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Assessment of UBC seismic design provisions using recorded building motions

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ABSTRACT: Recorded motions in two 10-story reinforced concrete buildings from three different earthquakes are analyzed in order to assess UBC seismic design provisions. In this, a proven system identification method (MODE-ID) has been applied to each set of recorded motions and the corresponding normal modes of vibration have been identified. These modal parameters were used to compute earthquake-induced forces, shears, moments, and drifts. Example results presented in this paper from the 1989 Loma Prieta earthquake motions show that the computed forces and moments exceed the corresponding UBC design values, but the computed interstory drift is substantially less than the code allowable. This comparison in conjunction with the lack of damage to the buildings during the earthquake has indicated the importance of the interstory drift as a measure of the severity of the seismic demand on buildings.

1 INTRODUCTION

The seismic design of buildings throughout the United States is based on the seismic design provisions of the Uniform Building Code (UBC). These provisions include procedures for calculating minimum design values of various parameters such as the base shear forces, overturning moments, interstory drifts, torsional moments and interstory shear forces. When used with the UBC design and detailing provisions that enable the building to develop adequate levels of strength, stiffness, and ductility, these design values are intended to prevent collapse of the building during strong ground shaking.

An excellent basis for quantitatively assessing the UBC design values is through an analysis of recorded earthquake motions in buildings. These recorded motions can be used to compute the above earthquake-induced forces, moments and drifts, which can then be compared with the design values. This comparison, in conjunction with the observed performance of the building during an earthquake, can provide an important frame of reference for evaluation and further improvement of building code seismic design provisions.

2 OBJECTIVE AND SCOPE

The two 10-story reinforced concrete buildings studied are the Great Western Bank (GWB) Building, and the Town Park Towers (TPT) Building in San Jose California. In each building, earthquake motions were recorded during the 1984 Morgan Hill (M_s=6.2), 1987 Mt. Lewis (M_s=5.5), and 1989 Loma Prieta (M_s=7.1) earthquakes. As shown by the peak acceleration values presented in Tables 1 and 2, these recorded motions encompass a wide range of intensity levels. This table

Table 1. Peak horizontal accelerations (g) recorded at GWB building.

Level	Base		Roof	
Direction	NS	EW	NS	EW
Morgan Hill (1984)	0.058	0.059	0.181	0.221
Mt. Lewis (1986)	0.029	0.036	0.078	0.077
Loma Prieta (1989)	0.090	0.110	0.250	0.380

Table 2. Peak horizontal accelerations (g) recorded at TPT building.

Level	Base		Roof	
Direction	NS	EW	NS	EW
Morgan Hill (1984)	0.056	0.061	0.216	0.133
Mt. Lewis (1986)	0.030	0.036	0.119	0.082
Loma Prieta (1989)	0.093	0.118	0.365	0.232

shows that in both buildings the weakest motions were obtained during the Mt. Lewis earthquake, and the strongest during the Loma Prieta earthquake.

The MODE-ID system identification methodology was used to identify the modal parameters for the normal modes excited for both buildings during each earthquake (Beck 1978, Werner et. al. 1987). These parameters were used to compute the base shear forces, overturning moments, interstory drifts, and interstory shears for the buildings. Finally these computed values were compared with the corresponding design values from the 1991 UBC and the values used in the original design of the buildings. Together with the observed seismic performance of the buildings, these comparisons were used to assess the seismic design provisions of the current code.

An advantage of this study of recorded building motions is that it is based on a formal system identification method, which has been shown to lead to more reliable estimates of modal parameters (Beck & Beck 1985). Also, since the strong motion records are typically obtained at only a limited number of floor levels, a more reliable interpolation of measured motion on a mode-by-mode basis is obtained. This is particularly advantageous for buildings where the higher mode effects are significant and hence is difficult to interpolate the recorded floor accelerations.

3 BUILDING DESCRIPTIONS

Both the GWB and TPT Buildings are within 10 to 15 miles from the epicenters of the three earthquakes (Figure 1). The geometry and instrument locations for each building are shown in Figures 2 and 3.

