available causing impact of the building against the north retaining wall and the warehouse near unit D. The extended part of the ground story, including the terrace canopy on the south and east sides of the building collapsed due to brittle failure of the supporting columns. Similar failures occurred in the tied columns in the first story. Although the spirally reinforced columns suffered considerable spalling and cracking,

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many of them were able to develop significant ductility; some, however, failed due to spirals terminating before the joint. The flat slabs and drop panels were also damaged in a number of locations in the first two floors.

## GROUND MOTION

The Olive View Medical Center was located less than 2 miles from the surface faulting in the zone of most intense shaking. The ground motion at this site was, unfortunately, not recorded. The only accelerogram obtained in the area of strongest shaking was the one at Pacoima Dam which had a peak value of 1.25 g, the largest so far measured during an earthquake. Because of the unusual location of the accelerograph(2) -- on a rocky spine adjacent to the dam with the rock undergoing extensive cracking during the earthquake -- this accelerogram may not be representative of the ground shaking at other nearby sites such as the medical center.

In general, except for the Pacoima accelerogram, the strong motion records were similar to the usual California earthquake records. In simulating ground motion for this site from a filtered white noise process, it is therefore appropriate to choose the filter frequency as 2.5 cps and damping 60% of critical(3). The filtered signal is multiplied by an intensity function of time, the shape and duration of which is selected to be similar to that for other records during the earthquake. The peak acceleration is estimated to be 0.5 g. The simulated ground motion and its response spectrum are presented in Fig. 2. The spectrum intensity for 20% damping is about 150% of that of the El Centro 1940 motion.

A very recent analysis(4) of Pacoima Dam including foundation and abutment rock has led to a derivation of the motion on firm rock near the base of the dam from the recorded motion at the instrument site. This computed Pacoima Dam base rock motion is possibly representative of the most intense ground shaking during the earthquake. A very unusual feature of this motion are three acceleration pulses each of 2/3 to 1 sec. duration which leads to largest spectral values in the one to two second period range (Fig. 2)

## EARTHQUAKE RESPONSE

Structural Properties: Shear walls in 2nd through 5th stories, assumed to remain elastic, are idealized as an 'equivalent' frame with rigid beams and column spacing same as in the bottom two stories. With the aid of finite element analyses of plates, flat slabs with drop panels are idealized as an 'equivalent' system of beams. The strength of flat slabs are determined from yield line patterns considering the drop panel and points of cut-off of reinforcing steel.

The areas and moments of inertia of members are based on their initial section, i.e., the changes in the resisting section due to cracking and spalling of concrete during the earthquake are not accounted for. The moment-curvature (M-$\phi$) relationship is determined from the stress-strain curve for concrete, including the effect of confinement due to transverse reinforcement. The interaction between moment and axial load is considered in defining the yield curve. Clear spans are considered in the computations of member properties.

For the purposes of inelastic dynamic analyses, the M-$\phi$ relationships are idealized as bilinear with yielding stiffness 3% of the initial elastic stiffness. The unloading and reloading is assumed to follow a bilinear hysteresis loop ignoring any degradation in stiffness or strength that may arise due to cracking and spalling of concrete. The members are assumed to have sufficient shear capacity to preclude brittle failures.

The mass of the structure is lumped at the floor levels. The damping matrix is taken to be proportional to the mass matrix with 5% of critical damping in the first mode of vibration.

Wing D Model: A complete analysis including coupled two-dimensional horizontal and torsional vibration, realistic inelastic material behavior and three-dimensional ground motion of this complex structure is impractical. Consequently, a part of Wing D of the building which appears to have vibrated primarily in the north-south direction is selected for a detailed study.

The structural system considered consists of frames 24, 26, 27, 28 and 29 in Wing D (Fig. 1), with frame 24 separated from the rest of the building along column line M. Preliminary analyses indicated that the terrace canopy, extending outside column lines G and M, failed very early in the earthquake at a drift of about 0.3 in.; it is therefore not included in the frames. Damage observations indicated that plastic hinges developed near the base of the ground story columns; the columns are therefore assumed as fixed at the ground floor level.

