SEISMIC RESPONSE AND ANALYTICAL MODELING OF THE CSULA ADMINISTRATION BUILDING SUBJECTED TO THE WHITTIER NARROWS EARTHQUAKE

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Abstract

This study examines the measured seismic response behavior of the CSULA Administration Building during the October 1, 1987 Whittier Narrows Earthquake. The shear forces and interstory drifts calculated from measured seismic responses are compared with UBC design requirements to ascertain the validity of UBC requirements. A three dimensional elastic model was constructed and subjected to the recorded base accelerations to study the correlation of analytical modeling predictions with measured seismic responses.

Introduction

The California State University at Los Angeles (CSULA) Administration Building was subjected to the October 1, 1987 Whittier Narrows Earthquake. The earthquake measured 5.9 on the Richter magnitude scale with an epicentral distance of approximately 9 km. The peak horizontal ground acceleration, as measured at the basement level of the building, was 0.39g.

The eight-story structure, with penthouse and basement levels, is of reinforced concrete construction with composite steel/concrete construction along two transverse frames at the first floor level. The typical floor framing plan, second floor framing plan and the ground floor plan are presented in Figure 1.

The structural system above the second floor level consists of beam and girder floor systems, shear walls and non-ductile moment resisting frames. This system frames into the second floor soffit which transfers loads to the first floor columns. The first floor columns are offset from the perimeter of the building on all sides. The basement level is only partial, so that some columns extend from the first floor to the basement while others stop at the first floor. First floor and basement columns are supported on a foundation of drilled caissons with bell bottoms.

The building was constructed with 4000 psi normal weight concrete at and below the second floor soffit and 4000 psi light weight concrete above the second floor soffit.

Measured Seismic Building Response

The CSULA Administration Building was instrumented with 16 accelerograms to measure seismic building response. Data from these accelerograms is the basis for the discussion herein. The locations of the accelerograms are shown in Figure 2.

The first floor columns are offset from the perimeter of the building on all four sides and the shear walls abruptly stop at the second floor soffit. Large girders in the second floor soffit transfer loads from the shear walls to the first floor columns. In addition, perimeter columns above the second floor soffit, along the north and south perimeters, which help transfer lateral loads in the longitudinal direction, also frame to the columns via transfer girders in the second floor soffit. Hence, the lateral force resisting system is not continuous from the roof to the foundation which leads to the expectation of a soft first story during strong ground shaking.

However, the data obtained during the Whittier Narrows Earthquake indicate that first story did not respond as a typical soft story. The second floor soffit and the first floor columns provided enough stiffness to prevent enormous lateral drifts at the second floor level. The levels of interstory drift are within the limits of the 1988 Uniform Building Code [1].

The absence of a soft first story can be attributed to the flexible frame and shear wall system above the second floor, to the relative stiffness of the second floor soffit and first floor columns, and to the relatively moderate intensity of ground shaking induced by the Whittier Narrows Earthquake. The earthquake was not strong enough to cause inelastic behavior in the structure and consequently unable to produce a soft first story. Despite the lack of soft story behavior during this earthquake, the probability of a soft first story occurring during strong ground shaking is inherently high due to the nature of the building.

The typical shear wall layout tends to promote torsional behavior in the structure. Shear walls are located along the east and west perimeter to resist transverse lateral loads. A shear wall is located in the northwestern quadrant of the building to resist longitudinal lateral loads. The shear wall layout is presented in Figure 1. (Shear walls surrounding stairwells at the east and west perimeter walls are not shown.)

Comparison of recorded displacements at the ground level and at the second floor level indicate that the building experienced torsion during the Whittier Narrows Earthquake. The twisting of the building was limited and the maximum relative displacement between the east and west perimeter walls, at the second floor level, is approximately 0.4 inches. The Whittier Narrows Earthquake was moderately intense and larger magnitude earthquakes may cause larger torsional response which could overstress the first floor columns and induce failure.

In addition, story shear forces were estimated at several time steps from the recorded accelerations. Accelerations at floors 3 to 8 were obtained by linear interpolation of the data from the second floor and the roof. The computed inertia forces were compared with the story shears as calculated per 1988 Uniform Building Code provisions [1] and are presented in Figure 4. It should be noted that the inertia forces only approximate the total lateral force per floor and is intended as a simplified method for estimating story shears.