The lateral force-resisting system for the GWB Building consists of moment-resisting frames with exterior shear walls in the transverse (east-west) direction, and moment-resisting frames in the longitudinal (north-south) direction. The floor and roof diaphragm consists of a one-way slab and joist construction. The foundation at the building's basement is a five-feet thick reinforced concrete mat. The aboveground floors and roof were constructed of lightweight concrete, and regular weight concrete was used at all other locations in the building.

The lateral force-resisting system for the TPT building consists of shear walls in both transverse (East-West), and longitudinal (North-South) directions. Some of the walls in the North-South direction do not continue above the sixth floor. The building's floor and roof diaphragms consist of one-way post-tensioned slabs. The building is founded on precast, prestressed concrete piles beneath all shear walls. Lightweight concrete was used for all superstructure elements.

The soil under both buildings consists of deep alluvial deposits which we have classified as Type S2 as defined in the 1991 UBC.

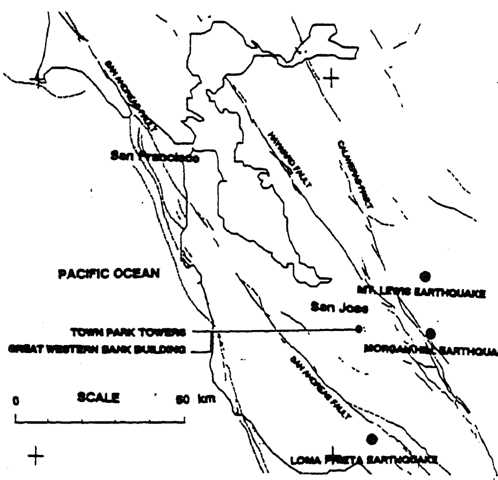

Figure 1. Location of Buildings.

4 METHODOLOGY

In this investigation, a formal system identification procedure named MODE-ID was applied to the strong motion records from both buildings. MODE-ID is a modal identification procedure for a system with an arbitrary configuration and classical normal modes, and can be applied to an array of corresponding response and excitation measurements (Werner et. al. 1987). A measure-of-fit parameter J, defined as the ratio of the mean-square output error between measured and model motions to the mean-square output from the measured motions is minimized using the data from all output channels over a prescribed time interval. The pseudostatic response is included by identifying or specifying an influence matrix that relates the pseudostatic response at the output channels to input (support) motions. Within a Bayesian probability framework, the parameters estimated by MODE-ID can be viewed as most probable values based on the given data (Beck 1990).

This paper describes modal parameters identified by MODE-ID for the "rocking-base" models of each building, in which the vertical motions recorded at the base of the buildings together with the horizontal motions measured at the roof and floors are considered to be output motions to be matched by the model. Input motions for the model consist of the horizontal motions recorded at the base of the buildings. In this case, the effects of soil-structure interaction from rocking are included in the modes of vibration, and the identified equivalent viscous damping factors represent the combined effects of internal damping of the structural and non-structural elements, and radiation and soil

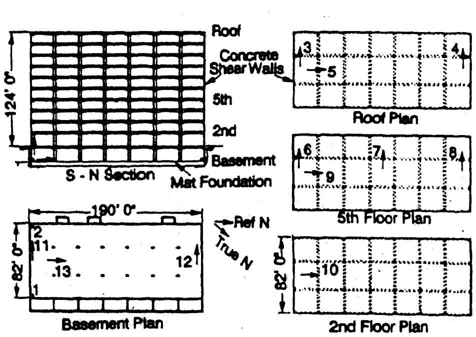

Figure 2. Instrument Location, GWB Building.