The validity of this Wing D model is established by comparing its periods and mode shapes of vibration with those for the complete building. The first three modes of the complete building, obtained using the computer program TABS[5], are presented in Fig. 3. The closeness of the translational and torsional periods indicates that neglect of torsion can result in considerable under-estimation of the response. The first two translational modes of the Wing D model are compared with the north-south components of the corresponding (2nd and 6th) modes of the complete building; the agreement is satisfactory. It should be noted that the triangular first mode implied in the code is not appropriate for this building (Fig. 4).

Elastic Analyses: The maximum displacements of Wing D (Fig. 5) due to the simulated ground motion indicate that the deformations are concentrated in the ground and first stories. The story shears exceed the design values by a factor of 10 or so (Fig. 6), indicating that appreciable yielding of the building, particularly in the first two stories, will result. Analysis of a modified Wing D in which the shear walls are continued down through the first and ground stories, with 14 in. and 16 in. thickness respectively, demonstrates that the deformations are now more uniformly distributed through the height.

Inelastic Response: A static analysis of the Wing D model for gravity loads is first performed. The response to the simulated ground motion, considering inelastic deformations and P-$\Delta$ effects, demonstrates that the slabs and columns of the lower two stories start to yield in the first two seconds even before the ground motion has reached its maximum intensity

(Fig. 7), causing elements which had low shear capacities, such as the first floor slabs and tied columns to fail prematurely and the outer shells of the spirally reinforced columns to spall. The yielding and associated large deformations are concentrated in the lower two stories (Figs. 8, 10-12).

Although the response to the simulated ground motion identifies some of the features of the damage that occurred to the building, it lacks in a number of ways. The computed displacements are much smaller than those caused by the earthquake, the drifts in the lower two stories are similar in magnitude in contrast to the permanent drifts due to the earthquake being much larger in the first story compared to the ground story, and the time history of displacements indicates a number of oscillations with reversal of drifts which is inconsistent with observed damage.

There are two main factors which account for these discrepancies. Firstly, the ground motion at the site during the earthquake is not known. Once a panel yielding mechanism (Fig. 9) forms, the subsequent response depends strongly on the characteristics of ground motion. An acceleration pulse of relatively long duration, such as in the Pacoima base rock motion (Fig. 2), may cause the first two stories to continue to yield in one direction resulting in irreversible large drifts. Secondly, failures of elements during the early phase of the earthquake and their influence on the subsequent response is not accounted for in the analyses. Thus, the damage to some columns in lines G and M due to the collapse of the terrace canopy, the drastic decrease in resistance of some of the flat slabs which failed rather prematurely, the sudden release of energy and transfer of load to the spirally reinforced columns and the decrease of story resistance due to brittle failure of the tied columns, the decrease in capacity of the spirally reinforced columns due to spalling of the outer shell (Fig. 13), and other such factors were not considered. Furthermore, torsion response is ignored and only the north-south component of ground motion is considered.

The response to Pacoima base rock motion (Fig. 2) demonstrates that the long duration acceleration pulses indeed cause much larger deformation in the first two stories and significant permanent distortion (Figs. 11, 12, 15). Although, as mentioned, the analysis procedure has a number of limitations, it seemed worthwhile to determine the response to more intense ground motions which may cause larger displacements consistent with those observed. For this purpose, the two ground motions of Fig. 2 were amplified arbitrarily by a factor of 2. The results (Figs. 11, 12) indicate that very large displacements are caused by the amplified Pacoima base rock motion. Among the four ground motions considered, this is the only one which causes first floor displacements to exceed 4 in., indicating impact against the warehouse. Once that happens, the building has two 'soft' stories when vibrating to the south but effectively only one when vibrating to the north, which will result in a large increase of drifts in the first story over those in the ground story.

The computed story shears indicate that the strength of the lower two stories was about four times the design value (Fig. 14). The strength of upper stories must exceed the design value by a much larger factor because of the presence of shear walls. That the building was much stronger than intended by the code and yet severely damaged is primarily because the ground motion was rather severe and the 'soft' lower two stories were called upon to absorb almost all the energy thus undergoing very large inelastic deformations.

## CONCLUDING REMARKS

Many of the design implications of the damage observed in the main building at the Olive View Medical Center have been discussed in a companion paper(6).

