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INVESTIGATION OF DESIGN AND ANALYSIS METHODS FOR STEEL FRAMED BUILDINGS

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ABSTRACT

The Strong Motion Instrumentation Program (SMIP) of the California Department of Mines and Geology has obtained significant records in a number of steel structures. The objective of this research was to conduct comparative studies of the response of two modern steel structures as predicted by current analytical methods and the response recorded by SMIP.

Realistic models of the structures were developed and were subjected to 3-dimensional dynamic analyses. The analyses indicated that proper modeling of connections, and floor diaphragms can lead to accurate predictions of the response. Also, current code procedures predict a period that usually is significantly smaller than actual period of vibration.

INTRODUCTION

While steel frame buildings have generally performed well during past earthquakes, many questions have been raised regarding the realism and adequacy of methods used in their design and analysis. Modern steel buildings, utilizing perimeter moment frames, are generally much more flexible and irregular than those constructed in earlier eras. Their dynamic response is likely to be lightly damped, more sensitive to nonstructural components, and more susceptible to torsional and higher mode vibrations. Thus, conventional modeling and design assumption may not represent actual dynamic response. The main objective of the study summarized herein was to investigate the adequacy and accuracy of design and analysis methods applied to steel structures by conducting comparative studies of recorded earthquake response and the response predicted using available analytical methods.

DESCRIPTION OF THE BUILDINGS UNDER INVESTIGATION

The Strong Motion Instrumentation Program (SMIP) of the California Department of Mines and Geology has obtained significant records in a number of modern steel frame buildings. Two of these buildings, both constructed in the late 1970's exhibit particularly interesting response: an office building in San Jose identified as CSMIP Station No. 57357 and another office building identified as CSMIP Station No. 24370 located in Burbank.

The San Jose Building is a 13 story frame with a nearly square floor plan (Fig. 1). It was designed in 1972 and construction was completed in 1976. The vertical load carrying system consists of a 3-1/2 inch concrete slab over metal deck supported, on steel beams, girders and columns. The lateral load resisting system consists of a strong perimeter moment resisting frame with tapered girders, and four interior moment frames in each orthogonal direction. An extra bay is provided along the west and south sides of the building to accommodate stairs, elevators and extra offices. Framing irregularities in this region and differences in architectural treatments on these sides of the building resulted in small torsional eccentricities. Nonetheless, the building would qualify as a torsionally regular building according to the provisions of the 1988 Uniform Building Code [3]. In addition to the recent Loma Prieta earthquake, the building was subjected to the 1984 Morgan Hill earthquake and the 1986 Mt. Lewis earthquake [4]. The building's response is characterized as being both particularly severe and long (Fig. 2) Twenty-two accelerometers were used to record the building motion. Maximum accelerations at the base of the building for both of these latter events was 4%g, while that for the Loma Prieta event was 11%g. Only the response during the 1984 and 1986 events will be examined herein.

The Burbank Building is a six story building with almost symmetric geometry. The building was excited during the 1987 Whittier Narrows earthquake [5]. Details of the steel structure of the building are shown in Figures 3 and 4. The structure of the building consists of hot rolled wide flange columns and beams supporting the floor steel deck and concrete slabs. The structure has a perimeter moment frame to resist the tributary gravity and total lateral load while the internal columns are designed to carry only their tributary gravity load. The beam-to-column connections in the perimeter frame are rigid, and all other connections are designed to act as simple connections and to carry shear only. During the Whittier Narrows earthquake the building was subjected to ground motions with 22% peak acceleration.

SCOPE AND METHODOLOGY

A coordinated investigation of these two buildings was conducted. The focus of study was on the characterization of the basic behavioral parameters as well as on the assessment and improvement of the current design procedures and analysis methods. The project involved the following specific tasks:

1. The engineering design parameters (periods, mode shapes, and damping) and important response characteristics were identified from an interpretation of recorded response.

2. Conventional and "realistic" models of the structures were developed using information obtained from the available structural drawings as well as from the actual survey of the buildings. In the San Jose building, the flexibility of the beam-column joints was modeled as was the architectural exterior finishes. In the Burbank building, the connections of perimeter frame was modeled as rigid whereas the interior connection were modeled as semi-rigid. Also, in this building the interaction of floor diaphragm and steel girders were modeled as partially composite to reflect the actual existing conditions.