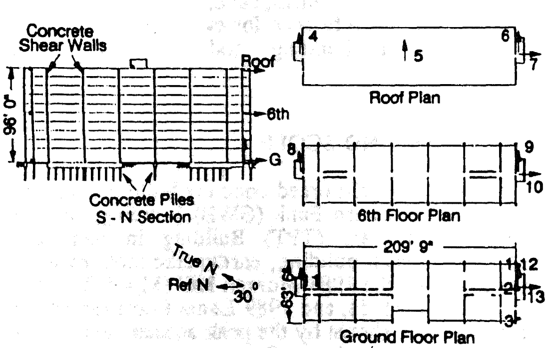

Figure 3. Instrument Location, TPT Building.

damping from the rocking of the building.

In addition, modal parameters were obtained for two different cases, namely "time-invariant" and "time-segmented" models. The former correspond to parameters identified by utilizing the entire duration of the recorded motion, and for the latter, the parameters are obtained by dividing the duration of shaking into discrete segments and identifying modal parameters for each segment. The resulting variations in the modal parameters from one time segment to another serve as a measure of the nonlinear response characteristics of the buildings during each earthquake. At this stage, a rather coarse segmenting of the recorded motions was implemented for illustrating general trends regarding the difference between the modal properties identified from a time-invariant model. The time-segmented models were identified only for the Morgan Hill and Loma Prieta Earthquakes, since only these motions were sufficiently strong to produce substantial nonlinearities.

Since only the mode shape components at the output channels are identified by MODE-ID, "missing" components were computed by smooth interpolation of the identified values. The resulting identified and interpolated modal parameters for each earthquake were then used to compute time histories of displacements and accelerations at each floor of the two buildings. These quantities, together with the mass distribution obtained from structural drawings, were then used to compute time histories of base shear forces, story shear forces, interstory drifts, and overturning moments.

5 MODAL PARAMETERS

The modal periods and damping ratios for the significant modes of the two buildings identified for the time-invariant and time-segmented models are shown in Tables 3 to 6. The adequacy of the identified building models can be seen from the relatively low values of J and by visual comparisons of measured and computed motions at selected accelerometer locations (Figures 4 and 5). The mode shapes identified from the Loma Prieta Earthquake motions for both the GWB and TPT buildings are shown in Figure 6, and are very similar to the corresponding mode shapes identified from the other earthquakes. In addition, the fundamental modes of vibration in each direction for both buildings exhibit very little torsion or out-of-plane motions. These modes, together with the pseudostatic response, have dominated the response of the buildings for all three earthquakes. One interesting result is the large in-plane diaphragm deformations in the second transverse mode for the GWB Building. However, this mode had only a

Table 3. Time invariant model for TPT building

Earthquake	Morgan Hill		Mt. Lewis		Loma Prieta	
Mode Description	T (sec)	D (%)	T (sec)	D (%)	T (sec)	D (%)
1st Trans. (NS)	0.649	5.9	0.633	3.6	0.725	5.8
1st Trans. (EW)	0.426	5.1	0.413	3.7	0.433	5.3
2nd Trans. (NS)	0.180	10.4	0.181	9.2	0.194	16.0
Measure of fit J	0.1445		0.0569		0.0854	

Table 4. Time invariant model for GWB building

Earthquake	Morgan Hill		Mt. Lewis		Loma Prieta	
Mode Description	T (sec)	D (%)	T (sec)	D (%)	T (sec)	D (%)
1st Trans. (NS)	0.914	4.0	0.911	2.9	1.008	4.4
1st Trans. (EW)	0.608	6.9	0.613	2.8	0.745	9.0
2nd Trans. (NS)	0.267	4.5	0.285	3.9	0.310	5.9
2nd Trans. (EW)	0.214	3.7	0.223	3.5	0.241	5.3
1st Torsional	0.372	3.5	0.392	3.0	0.435	11.9
Measure of fit J	0.1445		0.0569		0.0854	