The preliminary dynamic response analyses presented in this work demonstrate that the severe damage to this building was due primarily to the severity of the ground motion, and the large change in stiffness and strength across the second floor level caused by the termination of shear walls, thus imposing extremely large ductility requirements in the elements of the first two stories, which had apparently not been anticipated. It is extremely difficult to design structural elements, especially in reinforced concrete, to safely undergo such large inelastic deformations; even if it were possible, restoration of the building after an earthquake may not be feasible. Codes should require rational dynamic analyses of unusual and important structures so that deficiencies in their structural system can be identified in the design process.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. G. W. Housner and P. C. Jennings, "The San Fernando California Earthquake," Int. J. Earthq. Engng. Struct. Dyn., Vol. 1, No. 1, 1972.
2. M. D. Trifunac and D. E. Hudson, "Analysis of the Pacoima Dam Accelerogram -- San Fernando, California Earthquake of 1971," B. Seism. Soc. Amer., Vol. 61, No. 5, 1971.
3. P. Ruiz and J. Penzien, "Stochastic Seismic Response of Structures," J. Engng. Mech. Div., Am. Soc. Civ. Engrs., Vol. 97, No. EM2, 1971.
4. R. B. Reimer, R. W. Clough and J. M. Raphael, "Evaluation of Pacoima Dam Accelerogram," 5WCEE, Rome, June 1973.
5. E. L. Wilson and H. H. Dovey, "Static and Earthquake Analysis of Three-Dimensional Frame and Shear Wall Buildings," Report No. EERC 72-1, Univ. of Calif., Berkeley, 1972.
6. V. V. Bertero, et. al., "Design Implications of Damage Observed in the Olive View Medical Center Buildings," 5WCEE, Rome, June 1973.