3. The structural models were subjected to ground motions recorded at the site of the buildings. Elastic three dimensional analyses were used in the case of the San Jose building whereas inelastic, three dimensional, dynamic analyses were performed on the Burbank building. Using the results of the analyses, comparative studies of the measured and calculated results were conducted.

4. Recommendations are being formulated to improve the accuracy of the calculated response in steel structures.

SUMMARY OF RESULTS

Complete results of the studies are presented in Refs. 1 and 2. A summary of important results is given below:

San Jose Building. -- For the San Jose building, the recorded motions indicate that the building undergoes strongly coupled three-dimensional response. The first two modes are predominantly translational with periods of 2.2 and 2.1 secs. The third mode is predominantly torsional with a period near 1.7 secs. The closeness of these modal periods and the small eccentricities of the building resulted in a strong modal beating phenomenon in the building and coupled lateral- torsional vibrations. The fourth and fifth modes are predominantly translational with periods between 0.7 and 0.6 seconds. The 1985 and 1988 UBC would estimate the predominant translational period of the building to be 1.3 and 1.77 seconds, respectively, indicating the flexibility of the frames.

Drifts computed for the building using a three dimensional elastic analysis and considering 1988 UBC lateral forces are, however, less than 0.0018 times the story height; a value substantially less than permitted by the code. Similarly, the analyses indicated that the building's members would not generally be stressed in excess of code permitted values when subjected to 1988 UBC lateral forces. Thus, current code provisions would indicate that the building was generally quite satisfactory. Nonetheless, during the Mt. Lewis earthquake (with 4%g peak ground acceleration) the base shear actually developed in the building is estimated to be more than 400% greater than the code prescribed value and the drifts are nearly 300% greater than allowed by code.

The records indicate that the building has little damping. However, the closeness of the modal periods made precise interpretation of the records difficult. Damping in the N-S direction ranges between 3-4% and between 2-3% in the E-W direction.

Figures 2 and 5 show some of the response of the building for the two earthquakes considered. The response is notable in its 80 second length, with nearly 30 cycles of intense structural motion. The strong portion of the earthquake was however less than 10 seconds. The intensity of this response is believed to be due to (1) the modal coupling and beating that substantially increased the amplitude and apparent duration of the motion; (2) the apparent resonance of the building with the predominant period of the ground; and (3) the relatively light damping in the system. As a result, the ground accelerations at the site were amplified in the building by 500 to 700 %.

Several analytical models were developed for the building. The standard one was a bare frame model having rigid joints. The period obtained for this model was 1.93 seconds. However, the computed roof displacement was only half of the recorded value for the Morgan Hill earthquake. By modeling the flexibility of the panel zones, the periods and displacements were greatly improved. By further adjusting the mass distribution to reflect actual conditions and by incorporating the additional contributions of the architectural finishes to lateral stiffness, the slight slab contribution to beam stiffness virtually identical results could be obtained (See Fig. 2).

The analyses give some insight into the apparent causes of the severity and duration of the response. In Figure 5 shows response to the Mt. Lewis earthquake. In part 1a of the figure, the girders, and four interior moment frames in each orthogonal direction. An extra bay is provided along the west and south sides of the building to accommodate stairs, elevators and extra offices. Framing irregularities in this region and differences in architectural treatments on these sides of the building resulted in small torsional eccentricities. Nonetheless, the building would qualify as a torsionally regular building according to the provisions of the 1988 Uniform Building Code [3]. In addition to the recent Loma Prieta earthquake, the building was subjected to the 1984 Morgan Hill earthquake and the 1986 Mt. Lewis earthquake [4]. The building's response is characterized as being both particularly severe and long (Fig. 2) Twenty-two accelerometers were used to record the building motion. Maximum accelerations at the base of the building for both of these latter events was 4%g, while that for the Loma Prieta event was 11%g. Only the response during the 1984 and 1986 events will be examined herein.