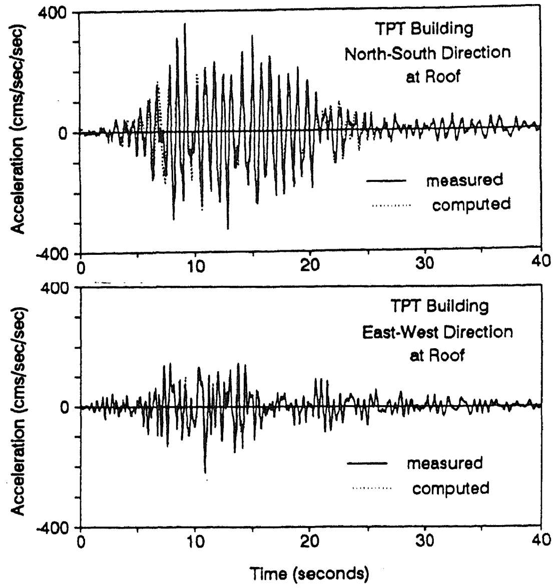

Figure 4. Comparison of Measured vs. Recorded Motions, TPT Building.

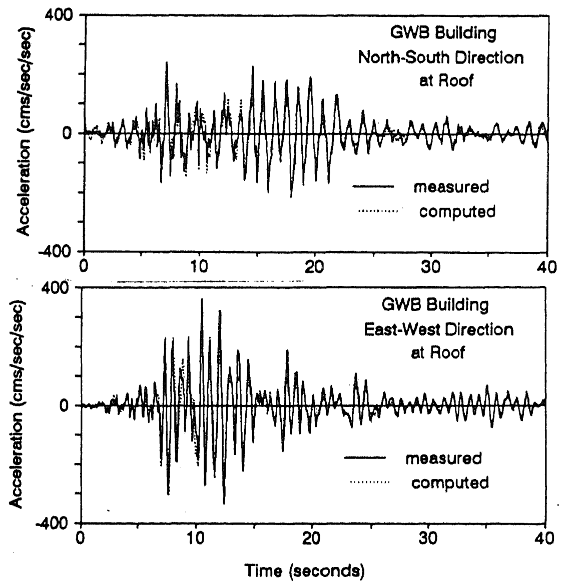

Figure 5. Comparison of Measured vs. Recorded Motions, GWB Building.

small contribution to the total response of the building during each of the three earthquakes.

Comparison of the modal parameters identified for both buildings during the three earthquakes suggest that the natural periods and damping ratios are not solely related to the intensities of shaking; i.e. the periods and damping ratios of the significant modes of vibration do not systematically increase from low intensity shaking (Mt. Lewis Earthquake) to higher intensity shaking (Loma Prieta). Instead, Tables 3 to 6 show that the time-invariant damping values identified for the fundamental modes of the TPT Building from the moderate strength Morgan Hill Earthquake are similar to the damping values identified from the much stronger intensity Loma Prieta Earthquake. In addition these tables show that the natural periods identified for the fundamental modes for the GWB Building from the relatively weak Mt. Lewis Earthquake to be similar to the values obtained from the Morgan Hill Earthquake motions.

A possible explanation for these observed trends may be obtained by examining the time-segmented model results in combination with the sequence of occurrence of the three earthquakes. As shown in Tables 5 and 6, the natural periods of vibration from the last time-segment of each earthquake are now closer in value to the initial segment of the following earthquake if some allowance is made for "healing" or recovery of building stiffness between each earthquake. Similarly, the close values of the damping ratio obtained from the time-invariant model of the TPT Building from Morgan Hill and much stronger Loma Prieta Earthquake could be attributed to the fact that the Morgan Hill Earthquake, being the first of the three earthquakes, may have altered the dynamic characteristics of the "virgin" building more than what might ordinarily be expected solely on the basis of the strength of shaking. These trends are further discussed in more detail in Werner, Nisar, and Beck (1992).

6 COMPARISON WITH CODE

Figures 7 and 8 provides a comparison of the base shear forces computed in each building from the Loma Prieta Earthquake motions, with both the 1991 UBC and the original design values. It shows that the computed base shear forces for the two buildings in each direction exceed both sets of code values. This trend is similar for overturning moments at the base and for the interstory shear values. However, although the code values were exceeded, the buildings did not suffer any damage, which is consistent with the very low values of interstory drift as shown in Figures 9 and 10. It is noted that the time-invariant model was used for the computation of the above response quantities, since a good fit to the measured data was obtained from this model (Figures 4 and 5). The time-segmented models did improve the fit, but not to a degree that would significantly affect the values of these response quantities.