Based on the measured data, the maximum base shear encountered by the building is approximately 1570 kips and 1070 kips in the transverse and longitudinal directions, respectively. The base shear, as calculated by the UBC seismic provisions, is approximately 1130 kips at working stress levels. The longitudinal base shear calculated from measured data is within the limits of the UBC working stress base shear. On the other hand, the transverse base shear calulated from measured data exceeds the working stress level provided by the UBC seismic design requirements. However, the inclusion of load factors by the UBC to calculate base shears at yield stress levels increases the base shear to approximately 1760 kips. The measured base shears for both directions are within this limit and no inelastic behavior is expected. It should be noted that in its determination of design lateral forces, the UBC assumes inherent inelastic behavior and general overdesign of the structure by the designer. Hence, shear forces calculated from recorded data may be larger than those computed per UBC seismic provisions.

Finally, displacement time-histories were evaluated to further understand the behavior of the CSULA Administration Building during the Whittier Narrows Earthquake. The motion encountered by the building is typical of many structures. At the start of the earthquake, ground accelerations and displacements are small and the building moves in the same direction as the ground without exhibiting considerable deformations. As the earthquake progresses, the ground accelerations and the ground displacements increase. With the increased lateral displacement, the structure is dragged along in the same direction as it tries to keep up with the ground motion. However, when the displacements suddenly change direction, the building is whipped beyond the maximum ground displacement. This behavior is characteristic of higher mode effects and is presented in Figure 5.

Analytical Seismic Building Response

In order to adequately predict the three-dimensional response of the structure, including torsional behavior, a three-dimensional analytical model was constructed and analyzed using SAP90 [2].

The following considerations were made in the development of the three-dimensional analytical model:

1. Fixed support conditions were assumed at the base of the first story columns and/or top of the basement walls. Because no acceleration records are available at this level, accelerations recorded at the basement level are assumed to propogate unaltered to the base of the first story columns. In addition, soil conditions were not available; hence, soil-structure interaction was not considered. The analytical studies support the validity of this assumption.

2. All floor slabs were assumed to be rigid diaphragms. The slabs have a minimum thickness of 6 inches and the floor joists are closely spaced. Again, the validity of this assumption is supported by the analytical studies.

3. Transverse shear walls along the east and west perimeter are modeled as two shear walls, one on each side of the building. Only the shear wall in the northwest quadrant is modelled for the longitudinal direction. Shear walls around stairwells were not considered.

4. A time-history analysis was performed using the recorded ground accelerations as measured at the basement level. Based on the recorded response data, a viscous damping value of 5 percent is used.

5. Uncracked gross cross-sectional properties were used for all structural members.

Computed and measured relative roof displacements for the transverse and longitudinal directions are compared in Figure 3. The seismic building responses calculated from the three-dimensional analytical model agree well with the responses from the recorded data. The calculated fundamental period of vibration is 1.63 and 1.58 seconds in the transverse and longitudinal directions, respectively. This compares well with the measured fundamental period of vibration of approximately 1.6 seconds. In the range of strong ground motion, i.e., the range from approximately 3 to 12 seconds, the calculated displacement response is slightly less than that obtained from measured data. Furthermore, the calculated response from the analytical model exhibits more higher mode effects than the measured response.

In the range of free vibration, i.e., the range from approximately 12 seconds and beyond, the seismic building responses calculated from the analytical model are virtually identical to the responses from the measured data. The similarity in the period of vibration and the amplitude of displacement suggests that the modeling assumptions made above are adequate to simulate actual building behavior.

In addition, base shears were also calculated for the three-dimensional analytical model subjected to the ground motions recorded for the Whittier Narrows Earthquake as measured at the basement of the CSULA Administration Building. The maximum base shears, calculated with the SAP90 computer program, are approximately 970 kips and 2900 kips in the longitudinal and transverse directions, respectively. The base shear computed in the longitudinal direction is similar to that computed from the measured response. However, the base shear calculated in the transverse direction is approximately twice as large as that computed from the measured response. This is not necessarily a modeling error since the calculation of base shear is time dependent and the maximum accelerations per floor may occur at the same moment creating a relatively large base shear. This response requires further investigation.

Conclusions

Based on the study of the CSULA Administration Building, the following conclusions can be made:

1. The structure performed well during the Whittier Narrows Earthquake as evidenced by the minimal damage observed. Since cracking was minimal, it is apparent that the interstory drift requirements per 1988 UBC are adequate. The anticipated soft first story did not occur during the

Whittier Narrows Earthquake, but is likely to occur during larger magnitude earthquakes.

