CORRELATION STUDIES OF SEISMIC RESPONSE OF REINFORCED CONCRETE MOMENT RESISTING FRAMES

Filip C. Filippou
Department of Civil Engineering
University of California Berkeley

ABSTRACT

This study focuses on the correlation of analytical predictions with the measured seismic response of two reinforced concrete moment resisting frames during the October 1, 1987 Whittier Narrows earthquake. The first building is a five story warehouse with a flat slab and a perimeter frame and the second is a twenty story hotel with moment resisting frames in both directions. Both buildings have a regular, rectangular, symmetric layout. To study the seismic response three dimensional models of the two buildings were subjected to the accelerations recorded at the base. The study assesses the effect of modeling assumptions on the fundamental period and the response of the buildings.

INTRODUCTION

On October 1, 1987 an earthquake measuring 6.1 on the Richter scale took place in the greater Los Angeles area with the epicenter located near the town of Whittier. Damage was moderate over a broad area and extensive in certain locations such as downtown Whittier. A preliminary report by the firm EQE, Inc. estimated total damage in excess of $100 million [1]. On October 4, 1987 an aftershock measuring 5.3 on the Richter scale with its epicenter located almost at the same point as the main event shook the area. A large number of structures in the greater Los Angeles area were instrumented as part of the California Strong Motion Instrumentation Program (CSMIP). These yielded a wealth of data on the seismic response of various types of structures.

The objective of this study is the utilization of the records obtained during the Whittier earthquake and its aftershock in assessing the capabilities of present state-of-practice analytical models to predict the seismic response of reinforced concrete buildings in the elastic range. The study attempts to show the effect of modeling assumptions on response prediction and their ramification in the earthquake resistant design of structures. A brief evaluation of current procedures for establishing the equivalent static lateral loads due to earthquake excitations is also presented.

Attention is focused on a particular type of structure in order to facilitate the comparison of response and the drawing of conclusions. The structural type of interest in the reinforced concrete moment resisting frame. Two such buildings were selected in the first phase: the first is a five story warehouse with a flat slab and a relatively stiff perimeter frame. The second is a twenty story hotel with moment resisting frames in both directions. Both buildings have a regular, rectangular, symmetric layout. The intent of the selection was to eliminate irregular buildings from consideration where torsional effects could be significant and concentrate on modeling assumptions for regular multistory buildings. In the following a brief description of the layout, the structural system and the instrumentation of each building is given. Subsequently, a discussion of modeling assumptions and the development of the three dimensional analytical model is presented followed by correlation studies of analytical predictions with measured response. A discussion of the results in light of current earthquake resistant design provisions and preliminary conclusions from this study complete the presentation.

FIVE STORY PERIMETER FRAME

The first building selected for this study is a five story plus basement warehouse located in Los Angeles at a distance of approximately 14 km from the epicenter of the main event and about 12 km from the epicenter of the aftershock. The building has a rectangular layout measuring 280' by 360.5'. Gravity loads are carried by a 11 3/4" thick flat slab resting on circular columns of 32" diameter. The lateral force resisting system consists of a perimeter ductile moment-resisting frame with rather deep beams and columns. The foundation consists of spread footings. The structure was designed in accordance with 1970 Los Angeles Building Code. Fig. 1 shows the typical floor plan and framing system.

The building was extensively instrumented with all instruments functioning well and triggering during, both, the main event and the aftershock. Thus a wealth of data is available for in-depth studies of the building response. Four instruments were placed in the basement of the building. Two of these recorded the N-S accelerations along the east and the west wall, one the E-W accelerations at the east-wall and one the vertical accelerations imposed on the building. Three instruments were placed in each of the following floors: second, third, and topmost (roof). The instrumentation layout is shown in Fig. 2. The presence of three instruments measuring lateral accelerations in the basement and on each of several floors facilitates the study of the torsional response, if any, of the building. By comparing the two N-S acceleration records at the base of the building it is readily concluded that no rotational excitation is imposed at the base of the building, since the two records are almost identical. This fact also gives confidence about the accuracy of the recorded excitations.

It is interesting to note that the comparison of roof displacement response histories recorded in the N-S direction at the east and west wall of the building clearly shows the effect of torsional response (Fig. 3). This effect is not very significant for such a regular, symmetric building, but is not negligible either. In order to predict the three dimensional response of the building including the effects of torsion, a three dimensional model was developed. Initially, the model only accounts for the stiffness of the perimeter frame assuming that the lateral stiffness of the frame composed of the flat slab and the interior columns is relatively small. This is not true, however, and subsequent refinements will account for the lateral stiffness of the flat slab. It is interesting to point out the relatively large story height of 24 feet which leads to a fundamental period of vibration of about 1.45 sec in the N-S direction and 1.30 sec in the E-W direction. The maximum base acceleration was 0.18g in the E-W and 0.14g in the N-S direction.