A comparison of natural period values identified for the predominant modes of vibration from the three earthquakes with the 1991 UBC "Method A" period and the period used in the design of the buildings is shown in Table 7. This shows a large variation between the code values and the actual computed values in most cases. Comparison of damping values from the time-invariant model with the 5% damping value used in the 1991 UBC shows that the computed damping values ranged from as low as 3% to as high as 9%. In this it is important to note that the damping values identified from MODE-ID are lower-bound estimates of the total damping of the soil-structure system for the two buildings, as these values do not include the radiation and material damping contributions from the translation of the foundation, although they do include the contributions from rocking of the base.

7 CONCLUDING COMMENTS

The apparent good performance of both buildings during the three earthquakes shows that they had sufficient seismic resistance for these levels of shaking. Low values of interstory drift suggest that the buildings did not suffer strong non-linear behavior. Since the Loma Prieta Earthquake led to only moderately strong levels of shaking, it is conceivable that for structures comparable to the GWB and TPT Buildings, the 1991 UBC values of lateral forces and overturning moments could fall well below the values of these quantities that might be induced by the stronger and longer duration motions from a larger earthquake. This, in turn, could result in larger ductility and reserve strength demands for the structure, and the structure will have to be particularly well designed and detailed in order to accommodate these increased demands. The comparison of the computed response quantities with the code values in conjunction with the observed performance of the buildings emphasizes the importance of interstory drift as a measure of the severity of the seismic demands on the structure. The requirements for limiting interstory drifts in the 1991 UBC may be very important in preventing serious structural and non-structural damage in buildings. This is also suggested by other researchers.

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Table 5. Time segmented model for rocking base case.

Earthquake	Time Segment (sec)	Measure of fit J	Longitudinal (NS) Direction First Mode T (sec)	D (%)	Second Mode T (sec)	D (%)	Transverse (EW) Direction First Mode T (sec)	D (%)	Second Mode T (sec)	D (%)	First Torsional Mode T (sec)	D (%)
Morgan Hill (1984 M=6.2)	0−11	0.0628	0.807	3.1	0.263	4.0	0.587	1.9	0.214	3.5	0.369	3.0
	11−17	0.0203	0.829	2.7	0.270	3.4	0.595	2.6	0.215	3.1	0.375	2.8
	17−34	0.0313	0.919	3.4	0.287	4.9	0.631	4.9	0.225	2.0	0.393	4.5
	34−40	0.0190	0.93	4.6	0.289	3.2	0.609	2.8	0.214	3.6	−	−
Mt. Lewis (1986 M=5.5)	0−40	0.0296	0.911	2.9	0.285	3.9	0.613	2.8	0.223	3.5	0.392	3.0
Loma Prieta (1989 M=7.1)	0−7	0.0493	0.926	3.6	0.296	3.2	0.633	4.4	0.226	3.4	0.389	4.4
	7−22	0.0734	1.01	3.6	0.314	4.8	0.751	7.6	0.242	5.1	−	−
	22−30	0.0243	1.048	3.4	0.329	4.3	0.743	5.8	0.252	5.6	0.473	10.2
	30−40	0.0224	1.045	2.3	0.326	4.0	0.739	4.0	0.256	4.0	0.477	5.9