2. Measured story shears were similar to those stipulated by the 1988 UBC design provisions for working stress. All shear forces were within the limitations for yield stress as provided by the UBC. The code assumptions of inherent inelastic behavior and general overdesign appear to be adequate.

3. The seismic building responses predicted by the three-dimensional analytical model, especially the free vibration responses, were in adequate agreement with the recorded seismic responses. The modeling assumptions of fixed base, 5 percent viscous damping, and uncracked sections were adequate in representing the actual structure.

4. Based on analysis of the recorded data and the three-dimensional analytical model, higher mode affects dominate the response of the CSULA Administration Building. Omission of higher modes in the analysis of the structure may be critical.

Further analysis of the three-dimensional model is presently in progress and will be reported on in a future paper.

Acknowledgements

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References

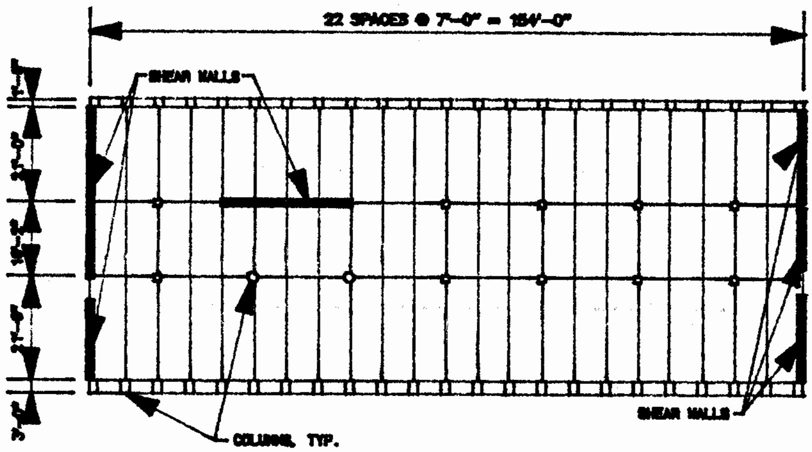
1. Uniform Building Code, 1988

2. E. L. Wilson and A. Habibullah, "SAP90, A Series of Computer Programs for the Static and Dynamic Finite Element Analysis of Structures," 1989.

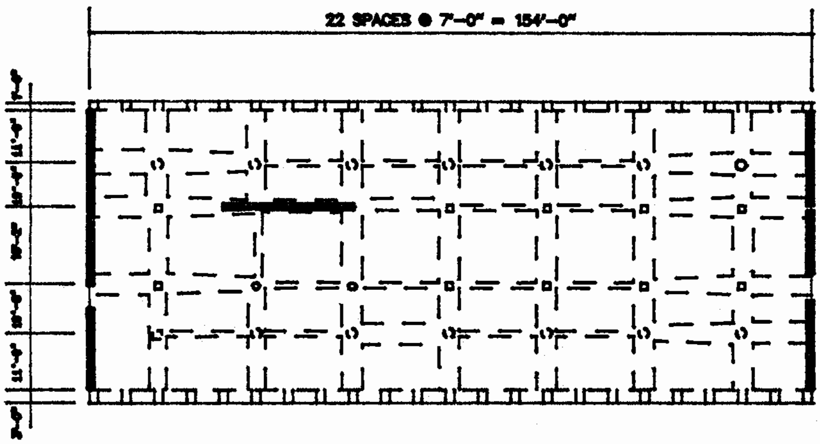
3. "Addition to Administration Building, California State University at Los Angeles," drawings prepared by John Sheffet, structural engineer, and Maynard Lyndon, architect, June 23, 1967.

4. SMIP, "Strong-Motion Data (Vol.2), Los Angeles - CSULA Admin. Building, Channels 1, 3 - 16, Whittier Earthquake of 1 October 1987," CSMIP Serial No. 762.