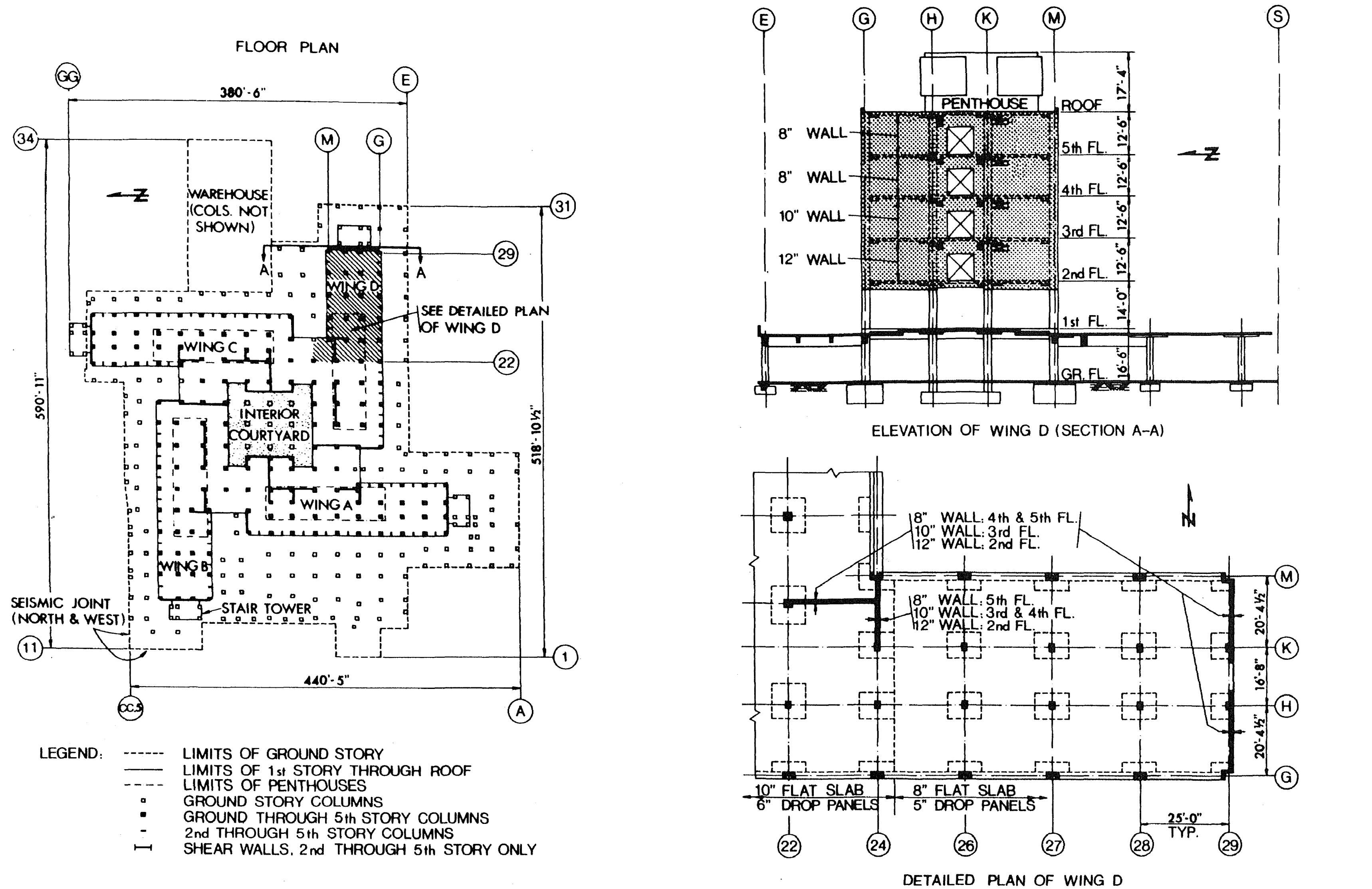
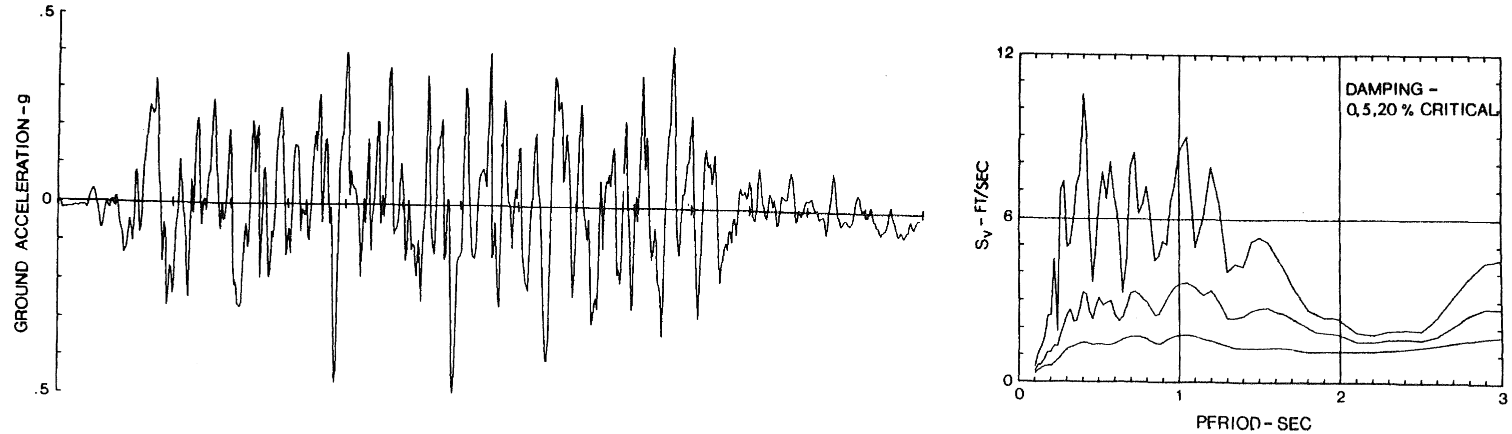
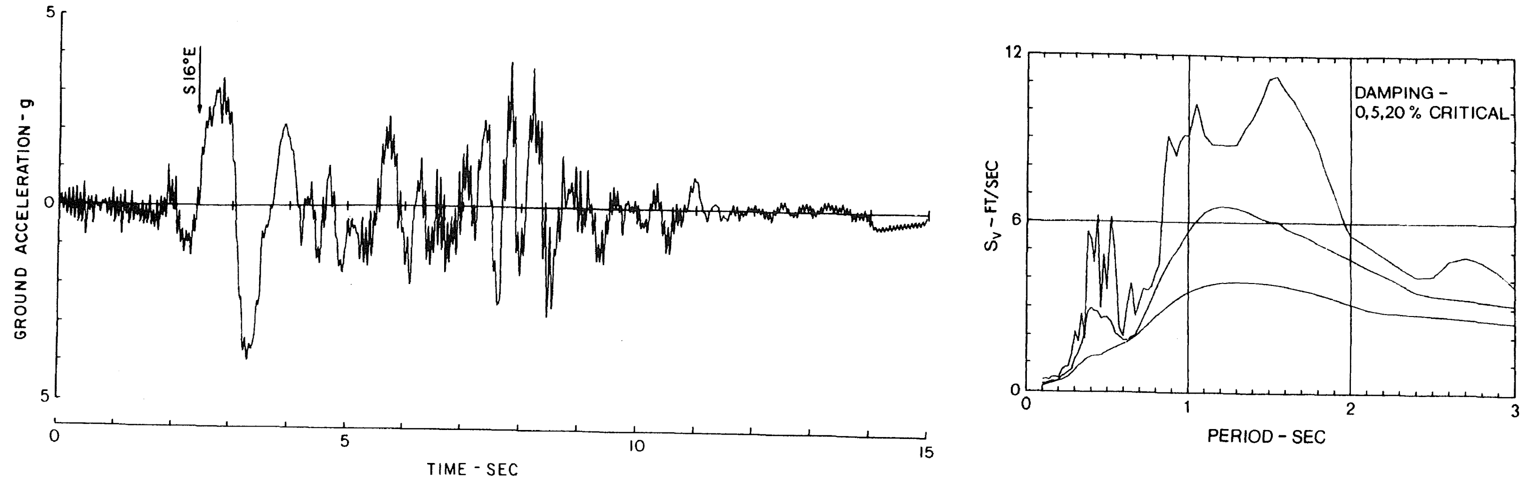