Table 6. Time segmented model for TPT building, rocking base case

Earthquake	Time Segment (sec)	Measure of fit J	Longitudinal (NS) Direction First Mode T (sec)	D (%)	Second Mode T (sec)	D (%)	Transverse (EW) First Mode T (sec)	D (%)
Morgan Hill (1984 M=6.2)	0−11	0.0690	0.581	3.8	0.173	8.2	0.413	4.7
	11−16	0.0277	0.621	5.0	0.175	2.4	0.426	5.3
	16−30	0.0156	0.649	4.4	0.182	7.3	0.435	5.1
	30−40	0.0368	0.645	4.4	−	−	0.426	5.1
Mt. Lewis (1986 M=5.5)	0−40	0.0569	0.633	3.6	0.181	9.2	0.413	3.7
Loma Prieta (1989 M=7.1)	0−5	0.0469	0.625	4.3	0.195	7.9	0.412	4.5
	5−10	0.0386	0.685	5.3	0.179	16.6	0.429	5.0
	10−15	0.0146	0.725	4.3	0.189	0.7	0.448	4.7
	15−40	0.02398	0.746	3.8	0.216	8.7	0.433	3.8

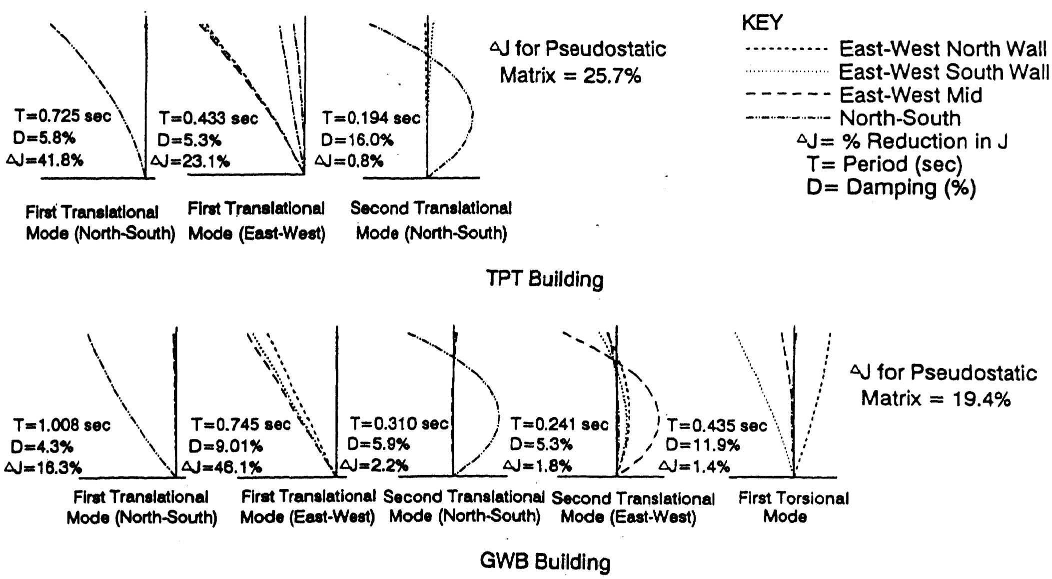

Figure 6. Mode Shape plots for the buildings.

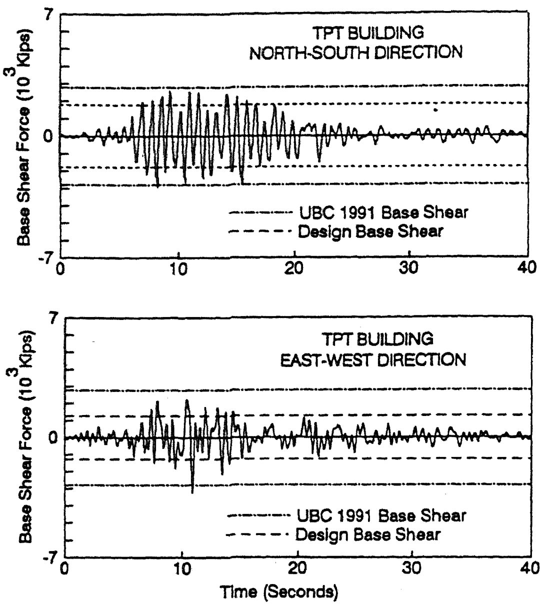

Figure 7 Base Shear Forces, TPT Building.