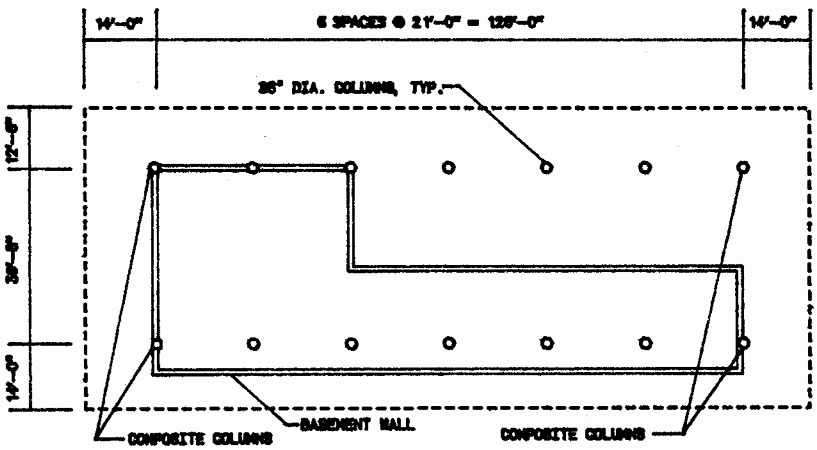
5. Albert C. Martin and Associates, "Preliminary Dynamic Analysis Survey, California State University, Los Angeles, California, Administration Building, Volume 1, Summary Report," August 1988.



Typical Floor Framing Plan (Floors 3 to 8)