FIG.1: MAIN BUILDING.




**SIMULATED GROUND MOTION**




**COMPUTED BASE ROCK MOTION AT PACOIMA DAM** (After Reimer, et. al.)

FIG. 2: GROUND MOTIONS AND VELOCITY RESPONSE SPECTRA.

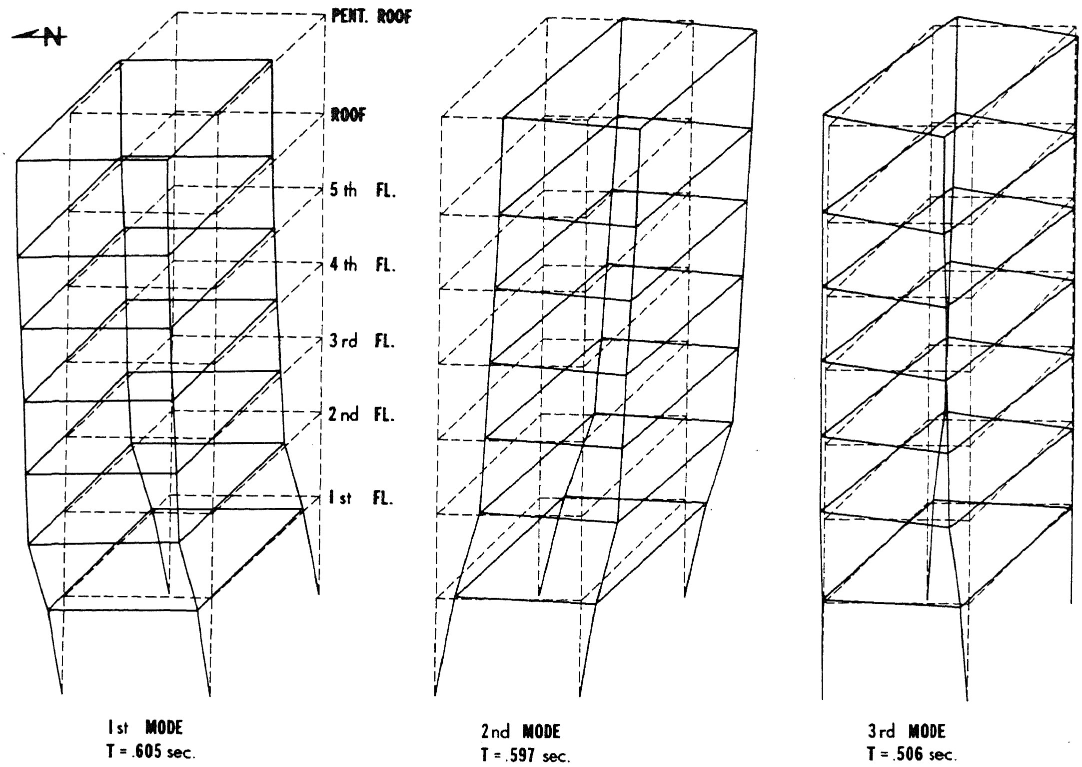



FIG.3: PERIODS AND MODE SHAPES OF ENTIRE STRUCTURE.

