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Measured and analytical response of a pile supported building

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ABSTRACT: The accuracies of simple soil-foundation-structure models are evaluated using recorded strong motion data obtained during the Loma Prieta earthquake. A thirty story moment resisting frame pile supported structure is utilized as an example. The building was insturmented by the US Geological Survey strong motion program and recorded 21 channels of acceleration time history of building response and 6 channels of free-field accelerations. The soil-foundation-structure model is presented, modal and time history analysis of this model is compared to its measured counterparts.

1. INTRODUCTION

This study consists of evaluating the validity of simplified soil-foundation-structure interaction models that are typically employed during the design process. For this purpose the analytical response of a pile supported building is compared to its measured response. The analytical and measured correlations are based on strong-motion data recorded on pile-supported buildings during the Loma Prieta earthquake, California in 1989.

A 30-story ductile moment resisting frame structure with a 1.5 meter thick concrete mat supported by 828 14"x14" concrete piles located in Emeryville, California is selected as the example structure for this study (Figure 1). The building was instrumented by the USGS strong-motion instrumentation program and that recorded 24 channels of acceleration during the Loma Prieta earthquake [Celebi, 1990]. The objectives of this paper are to present a) the simplified integrated model of the soil, foundation and the superstructure, and b) analytical responses and correlations with the measured counterparts. The primary objective of this paper is to assess the validity of commonly accepted simplifying assumptions in evaluating the dynamic response of buildings during the design process.

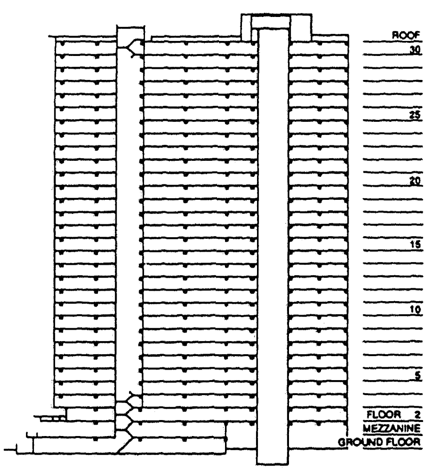
1.1 Site, Building and Foundation:

The building site is on Christie Avenue in Emeryville, California. The San Andreas fault zone, which is located approximately 25 km west of the site, is the predominant active fault in the San Francisco Bay Area.

Figure 1. Pacific Park Plaza Building

Woodward-Clyde (1985) Consultants conducted the site suitability analysis using twenty one soil borings. These borings were drilled to depths in the range of 6.7 meters to 38.5 meters. The laboratory tests reported included moisture content, dry density, unconfined compressive strength and grain-size analyses. The site is located in an area which was developed by placing fill over a former tideland of the San Francisco Bay. For improved seismic performance of the site, the medium dense sand fill in the tower area and the garage area was densified by the vibroflotation method.

The building is 30 stories at a total height of 95 meters (312 feet) and contains 583 condominium units. The building plan is axisymmetric with three wings of 120 degrees apart and a central core. The central core contains two elevator shafts and each wing contains a stairwell. A five-level parking structure is non-structurally attached to the West Wing of the building. The building structure is reinforced concrete ductile moment resisting space frame. The analysis presented here only deals with the building structure and the influence of the parking structure is neglegted. The building also includes structural walls up to the second floor level within the wings and the central core. The plan and elevation of the building structure are shown in Figures 2(a) and 2(b).