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The analyses give some insight into the apparent causes of the severity and duration of the response. In Figure 5 shows response to the Mt. Lewis earthquake. In part 1a of the figure, the response of the analytical model with 1% viscous damping is compared with the recorded motion. Parts 1b and 1c of the figure show that the first and second modes individually have lightly attenuated response after about 30 seconds of motion. However, the two modes go in and out of phase resulting in constructive and destructive interference which produces a large dip in the combined response at 30 sec. and an increase in response up to time 60 secs. To further confirm that this response is not associated with the input motions, the analysis was run with zero input following time equal 40 secs. Also, an analysis was performed considering 5% viscous damping. In this case (part 2a of Fig. 5), the response of the individual modes attenuate so quickly that virtually no beating under free vibration can occur and little significant motion occurs after about 25 seconds.

Analyses also indicate that many of the members were probably not overstressed during the Morgan Hill and Mt. Lewis events. While only a few columns were loaded beyond yield, most of the perimeter beams, especially in the lower levels, exceeded their plastic moment capacity, in several cases by as much as 40%. While these numbers are not by themselves cause for concern, they indicate that the building is particularly susceptible to structural damage and that the potential response of the building to earthquakes larger than the 4%g events considered herein should be examined with care. The motion of the building during the Loma Prieta earthquake, where motions three times larger than the Morgan Hill event were recorded, indicated substantially reduced amplifications and torsional contributions. It may be conjectured that the increased yielding in the structure separated the modes, thereby reducing modal coupling effects, increasing effective damping and moving the apparent periods of the structure away from the predominant period of the ground. Precise determination must await processing of the records and complete inelastic analysis of the building.

Burbank Building. -- In the Burbank building, the recorded period of vibration in seconds for the first five modes were 1.32 (N-S), 1.30 (E-W), 0.83 (torsional), 0.44 (N- S), 0.42 (E-W) and 0.28 (N-S). Using provisions of UBC(1988) the first period of vibration for both N-S and E-W direction would be calculated as 0.95 second. In both directions, the measured as well as calculated periods were much longer than the predictions of the current Uniform Building Code. The longer period of this building is attributed to more flexibility of the perimeter frame steel structure relative to typical moment frames.

Using the Half-Power Method, the modal damping for the Burbank building was calculated to be 4% and 3% of critical damping for the first and second modes respectively.

Figure 6 shows drift response of third floor and roof in N-S and E-W directions. Generally, the response in N-S direction was greater than the response in the E-W direction. Also, the response showed significant higher mode effects in both E-W and N-S directions.

In conducting dynamic analysis of Burbank building, initially a conventional model of the structure was built. In this model mass was established by using weights prescribed by UBC. The connections were modeled as fixed or simple according to the design assumptions. In addition, since floors are non- composite, they were not included in the modeling. This model which represents common modeling procedures used today, was subjected to base accelerations recorded at the foundation level of the building. The response predicted by this model was significantly different from recorded response.

By studying the response of the conventional model and also by surveying the actual building, it was realized that the actual condition of the building differs substantially from code prescribed conditions. The mass in the building was much less than dead load of the code. The floor decks were connected to the girder by limited number of shear studs resulting in partial composite action. Connections that were assumed to be simple, actually were more or less semi-rigid.

Column base connections are conventionally modeled as fixed or pin, however, the actual base connections were either close to fixed or close to semi-rigid joints. In the refined model of the structure, the realistic conditions were modeled properly.

The three-dimensional realistic model of the steel structure was subjected to base excitations using the Inelastic computer program FACTS. The response of the realistic model was very close to the response recorded by the SMIP. Samples of the predicted and measured drift response are shown in Figure 6 which indicate reasonably close correlation between predicted and recorded responses.

CONCLUSIONS

The results of this study identify the importance of having well instrumented building records. Several interesting phenomena not accounted for in building codes and design practice were identified.

For the San Jose building it was shown that nearly symmetric, lightly damped structures with closely spaced modes can experience severely amplified motion as a result of coupled torsional-translational response. The analysis also confirmed that modeling needed to account for the flexibility of the beam-column panel zones. Current codes were unable to predict the severity of the observed responses of this building. Additional studies of this building in the inelastic range using the Loma Prieta records are needed as are improved instrumentation in the form of free field accelerometers.

The study of Burbank building indicated that in order to obtain realistic response from a dynamic analysis, the major elements of the structure such as floor diaphragms and connections should be modeled properly and more realistically than conventionally done in design offices. Also, realistic value of mass should be used in dynamic analyses and not the nominal values prescribed by building design codes.