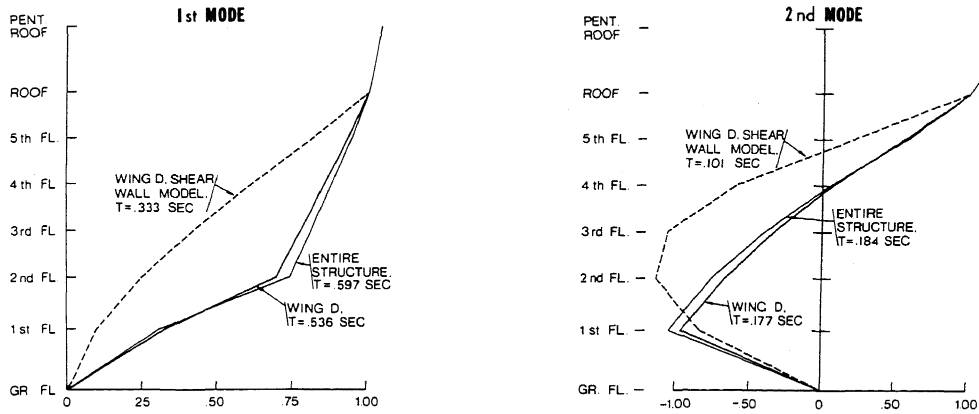



FIG.4: COMPARISON OF WING D AND ENTIRE STRUCTURE MODES.

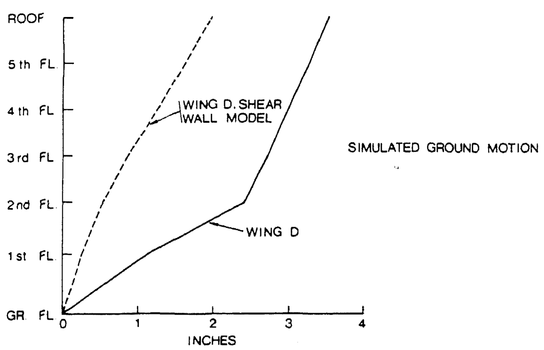



FIG.5: MAXIMUM ELASTIC FLOOR DISPLACEMENTS.

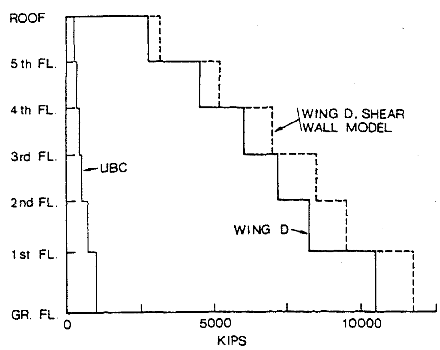



FIG.6: MAXIMUM ELASTIC STORY SHEARS.