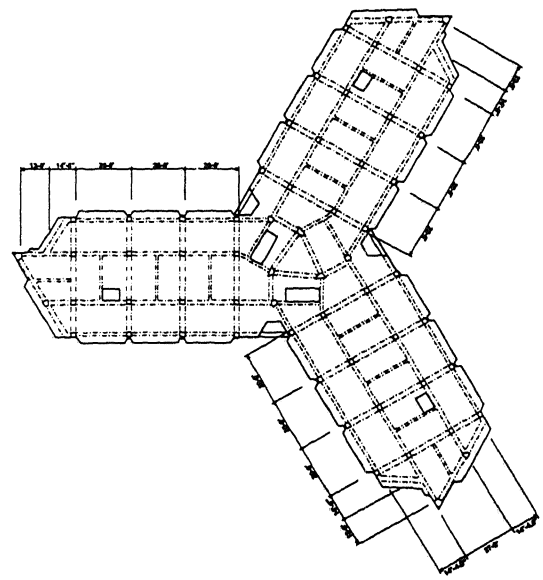


Figure 2. Elevation and Plan of Pacific Park Plaza

1.2 Recorded Motions:

The strong-motion instrument array installed at this site and within the building was triggered at the time of the Loma Prieta earthquake. The building was instrumented at four levels: ground, 13th, 21st and 30th floors and the instrument array recorded 21 channels of strong-motion data during the earthquake. The strong-motion instrumentation scheme and orientation within the building are shown in Figure 3. Five strong-motion instruments were placed on 13th, 21st and 30th floors measuring the horizontal accelerations. The ground floor contains six devices, two for horizontal and four vertical acceleration measurements. The strong-motion devices measuring the vertical component of motion were placed to assess the foundation rocking by comparing the phase and amplitude differences between the devices located at the extremities. The recorded vertical motion maximum amplitudes were between .05g to .06g. The peak horizontal ground acceleration was recorded 0.39 g at the 30th floor and 0.22g at the ground floor.

Tri-axial free-field motions were recorded at two locations. The free-field north instrument recorded a peak acceleration of 0.232 g in the 260 degree direction and the free-field south records showed a peak acceleration of 0.260 g in the 350 direction. The shelter for the free-field north instrument is located approximately 190 ft north west from the center of the condominium tower, and the free-field south motions were recorded about 480 ft south west from the center of the building [Kagawa,1992].

2. SOIL-FOUNDATION-STRUCTURE MODEL

The foundation mat is supported by 828 concrete piles, and the geometrical shape of the mat is rather unique. Therefore, a foundation model with any desired degree of complexity may be constructed and tested for performance evaluation. In this paper, however, results of

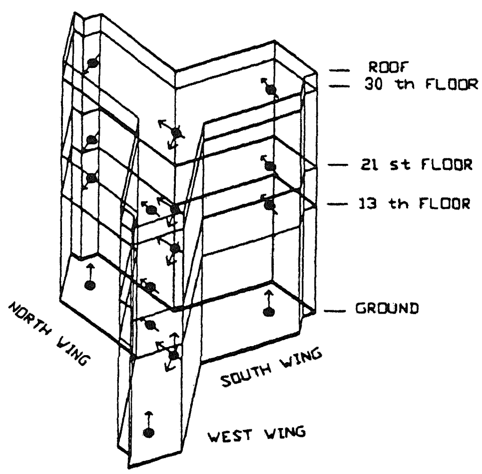


Figure 3. Strong Motion Insturmentation Scheme

analyses, which employed simplest possible foundation modeling is presented. The kinematic interaction between the foundation and soils was approximately evaluated from one-dimensional site-response analyses.

The inertial interaction between the foundation and soils is represented by discrete springs and dashpots whose numerical values were determined from lumped-parameter analogues based on elastic half-space theories. Textbook spring and dashpot formulae for circular foundations are used for this purpose [8]. The foundation model parameter lower and upper bound values are given below in Table 1.

The structural model of the building is developed in the most general form and the simplifications for developing a lumped stick model are imposed during the analysis. The finite element model of the structure developed for this purpose contains 2304 nodes and 5200 elements. In this study the building is primarily represented as a wire model with inclusion of structural walls. Each beam and column is represented as a three dimensional beam/column element with six degrees of freedom at each node for a total of 13824 degrees of freedom describing the full superstructure [Olowokere, 1991].

The flexibility properties of the models are obtained from the average deformations computed at the center of rigidity of each floor. The axial flexibility of the columns are included and based on the gross cross-sectional area. The influence of parameters that affect the structural flexibilities are evaluated by developing two flexibility matrices representing the upper and lower bound estimates of these parameters. The flexibility analysis is performed using ABAQUS (1989) finite element analysis program.