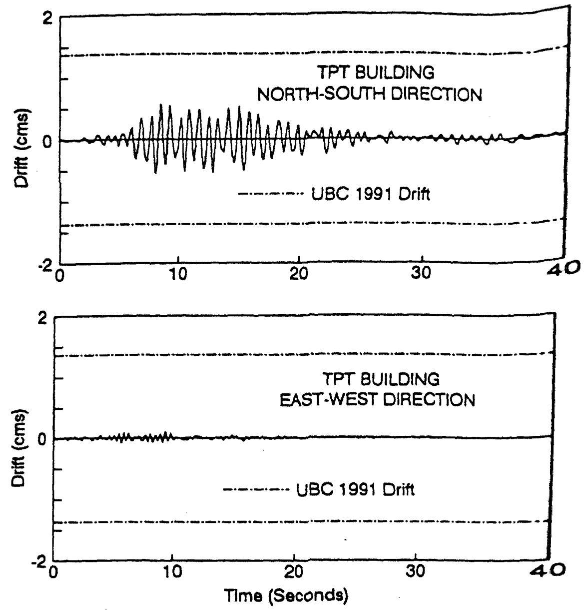

Figure 9 Interstory Drift for Top Story, TPT Building.

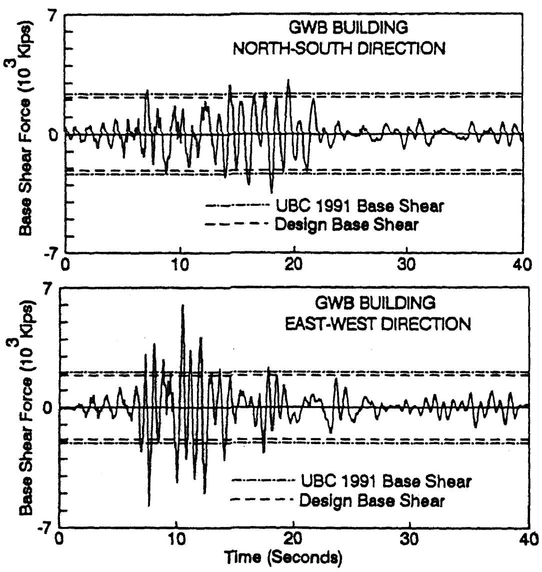

Figure 8 Base Shear Forces, GWB Building.

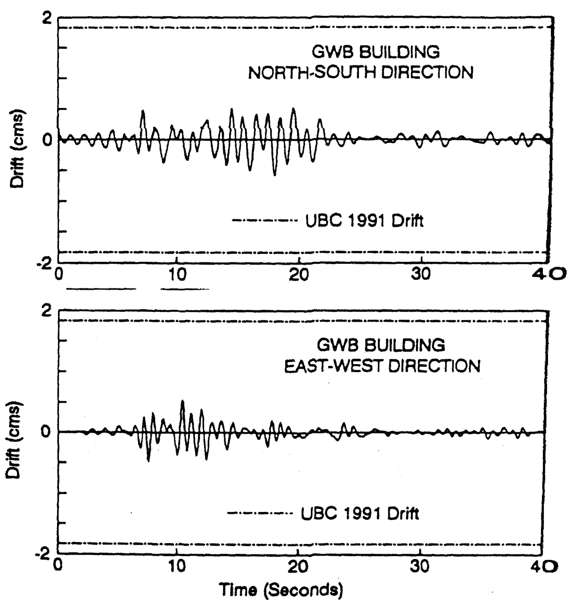

Figure 10 Interstory Drift for 2nd. Story, TPT Building.

Table 7. Comparison of Natural period with code values

	GWB building		TPT building	
	NS	EW	NS	EW
Morgan Hill	0.914	0.608	0.649	0.426
Mt. Lewis	0.911	0.613	0.633	0.413
Loma Prieta	1.008	0.745	0.725	0.433
UBC 1991	0.743	0.743	0.610	0.610
Design	0.450	0.690	0.330	0.600