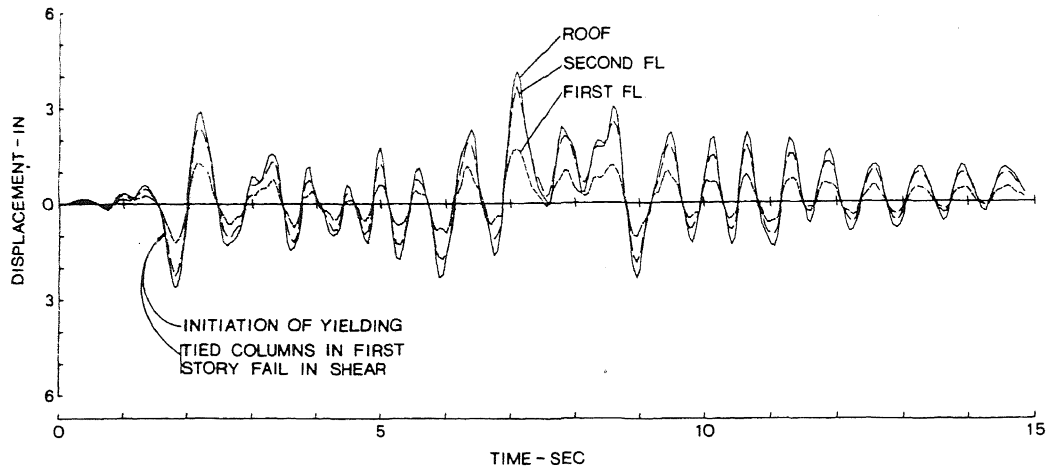



FIG.7: DISPLACEMENT RESPONSE TO SIMULATED GROUND MOTION.

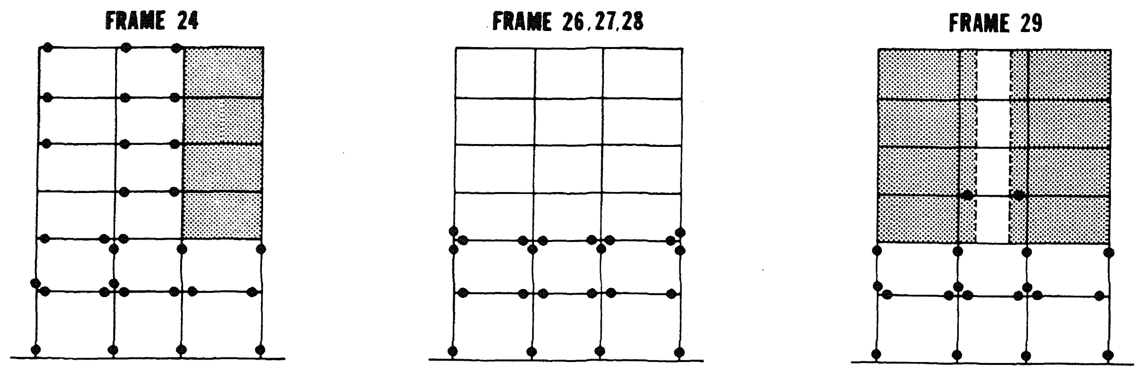



FIG.8: PLASTIC HINGES - SIMULATED GROUND MOTION.

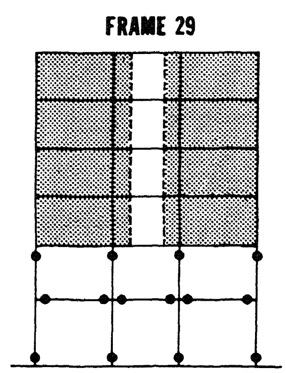



FIG.9: PLASTIC HINGES AT 1.75 SEC.

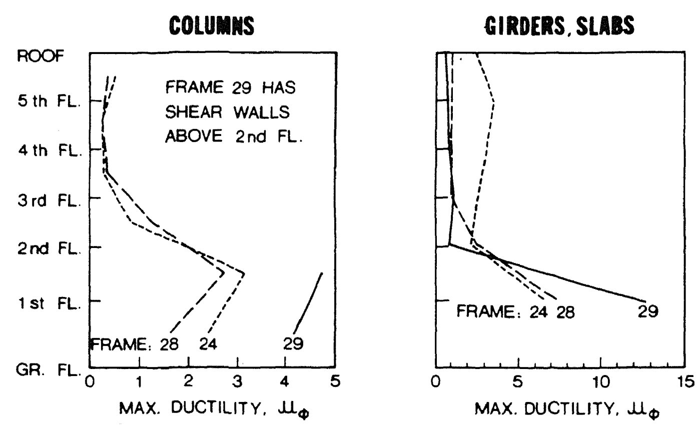



FIG.10: DUCTILITY REQUIREMENT - SIMULATED GROUND MOTION.