The following considerations played an important role in the development of the three dimensional building model:

- In order to avoid complications in the response caused by the building foundation the model was assumed to fixed at the top of the basement walls. Unfortunately, no acceleration records are available at this level. Accounting for the relatively large in-plane stiffness of the basement walls it is assumed that the base acceleration propagates unaltered to the top of the basement. The analytical studies support the validity of this assumption.
- The depth of the members of the perimeter frame is considerable. The depth of the beams and the width of the columns is 66"=5.5 ft while the clear of the beams amounts to 22.5 ft in the transverse direction and 29.5 ft in the longitudinal direction. More importantly, the clear column height measures 19.5 ft. The span to depth ratio, thus, ranges from 3.5 in the case of columns to 5.5 in the case of beams in the longitudinal direction. Special attention needs to be devoted to the shear deformations of the members and the distortion of the beam-column joint region. The effect of this parameters is discussed in detail below.

- The mass of the floor slab is considerable. It makes up about 65% of the total mass of the building, the remainder contributed by the members of the perimeter frame (27%), the large interior columns (6%) and two small staircases located in the perimeter of the building.
- The building is so perfectly regular and symmetric that the small torsional response is caused by non-symmetric distribution of stiffness due to cracking of members and by the presence of two small staircases located in the perimeter of the building. The former effect is very difficult to evaluate. To account for the latter box-type columns are used to model the stiffness of the staircases.
- Initially, properties of structural members are based on gross uncracked concrete sections. This will be revised in subsequent refinements to account for the effect of cracking.

The program SAP89 developed by E.L. Wilson [3] is used in all following studies, which were all performed on a microcomputer.

	Study in Ref. [2]	Final model	Rigid panel zones	Neglect shear deformations	Remarks
Mode Number	Period [sec]	Period [sec]	Period [sec]	Period [sec]	
1	1.39	1.43	1.27	1.37	N-S
2	1.22	1.27	1.12	1.29	E-W
3	0.76	0.87	0.77	0.85	Rotation
4	0.46	0.47	0.42	0.46	N-S
5	0.40	0.41	0.36	0.42	E-W
6	0.26	0.29	0.25	0.28	Rotation

Table 1 Comparison of periods of vibration of first six modes of vibration of 5-story perimeter frame

Table 1 summarizes the periods of first six modes of vibration of the structure obtained in four cases: first a study conducted in ATC-2 Report and three cases with the present three-dimensional model investigating the flexibility of beam-column joint panel zones and the effect of shear deformations in structural members. The final model includes the effect of shear distortions and accounts for the flexibility of panel zones through a panel zone factor of 0.5, which is halfway between a fully rigid panel zone (factor=0) with member stiffness based on clear span dimensions and a completely flexible panel zone (factor=1.0), in which case the center-to-center dimensions of members are used.

It is interesting to point out that the fundamental period of vibration of the building is predicted equal to 1.08 sec in, both, N-S and E-W, directions by formula (12-3) of the 1988 Uniform Building Code.

In conducting the dynamic response analyses presented in the following the mass and stiffness properties of the final model in Table 1 are used. The damping ratio is assumed equal to 5% of critical in all vibration modes with ten modes being used in the analysis.

The correlation of the predicted response with measured data is presented in Figs. 4-6. Fig. 4 shows the roof displacement response history at the west wall in the N-S direction. Fig. 5 shows roof the displacement response history at the west wall in the E-W direction and Fig. 6 compares the two calculated response histories in the N-S direction to highlight the small torsional response that is obtained due to the non-symmetric arrangement of two staircases in the building. It can be concluded that the analytical model is capable of very accurately predicting the elastic response of this building to a moderate ground motion.

TWENTY STORY MOMENT FRAME

The second building used in this study is a twenty story hotel with basement located in the Hollywood area at a distance of 28 km from the epicenter. This building experienced a smaller ground excitation than the five story warehouse building, because of the larger epicentral distance. The maximum ground acceleration at the base was 0.11g in the N-S direction and 0.09g in the E-W direction. The lateral load resisting system consists of ductile moment resisting frames in, both, the N-S and the E-W direction. Two way slabs carried the gravity loading to the girders of the frame. The slab is 4" thick, with the exception of the corridors where the thickness is increased to 6". Of particular interest is the fact that 110 pcf lightweight concrete of 3,000 psi nominal strength was used for the slab and beams above the first floor. The columns are also of lightweight concrete with a nominal strength of 4,000 psi in the first ten floors and 3,000 psi in the floors above the tenth. Normal weight concrete is only used in the members of the basement and the first floor (lobby). A transverse section of the building is shown in Fig. 7.

The same building has been the subject of a study in connection with San Fernando Earthquake of February 9, 1971 published in [4]. The building, which was completed in 1967, was subjected to peak ground acceleration of 0.18g during that earthquake and suffered only minor architectural damage totaling $2,100 [4]. The field observations and the ensuing study did not indicate any structural damage. It is interesting to point out that two a dimensional planar frame model in the two principal directions of the building was used in the studies reported in [4].