ACKNOWLEDGMENTS

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3. Uniform Building Code., 1988 Edition, International Conference of Building Officials, Whittier, California, May 1988.

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5. "CSMIP Strong-Motion Records from the Whittier, California Earthquake of 1 October 1987", Report OSMS 87-05, California Department of Conservation, Division of Mines and Geology, Office of Strong Motion Studies, October 1987.

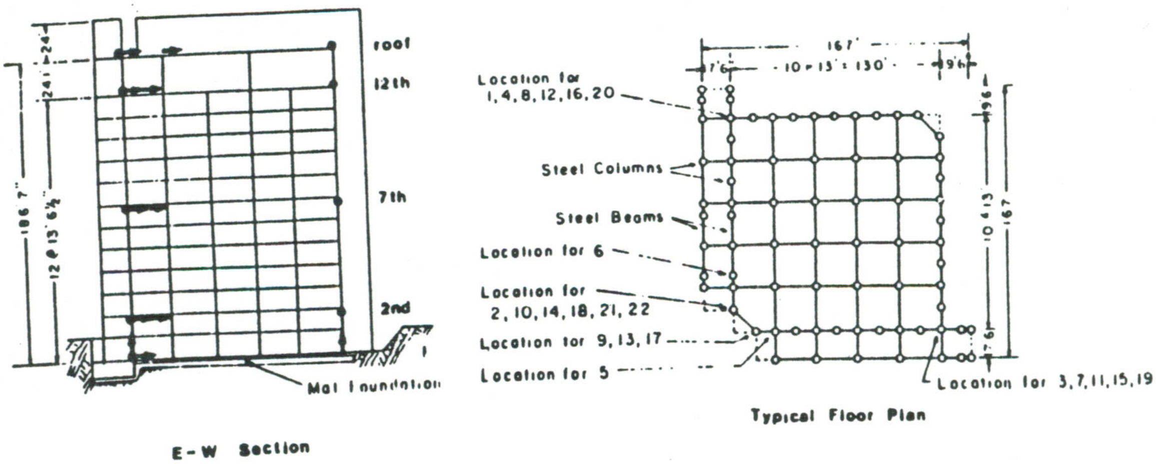


Figure 1 - San Jose Office Building. Plan and sensor layout.