Second Floor Framing Plan



Ground Floor Plan

Figure 1

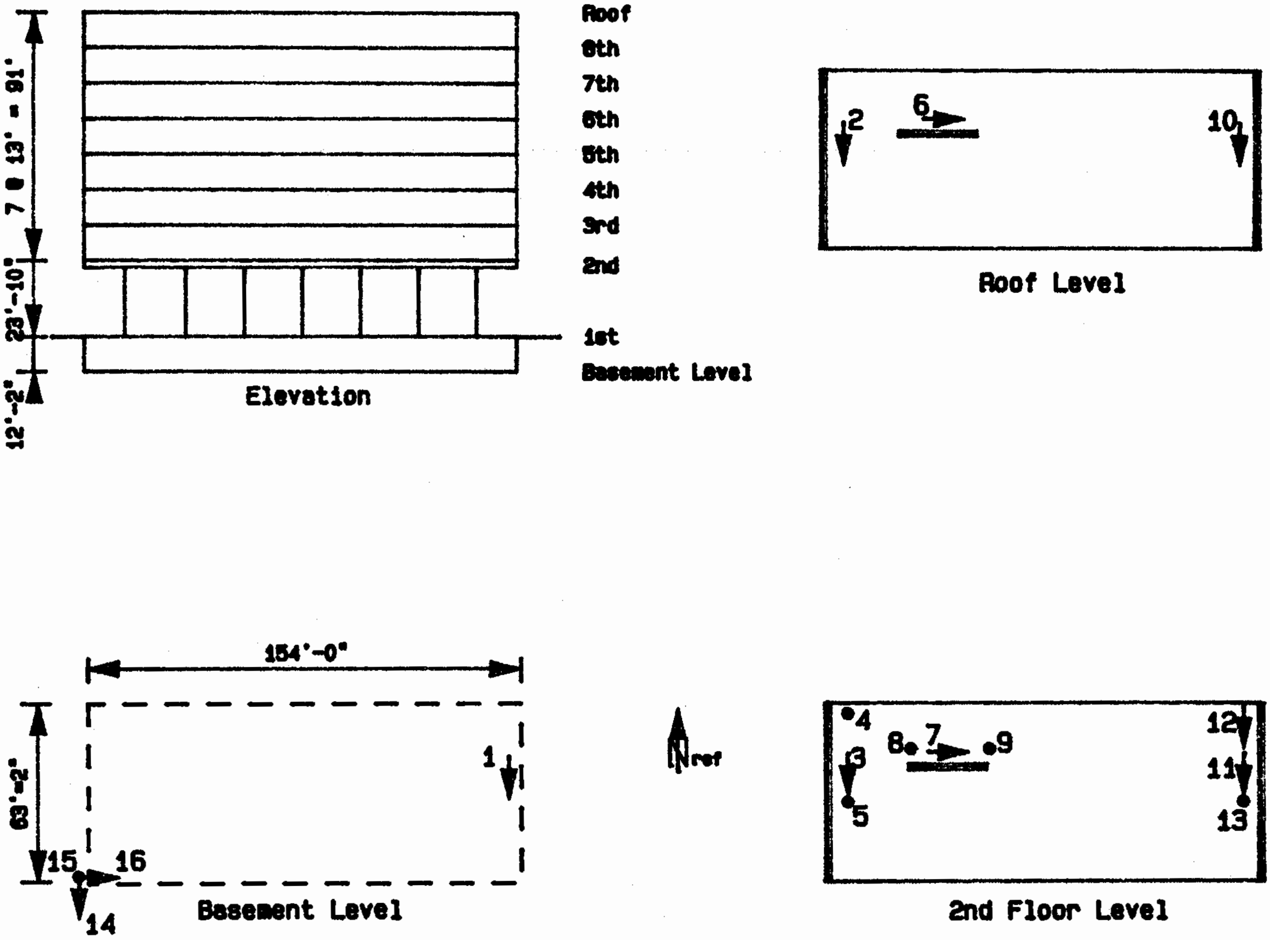


Figure 2 Sensor Layout for CSULA Administration Building
(CSMIP Station No. 24468)