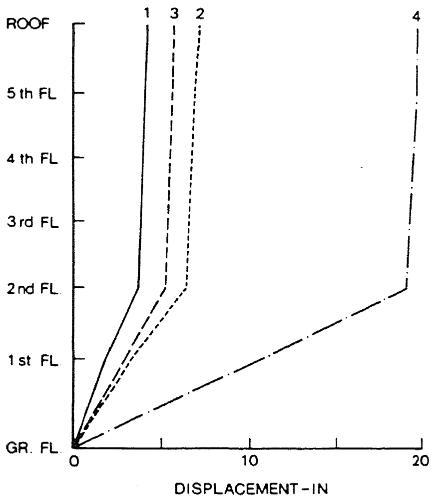



FIG.11: MAXIMUM FLOOR DISPLACEMENTS.

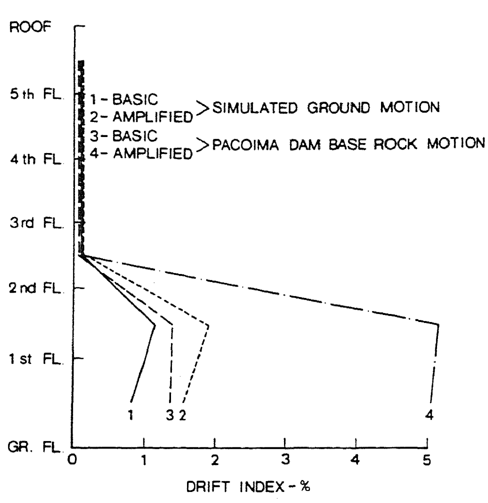



FIG.12: MAXIMUM STORY DRIFT INDICES.

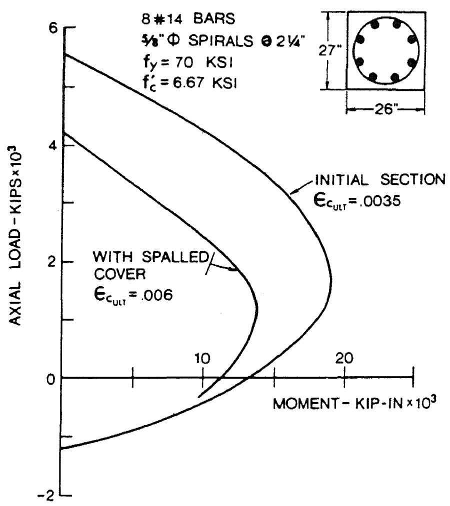



FIG.13: COLUMN INTERACTION CURVE.

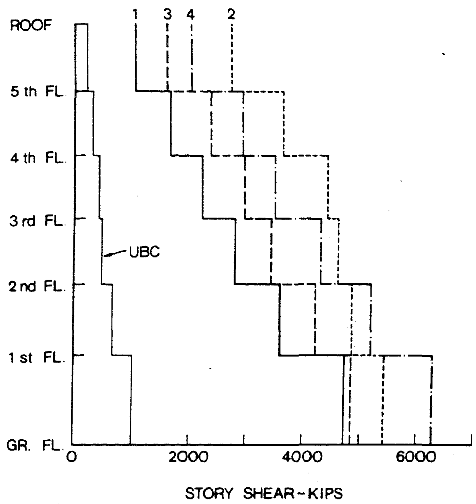



FIG.14: MAXIMUM STORY SHEARS.

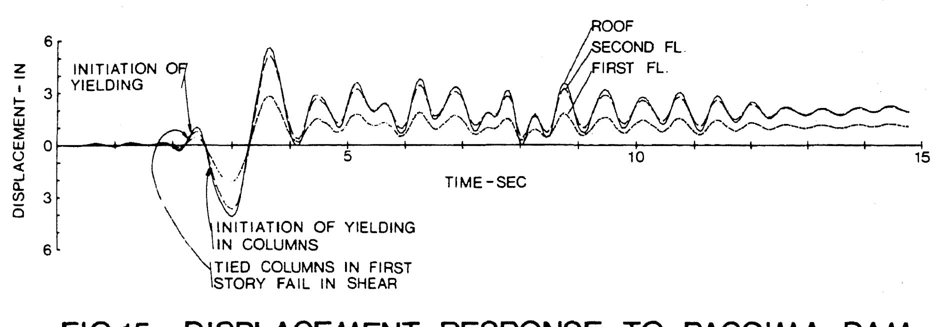



FIG.15: DISPLACEMENT RESPONSE TO PACOIMA DAM BASE ROCK MOTION.