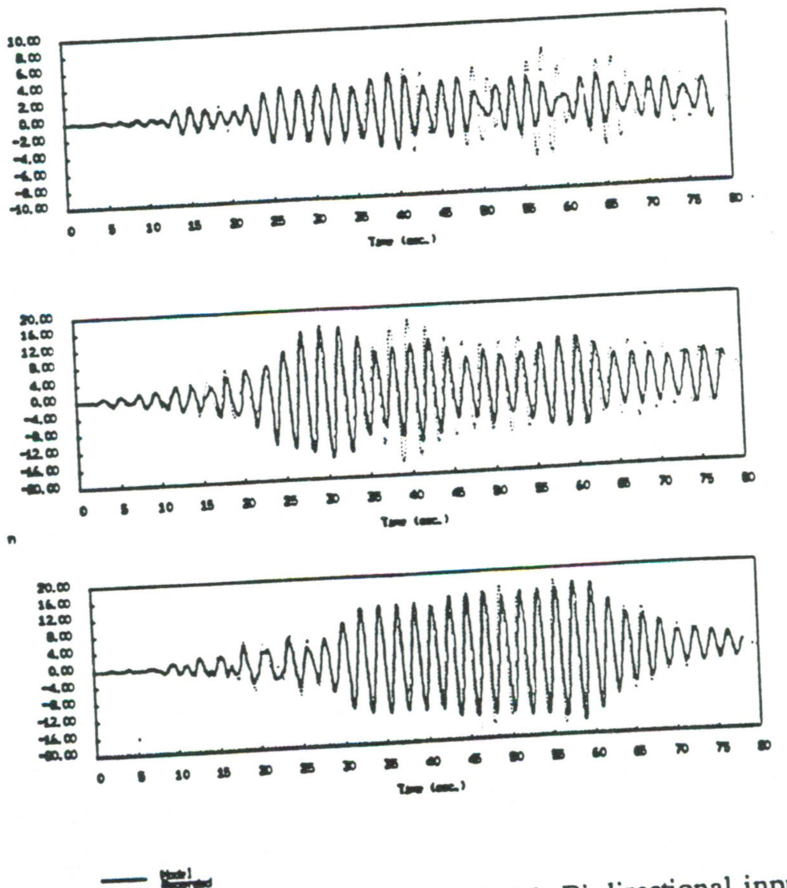
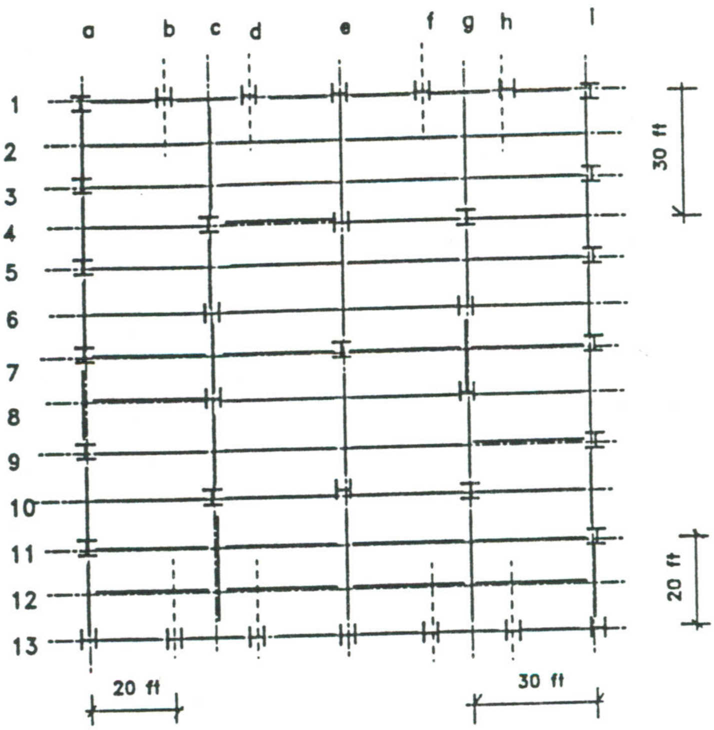
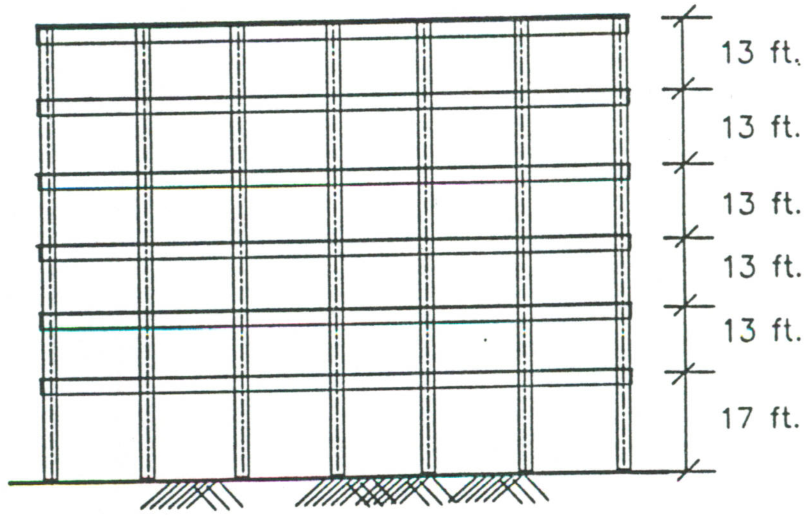


Figure 2 - San Jose Office Building. Best fit Model. Bi-directional input. Morgan Hill Earthquake. Twelfth floor motion. a) Derived torsion (SW-NW corner relative displacement). b) EW relative displacement, SW corner. c) NS relative displacement, SW corner.