The center of stiffness of all the floors except first floor coincides with the geometric center of the floor which is located at the center of a triangular frame within the center core of the structure. The flexibility matrix terms are computed by applying forces at each node of the respective floor in the coordinate directions and by averaging the deformations of all the nodes at the center of stiffness of each floor. The initial flexibility model is generated for ten discrete points which corresponds to the deformations of the center of stiffness of 4th, 7th, 10th, 13th, 15th, 18th, 21st, 24th, 27th, and 30th floors.

The soil-foundation properties are described by its stiffness and inertia matrices. The structure model however, was developed in the flexibility frame. The models are combined in the following steps: i) converting the soil-foundation model to the flexibility frame, ii) computing the corrected flexibilities of the structural degrees of freedom with flexible foundation and, iii) by inverting the resulting flexibility matrix to the stiffness frame. Six flexibility matrices are obtained by combining the three foundation model parameter estimates with the two struc-

Table-1. Foundation constants for simplified foundation model.

	Lower Bound		Upper Bound	
Mode type	Spring Coefficient	Damping Coefficient	Spring Coefficient	Damping Coefficient
Sliding(lb-sec/in)	3.75×10^{7}	4.62×10^{6}	6.75×10^{7}	6.46×10^{6}
Coupling(lb-sec/rad)	-1.125×10^{9}	-1.38×10^{8}	-2.03×10^{9}	-1.94×10^{8}
Rocking(lb-in/rad)	3.05×10^{13}	1.37×10^{11}	6.21×10^{13}	1.93×10^{11}
Torsional(lb-in/rad)	3.67×10^{13}	1.41×10^{12}	7.32×10^{13}	1.99×10^{12}

tural model parameter estimates. The two flexibility matrices for the translational and rocking degrees of freedom of the soil-foundation-structure model given below in Table 2 and 3 are for the softest and most stiff models

3. ANALYSIS RESULTS

3.1 Modal Analysis:

Modal analysis is conducted and compared to the measured responses in the frequency domain. The modal analysis results are also compared with the forced and ambient vibration measurements reported by other investigators [6]. The frequencies of the first two modes are computed for the model parameters ranges corresponding to the upper and lower bound flexibility estimates are shown in Table 4 below. Also, shown are the measured frequencies and experimental frequencies. The experimental frequencies were obtained by other investigators immediately upon the completion of building construction [Stephen, 1985]. The measured frequencies are obtained from frequency domain analysis of recorded acceleration time histories.

3.2 Time History Analysis:

The acceleration and displacement time histories of the model is computed in the transverse direction. The stick model is subjected to the two components of base motion developed from the free field motion analysis [Kagawa, 1992]. The building response is than resolved to N260 and N350

Table-2. Flexibility Matrix in N-S direction.(1.0 E -08 in/lb)

Rock	0.0000033											
G	0.000098	2.67										
4	0.00152	2.72	8.77									
7	0.00264	2.75	10.43	18.71								
10	0.00376	2.78	11.05	21.07	30.31					Sym		
13	0.00489	2.82	11.67	22.28	33.33	43.56						
15	0.00564	2.84	12.07	23.08	34.57	46.39	53.25					
18	0.00676	2.87	12.69	24.25	36.37	48.99	57.48	69.15				
21	0.00788	2.91	13.31	25.42	38.19	51.56	60.61	74.52	90.63			
24	0.00901	2.94	13.91	26.58	39.99	54.08	63.66	78.48	97.83	116.6		
27	0.0102	2.97	14.51	27.72	42.05	56.54	66.63	82.29	102.7	124.9	145.1	
30	0.0116	3.02	15.31	29.22	44.04	59.72	70.47	87.21	109.1	132.8	156.8	189.7

Table-3. Flexibility matrix in N-S direction.(10 E-08 in/lb)

Rock	0.0000017											
G	0.000049	1.48										
4	0.000757	1.50	4.75									
7	0.00132	1.52	5.64	9.74								
10	0.00188	1.54	5.94	10.98	15.58					Sym		
13	0.00244	1.56	6.23	11.54	17.09	22.14						
15	0.00282	1.57	6.43	11.91	17.67	23.56	26.91					
18	0.00338	1.58	6.73	12.45	18.48	24.76	28.97	34.69				
21	0.00394	1.60	7.03	13.01	19.32	25.92	30.37	37.28	45.03			
24	0.00449	1.62	7.33	13.57	20.17	27.07	31.75	39.06	48.52	57.58		
27	0.00506	1.63	7.62	14.12	20.99	28.21	33.11	40.76	50.72	61.58	71.31	
30	0.00581	1.66	7.99	14.82	22.04	29.61	34.76	42.83	53.27	64.76	76.41	88.11

Table-4. Analytical and measured frequencies (Hz).