	Study in Ref. [4]	Final model	Remarks
Mode Number	Period [sec]	Period [sec]	
1	2.21	2.31	Transverse
2	2.15	2.05	Longitudinal
3	——	1.91	Rotation
4	0.79	0.79	Transverse
5	0.74	0.70	Longitudinal
6	——	0.64	Rotation

Table 2 Comparison of periods of vibration of first six modes of vibration of 20-story frame

In this study a three-dimensional model of the entire structure was developed. All structural members, including staircases, were included in the model. Modeling of the twenty story building was a major effort, since the final structural model consists of 3,855 members, 1,767 joints and almost 5,000 degrees of freedom. In spite of its large size the model was analyzed on a personal computer with a 30 MB hard disk.

Many of the same modeling considerations as discussed in the case of the five story warehouse also played an important role here. Since the structural members were not deep in this case, distortions due to shear are negligible and were not accounted for. The effect of partially rigid joints is, however, again significant. Table 2 shows the periods of the first six mode shapes and compares the results of this study with those of Ref. [4].

The correlation of the predicted response with measured data is presented in Figs. 8 and 9. Fig. 8 shows the 16th-floor displacement response history at the west wall in the N-S direction. Fig. 9 compares the two measured 16th-floor displacement response histories in the N-S direction to show the significant torsional response that is obtained in this case. Even though Fig. 8 indicates a satisfactory agreement between predicted and measured displacements of the 16th-floor further studies are required to establish the cause of the significant torsional response.

CONCLUSIONS

The three dimensional model of a five story perimeter frame building yields very satisfactory agreement between calculated and measured response values. This seems to support the assumptions made in the development of the analytical model. The agreement is not as good in the case of a twenty story moment resisting frame, in large part due to the significance of the torsional response which is not represented by the model. Further studies are currently under way to shed light on the relation of modeling assumptions with actual building stiffness and mass distribution.

ACKNOWLEDGEMENTS

This study is supported by a grant from the Strong Motion Instrumentation Program of the Department of Conservation of the State of California. This support is gratefully acknowledged. The views expressed in the paper are those of the author and do not reflect the opinion of the sponsoring agency. The assistance of Drs. Shakal, Huang and Ventura is greatly appreciated. I would also like to thank graduate student David Sze for his tireless efforts in the preparation of the data of the analytical models. Finally, my most sincere thanks go to Professor E.L. Wilson for providing a copy of his powerful microcomputer program and for much advice during the course of the study.

REFERENCES

[1] A.F. Shakal, et. al., "CSMIP Strong-Motion Records from the Whittier, California Earthquake of October, 1, 1987. Report No. OSMS 87-05, California Department of Conservation, Oct. 1987.

[2] Applied Technology Council, "An Evaluation of A Response Spectrum Approach to Seismic Design of Buildings", Report No. ATC-2, San Francisco, Sept. 1974.

[3] E.L. Wilson, "SAP-89: A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures", Users Manual, University of California, Berkeley, Report, January 1989.

[4] L.M. Murphy, Editor, "San Fernando, California, Earthquake of February 9, 1971, U.S. Department of Commerce, Washington, D.C. 1973.

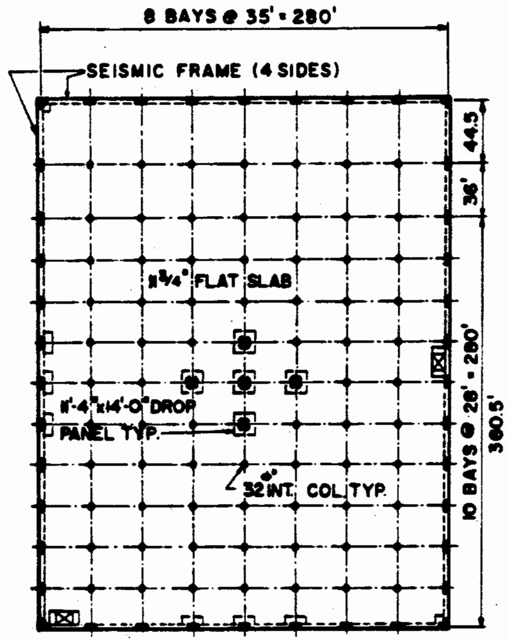


FIG. 1.0.1. Typical Floor Plan.

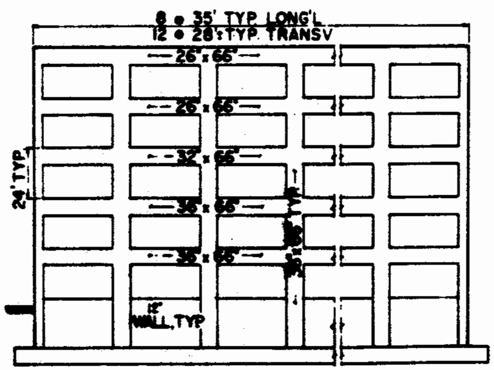


FIG. 1.0.2. Frame Elevation.