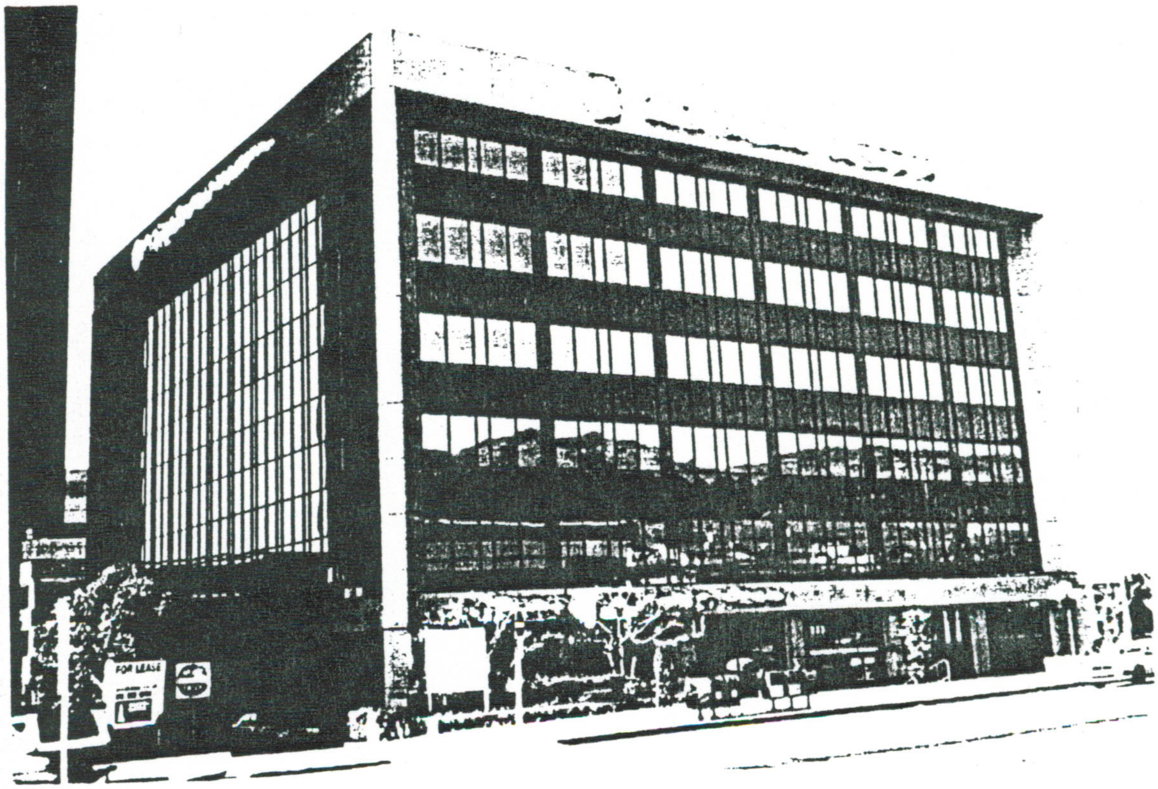


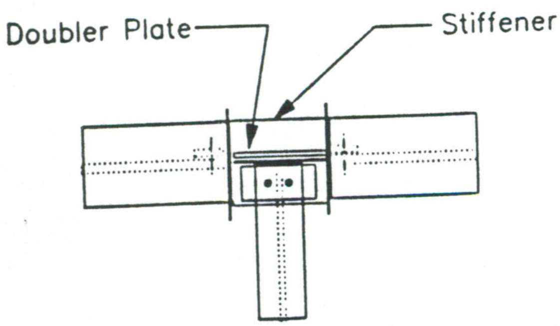
TYPICAL FRAMING PLAN



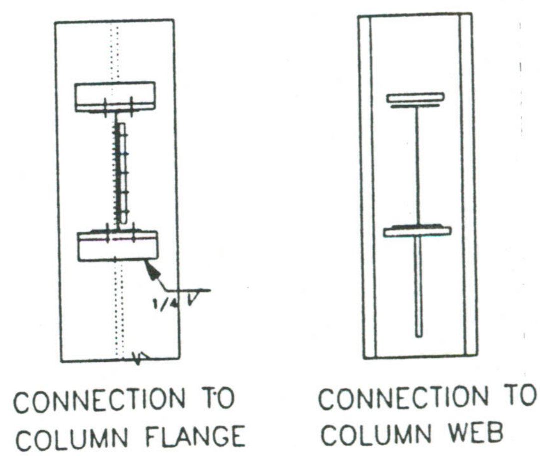
Elevation

Figure 3. Office Building in Burbank





RIGID FRAME MOMENT CONNECTION



SHEAR CONNECTIONS

Figure 4. Ofice Building in Burbank and its Connection Details