	Mode	Analytical		Experimental	Measured
		Lower bound	Upper bound		
N-S	1	0.334	0.474	0.590	0.38
	2	0.854	1.209	1.660	
E-W	1	0.334	0.475	0.595	0.38
	2	0.854	1.209	1.675	
Torsion	1	0.428	0.561	0.565	0.38
	2	1.146	1.461	1.700	

Table-5(a). Maximum Accelerations(in/sec^2) in the direction of 260 degrees

Floor	Measured	Model-1	Model-2	Model-3	Model-4	Model-5	Model-6
G	82.22	107.26	119.52	128.18	110.54	122.78	130.79
13	99.76	96.13	96.16	95.89	98.64	102.02	101.8
21	94.02	52.02	59.51	63.43	75.97	86.63	98.49
30	145.71	177.49	176.49	175.42	162.47	164.29	171.50

Table-5(b). Maximum Accelerations(in/sec^2) in the direction of 350 degrees.

Floor	Measured	Model-1	Model-2	Model-3	Model-4	Model-5	Model-6
G	68.36	81.68	86.15	88.64	82.52	86.92	89.29
13	104.79	80.87	83.37	83.97	88.85	91.45	92.06
21	72.05	60.22	63.58	64.54	36.36	36.96	39.26
30	96.16	127.77	132.91	133.83	124.25	133.44	136.09

degrees by simple superposition the computed response in two orthogonal directions. There are a total of six cases analyzed which correspond to three soil-foundation and two structural modeling assumptions. The comparisons of measured and computed accelerations with enveloping values are given in Tables 5 (a) and (b) for two translational coordinate directions.

The models are numbered from one to six corresponding to increasing fundamental frequency. Figure 4 is included to show the comparative acceleration time history of the 30th floor in the dominant (260 degree north) direction. The computed response is for the best estimate analytical model.

4. SUMMARY AND CONCLUSIONS

The results of a study on the correlations of analytical and measured response of a pile supported reinforced concrete moment resisting frame building, during the Loma Prieta earthquake, is presented. In this study the soil-foundation system is modelled as a equivalent circular mat and the superstructure is represented by a stick model. The damping and stiffness coefficients of the structure and foundation are selected such that they represent upper, lower and mid point representative values.

The following conclusions are reached from the analysis and correlations:

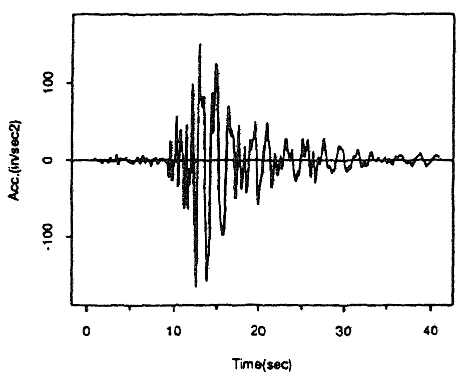


Figure 4. (a) Computed Response

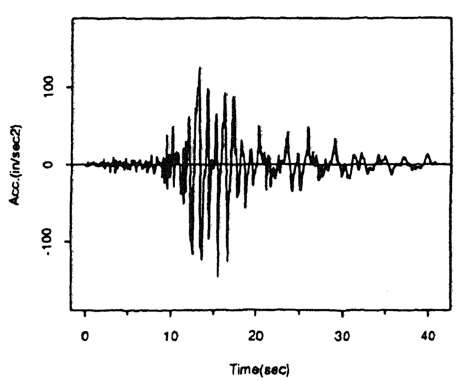


Figure 4(b) Measured Response

1. Soil-foundation-structure flexibility models show a variation of 100% between the upper and lower bound parameters.

2. The ranges of analytically obtained design shears and overturning moments are within the ranges computed from measured responses.

3. Comparisons in the time domain between the measured and computed responses show appreciable similar behavior.

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