FIG. 1

SENSOR LAYOUT

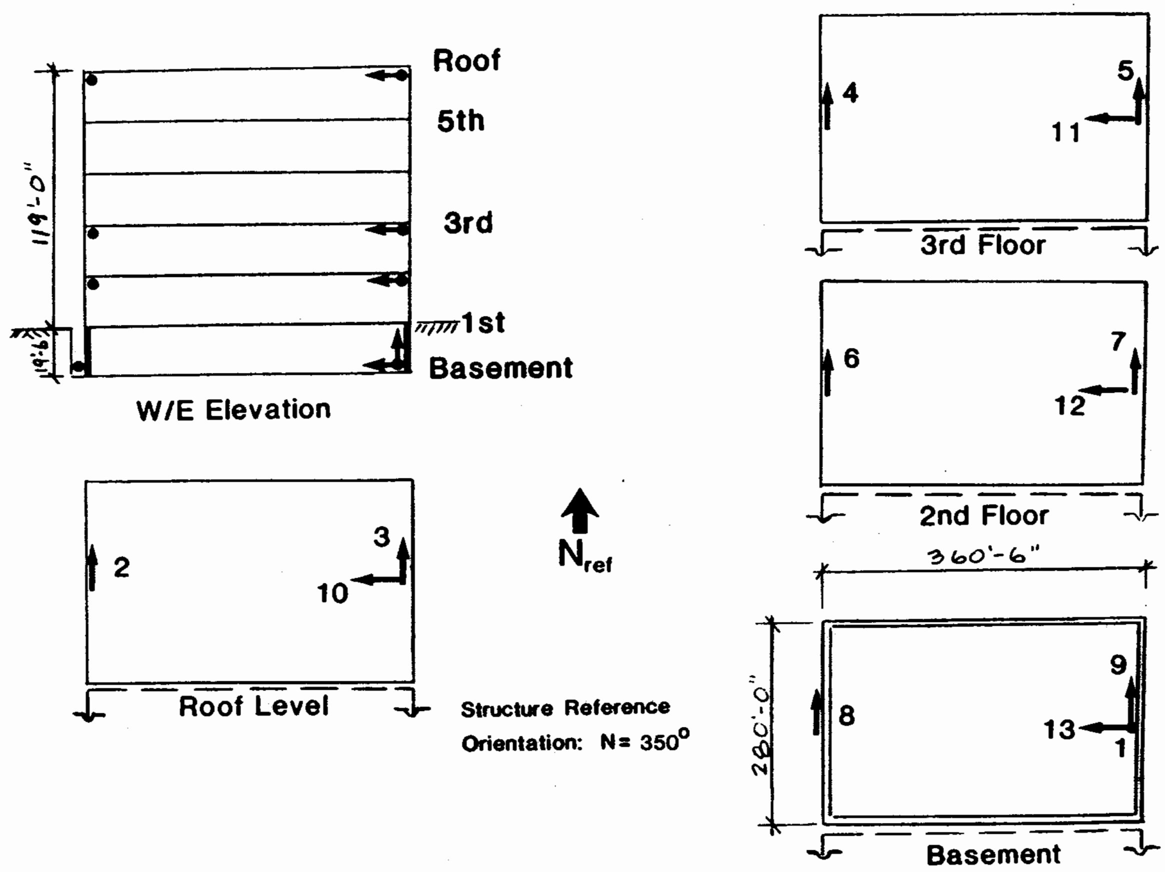


FIG. 2

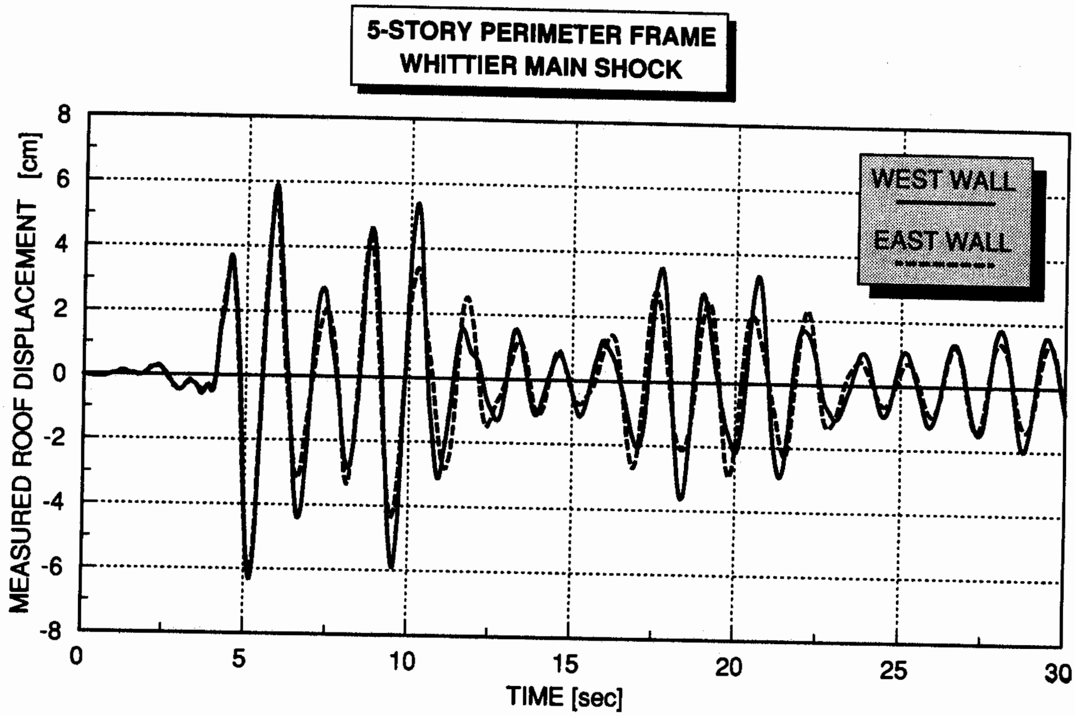


FIG. 3 COMPARISON OF ROOF DISPLACEMENT RESPONSE AT EAST AND