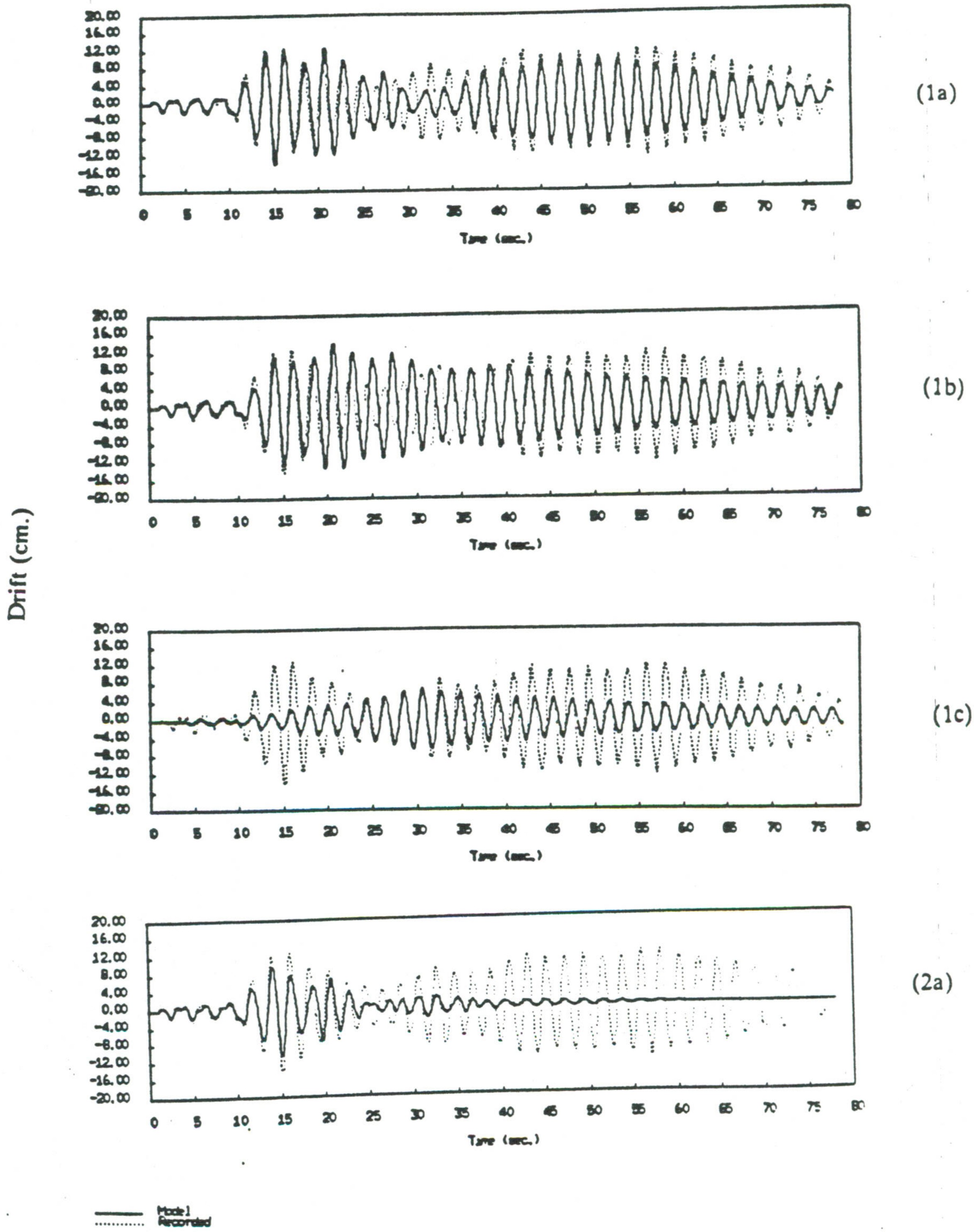


Figure 5. San Jose Office Building. Best fit Model. Bi-directional input. Mt. Lewis Earthquake. Input ground motion 0-40 seconds. Twelfth floor EW relative displacement, SW corner. 1a) First three modes, low damping model (1%) 1b) First mode. 1c) Second mode. 2) First three modes, increased damping model (5%)

