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Dynamic Response of the Oakland Outer Harbour Wharf during the 1989 California Loma Prieta Earthquake

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ABSTRACT

The dynamic behaviour of a batter piled wharf at the Oakland Outer Harbour during the 1989 Loma Prieta earthquake was evaluated using strong motion data collected at the site. The study focussed on analysing the response of the structure during the seismic event and on determining the fundamental frequencies of vibration of the structure. Relative movement analysis showed that individual sections of the structure move independently across separation joints. Potential natural frequencies of the structure were determined at 3.2 Hz, 4.5 Hz, and 6.6 Hz. The potential natural frequency values compare reasonably well with the values obtained from a computer model developed for the study.

INTRODUCTION

The structural dynamic behaviour of a batter piled wharf at the Oakland Outer Harbour was evaluated using strong motion data collected at the wharf site during the 1989 California Loma Prieta earthquake. A study conducted at the University of British Columbia (UBC) focussed on analysing the response of the structure during the seismic event and on determining the fundamental frequencies of vibration of the structure.

The Magnitude 7.1 Loma Prieta earthquake occurred on the San Andreas fault 16 km east of Santa Cruz and 33 km southwest of San Jose at 5:04PM (PDT) on October 17, 1989. The epicentre of the earthquake relative to the location of the wharf is shown in Fig. 1. Peak horizontal ground accelerations near the epicentre reached 0.64 g (Shakal et al 1989). At the location of the Oakland Outer Harbour Wharf (OOHW) free field ground accelerations reached 0.29 g. In the vicinity of the OOHW, well documented failures occurred on the Bay Bridge and Cypress Viaduct. In addition, directly adjacent to the OOHW, the Seventh Street Pier (a structure similar to the OOHW) sustained considerable damage to its piles and deck. The location of the Bay Bridge (A), the Cypress Viaduct (B) and the Seventh Street Pier (C) are shown relative to the Oakland Outer Harbour Wharf (D) in Fig. 2.

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Wharf Structure

The OOHW is a batter piled wharf designed and constructed in the mid 1970's. The main wharf apron is made up of ten independent sections separated structurally by construction joints. These independent sections have lengths between successive control joints varying from 36.5 m to 88 m, making a total length of approximately 490 m. The width of the wharf is approximately 19 m and the total structure, including all the independent sections, has a total mass of approximately 18.5(10^6) kg. The main support system consists of vertical and batter solid square precast prestressed concrete piles spaced at 3.8 m in the east-west direction and 1.8 m, 2.7 m or 3.6 m in the north-south direction. A sheet piled wall, which mainly serves to isolate the wharf from the backlands to the east, supports the extreme east side of the wharf. The piles are cast integrally with cast-in-place concrete pile caps which are generally 1.2 m wide by 1.2 m deep and cast-in-place concrete slabs which range in depth from 0.46 m to 0.63 m. Batter piles typically frame into a pile cap while vertical piles generally frame directly into the slab. The pile caps span in the north-south direction along the longitudinal axis of the wharf. The piles, pile caps and slabs form integral units that make up each of the individual wharf sections between successive isolation joints. The piles range from a free standing length through water and air of 4.0 m for the sheet steel wall to 13.75 m for the concrete precast pile at the west side of the structure. Elevation and plan views of the structure are shown in Fig. 3. The supporting soils at the site consist of a 20 m layer of loose to medium dense sand overlying several layers of clay and medium dense sand. These layers of soil are in turn supported by bedrock some 150 m below the surface of the soil.

Strong Motion Data

Seismic data for the OOHW was obtained by the California Strong Motion Instrumentation Program (CSMIP). The data consisted of digitized and processed acceleration records from eleven of twelve sensors located on the site. A recorded signal for free field vertical sensor number 2 was not available from CSMIP because the sensor had malfunctioned. The locations of the sensors and their positive orientations are shown in the plan view of the structure in Fig. 3. Note that only the middle section of the wharf contains two sensors measuring in the same direction. The sensors were positioned with the positive direction of the sensors on bearings of N35°E and N55°W for the longitudinal and transverse sensors respectively. Considering the location of the wharf relative to the epicentre, and the orientation of the sensors, the records show that ground waves moved through the site in a direction which was approximately 45° to the long axis of the wharf.