WEST WALL, N-S DIRECTION, 5% DAMPING

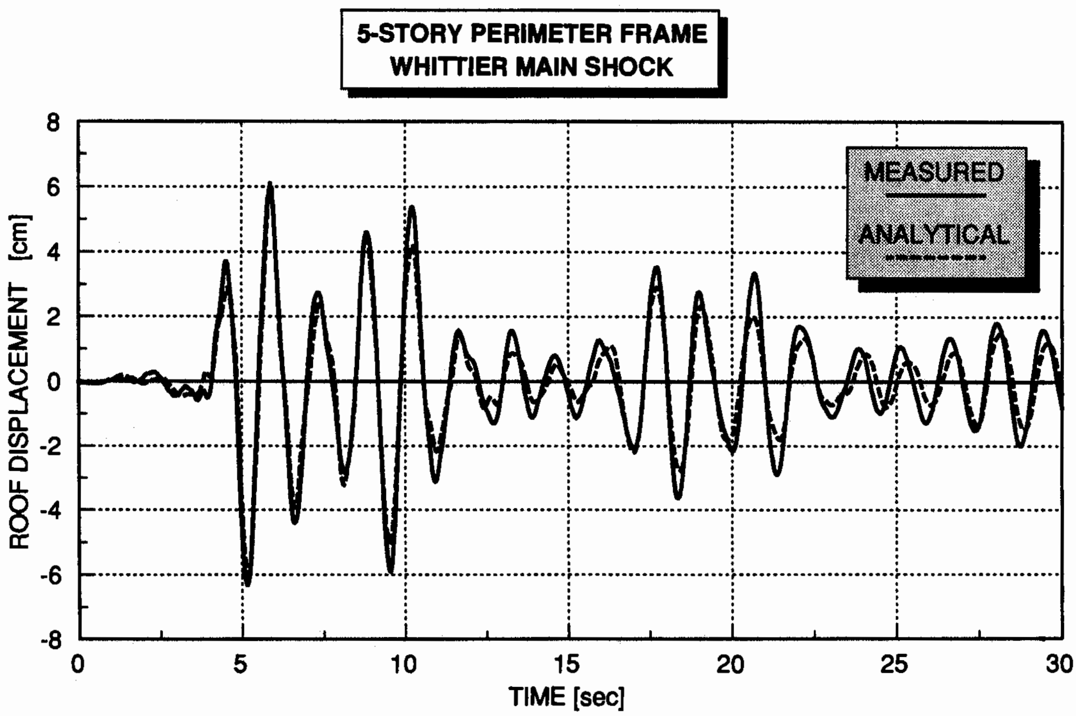


FIG. 4 ROOF DISPLACEMENT RESPONSE AT WEST WALL N-S DIRECTION, 5% DAMPING

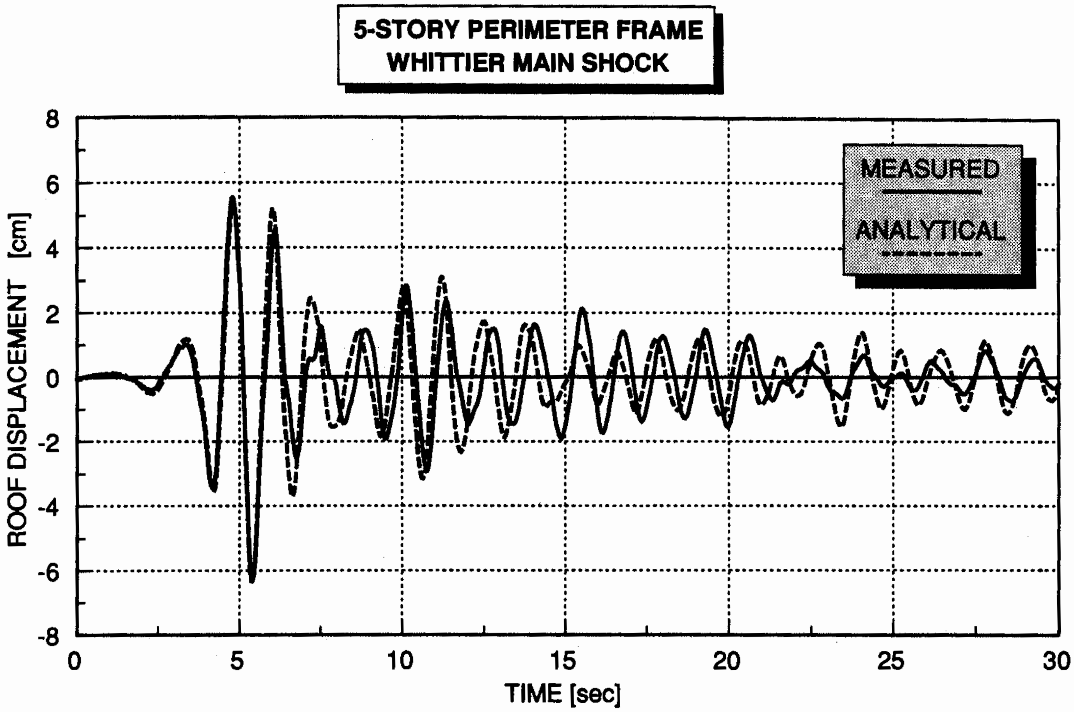