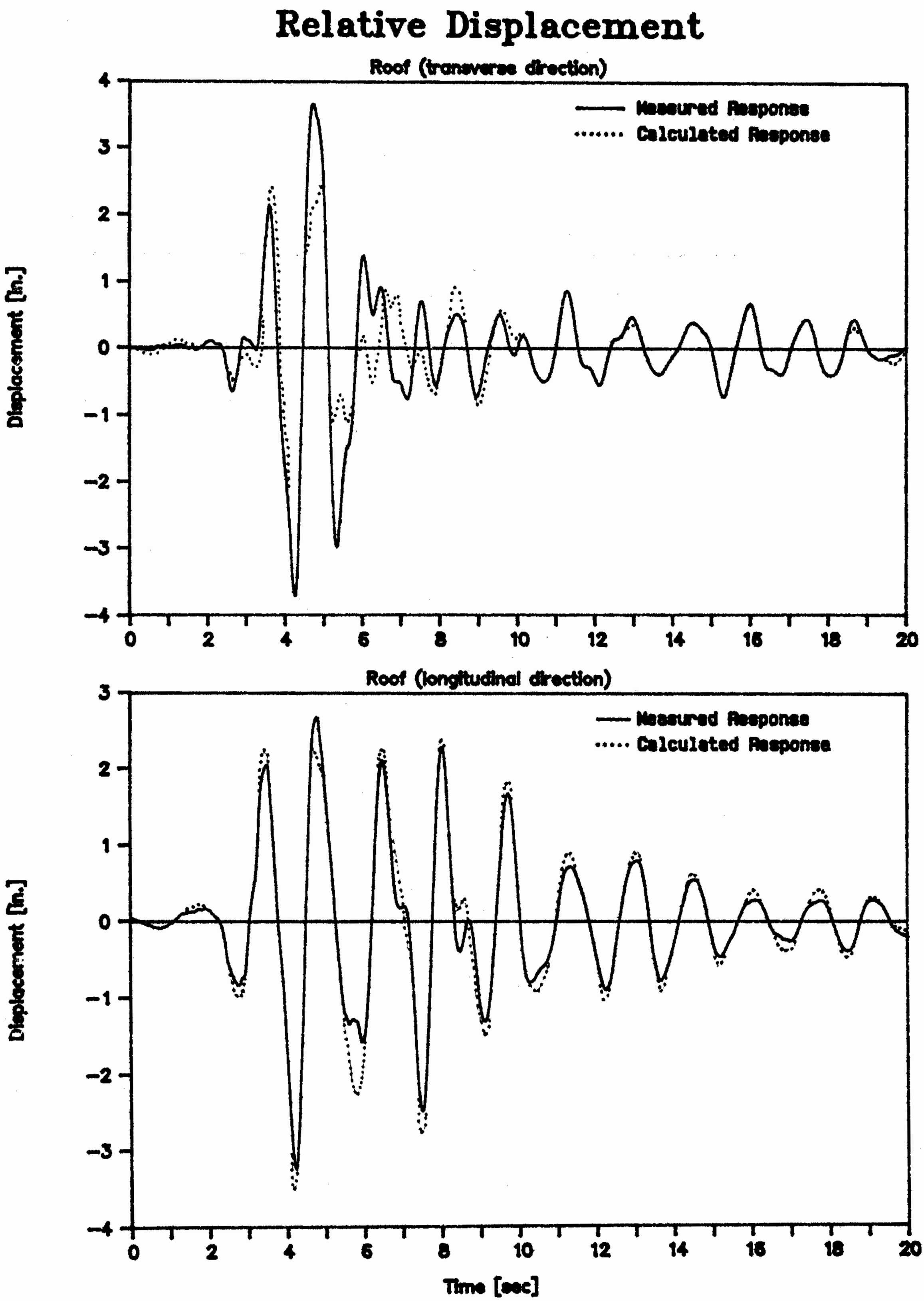
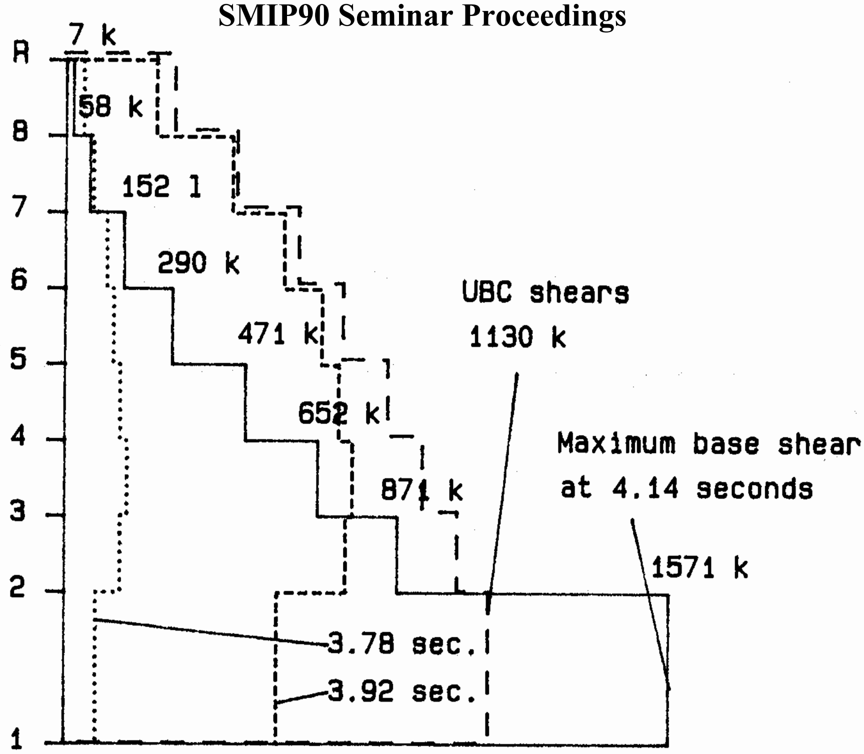
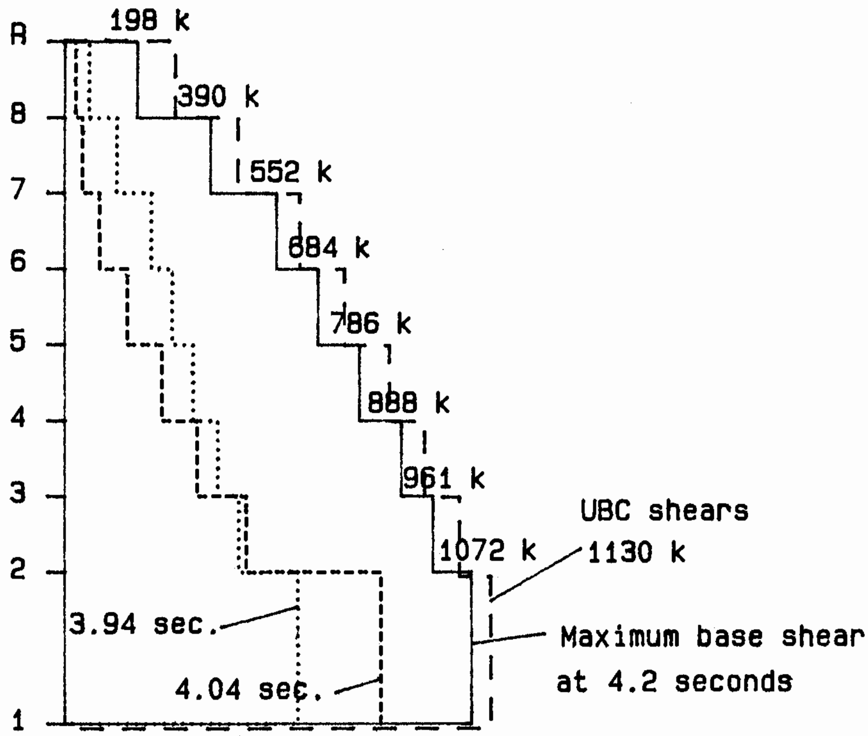