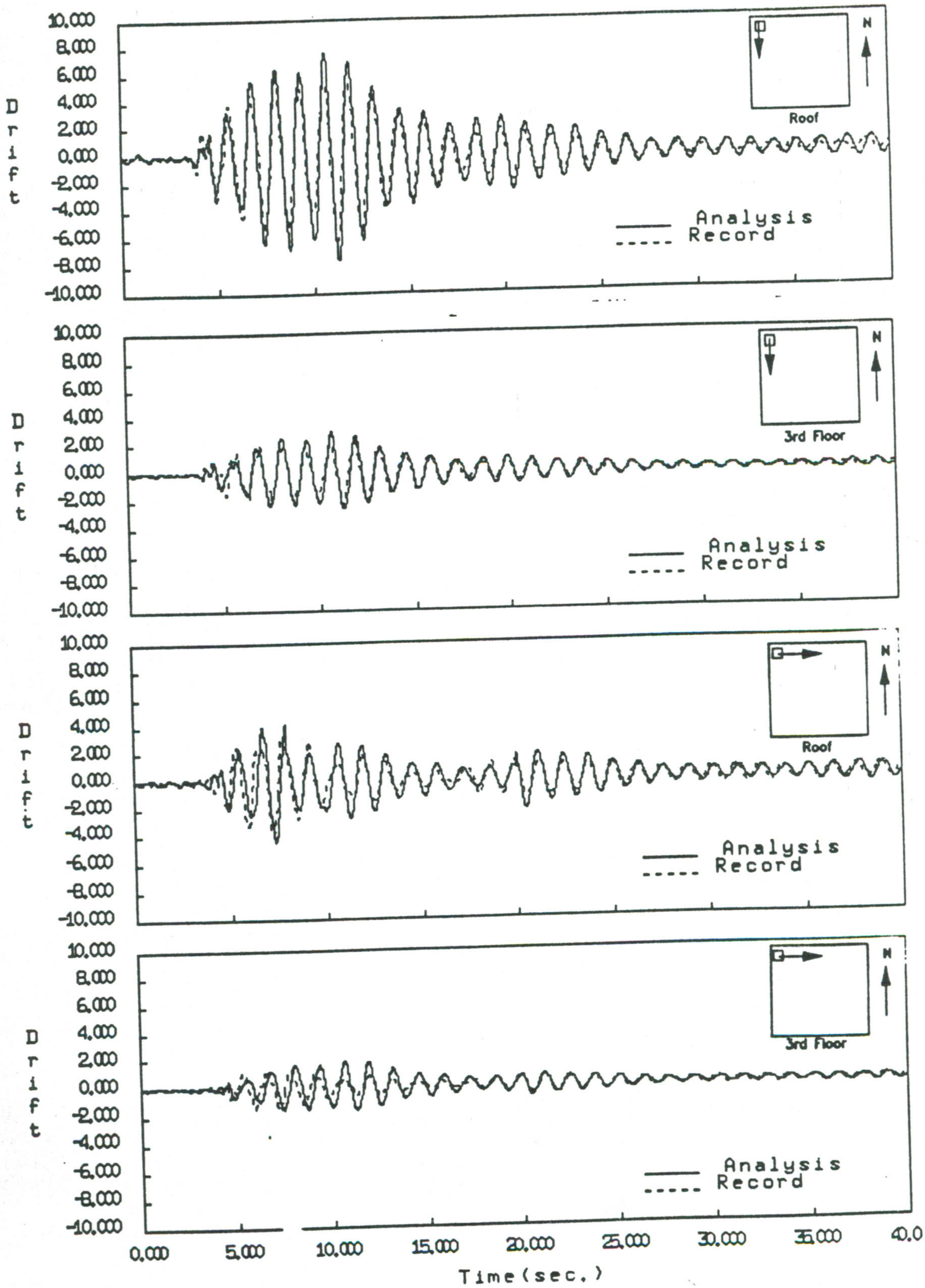


Figure 6. Correlation of Predicted and Recorded Responses for Burbank Building