FIG. 5 ROOF DISPLACEMENT RESPONSE AT WEST WALL E-W DIRECTION, 5% DAMPING

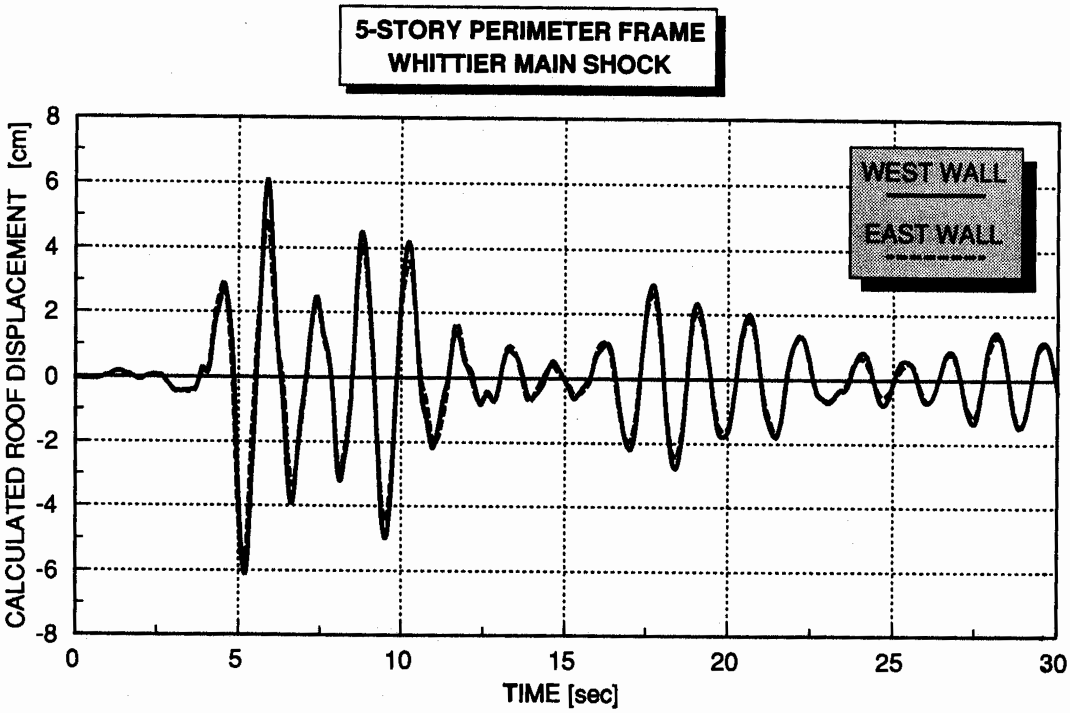
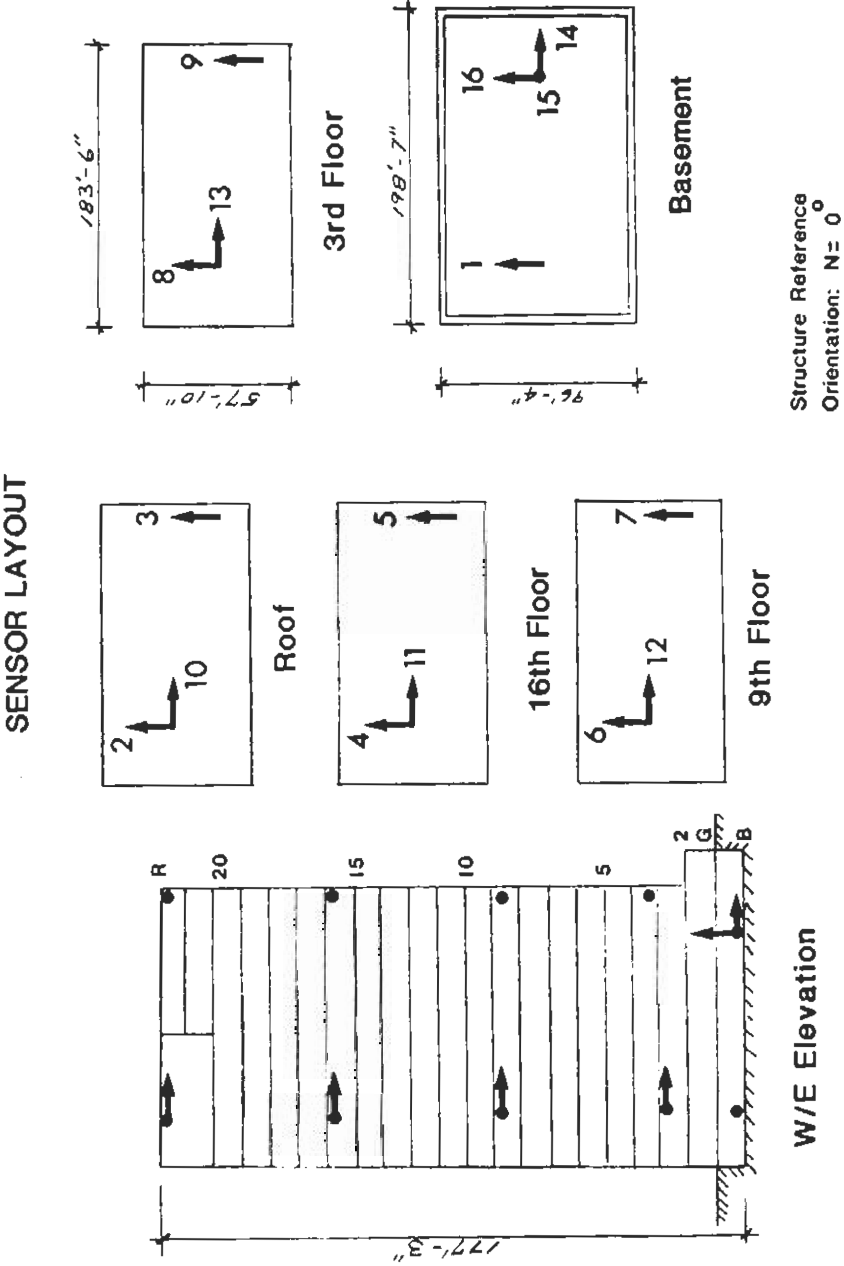