Figure 3 Calculated and Measured Response



Base Shear in the Transverse Direction



Base Shear in the Longitudinal Direction

Figure 4 Story Shears

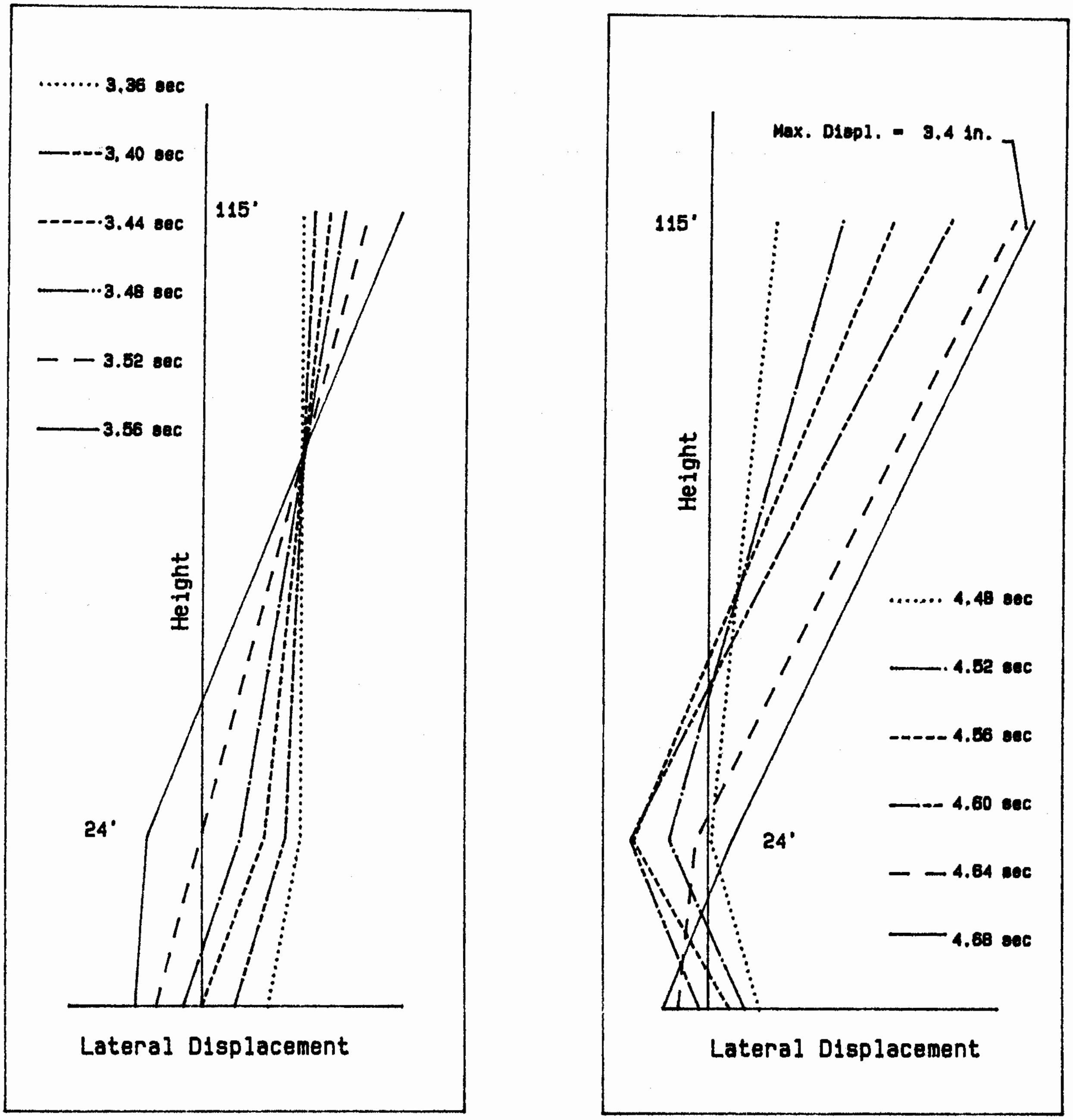


Figure 5 Absolute Displacement Time-Histories