FIG. 6 COMPARISON OF ROOF DISPLACEMENT RESPONSE AT EAST AND WEST WALL, N-S DIRECTION, 5% DAMPING


SENSOR LAYOUT
Roof
16th Floor
9th Floor
3rd Floor
Basement
183'-6"
57'-10"
198'-7"
96'-4"
Structure Reference
Orientation: N= 0°
W/E Elevation
177'-3"
R
20
15
10
5
2
G
B
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

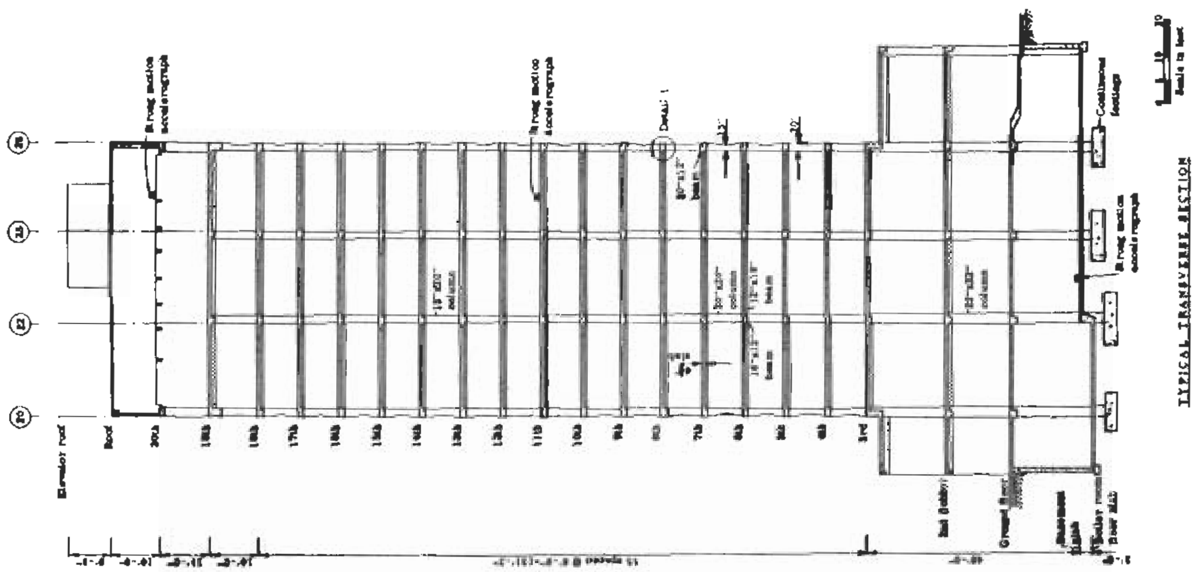

TYPICAL TRANSVERSE SECTION

FIG. 7

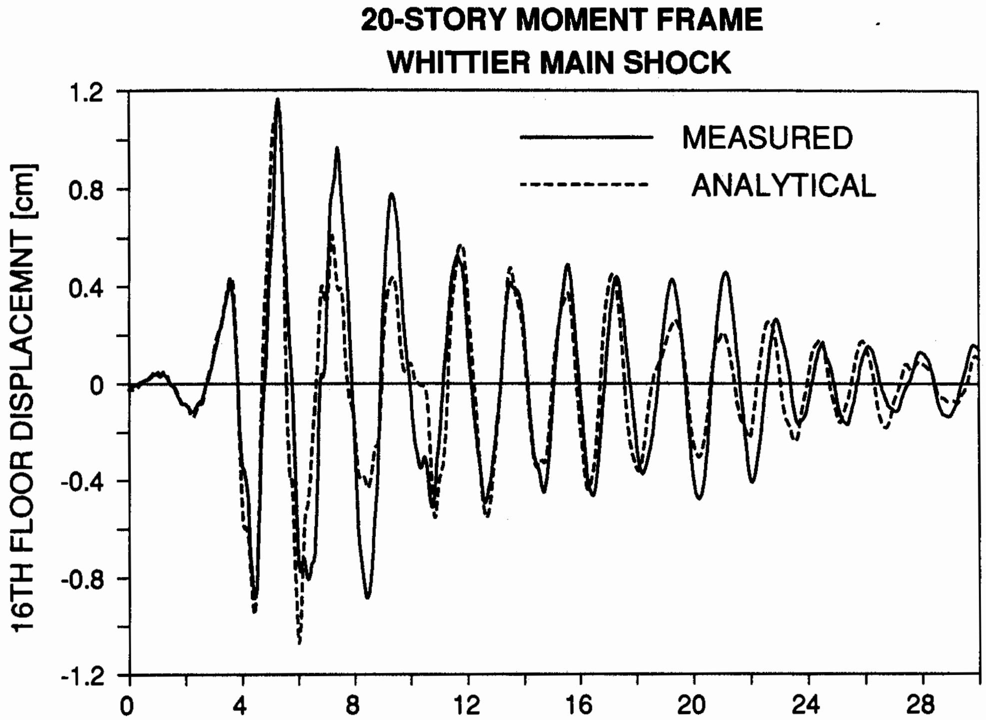


FIG. 8 COMPARISON OF 16 FLOOR DISPLACEMENT RESPONSE

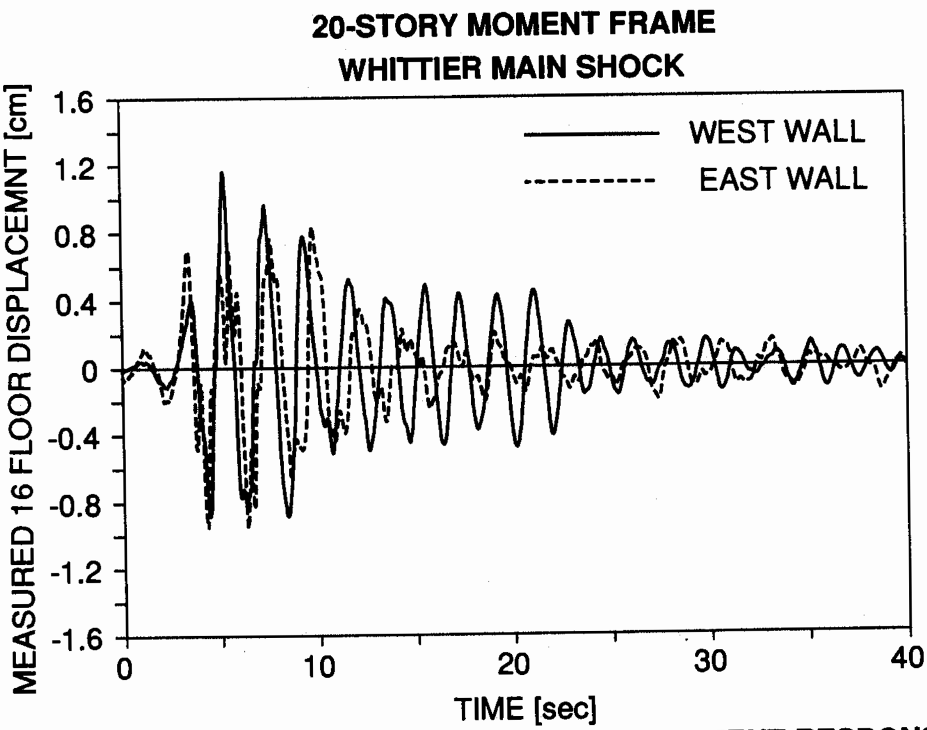


FIG. 9 COMPARISON OF 16 FLOOR DISPLACEMENT RESPONSE AT EAST AND WEST WALL, N-S DIRECTION