The corrected acceleration time histories for each of the recorded signals are shown in Fig. 4. The peak structure accelerations were 0.45 g in the longitudinal direction and 0.32 g in the transverse direction. The peak free field accelerations for each of the principal directions were 0.29 g in the longitudinal direction and 0.28 g in the transverse direction. The functioning free field sensor measuring accelerations in the vertical direction recorded a peak value of 0.07 g.

DATA ANALYSIS

Extensive analyses were conducted on the strong motion data in order to improve the understanding of the seismic behaviour of the OOHW (Yee, 1995). Results from real time displacements and frequency domain analyses are discussed in the following sections.

Response Spectra

The 5% damping response spectra corresponding to selected longitudinal and transverse motions are shown in Fig. 5 and Fig. 6 respectively. Each figure includes the response spectra for accelerations recorded at the two free field (FF) stations and at two locations on the structure (Str). Fig. 5 shows that the spectral values for the north free field station (Ch.12) are generally larger than those for the south free field station (Ch.3). Significant peaks occur at 1.1 Hz, 1.5 Hz and 2.4 Hz for the south station and at 1.1 Hz, 1.5 Hz, 3.0 Hz, 3.9 Hz and 5.4 Hz for the north station. These frequencies possibly correspond to natural frequencies of the free field soil column. The shape of the spectra for the structural motions is similar to, but generally greater than the shape of the spectra for the free field motions. The similarity in the free field and structure spectra indicate that the structure motions are dependent on the free field motions. The higher structural spectral values indicate that there are inertial effects added to the ground motions.

Fig. 6 shows no regular trend in the response spectra for the free field stations (Ch.1 and Ch.10) except that high energy levels are present in the range of 1.2 Hz to 2.0 Hz. Significant peaks occur at 1.6 Hz, 2.4 Hz and 4.8 Hz for the south station and at 1.6 Hz, 2.5 Hz and 3.8 Hz. The spectra for the structural motions show significant peaks at 1.6 Hz and 3.2 Hz. The peak at 1.6 Hz corresponds to a free field peak, but the peak at 3.2 Hz appears to be independent of the free field spectra and is therefore a potential natural frequency of the structure. Other peaks that appear to be independent of the free field spectral values occur at approximately 4.8 Hz and 6.7 Hz, again indicating possible natural frequencies. In the other regions of the spectrum, the trends of the free field and structural spectral values indicate that there are inertial effects added to the ground motions.

Relative Movement Analysis

Analysis of relative movement between separate sections of the wharf was conducted using a computer animation program developed at UBC. Using the structure time histories, the program provides instantaneous displacement plots for the wharf at several times during the seismic event. Several plots are shown in Fig. 7 to illustrate the relative movement of different sections of the wharf. Little relative motion between the sensors was detected during the initial stages of the earthquake, but differential activity was apparent after the first 12 seconds of the recorded motions. The absolute displacements reached 11.2 cm in the longitudinal direction and 8.2 cm in the transverse direction. The maximum differential displacements reached 3.1 cm between the structure longitudinal sensors and 4.4 cm between the structure transverse sensors.

Frequency Response Analysis

The transfer, phase and coherence functions between the free field (input) signals and the structure (output) signals were calculated using a computer analysis program developed at UBC. The results of the analysis for the transverse direction using the north free field record as input and each of the signals from the sensors in the middle portion of the wharf as output are plotted in Fig. 8 and Fig. 9. Results of the analysis show that there are several peaks in the frequency response function. If one assumes a system with classical damping, potential natural frequencies of the structural system are those frequencies which possess peaks in the frequency spectrum along with phase angles near 90° or 270° and a coherence close to unity. For systems with non-classical damping or systems with multiple inputs, the phase and coherence requirements may not be satisfied. From the results of the analysis, the best candidates for natural frequencies of the system are 3.2 Hz, 4.5 Hz and 6.6 Hz.

COMPUTER MODELLING

A computer model of the middle section of the instrumented structure was created and analysed using the finite element program SAP90 (Wilson et al). Details of the computer model are given in Yee (1995). The analysis included the determination of natural frequencies and mode shapes of the structure. The first three theoretical natural frequencies of the structure were found to be 3.2 Hz, 4.3 Hz and 9.4 Hz. The mode shapes of the three respective natural frequencies are shown in Fig. 10. These natural frequencies compare reasonably well with the values determined from the recorded data. The table in Fig. 10 shows a comparison of results including the principal directions of each of the three modes and their associated participating masses. It is interesting to note that the fundamental mode shape includes torsional and translational motions.

CONCLUSIONS

Strong motion data for the Oakland Outer Harbour Wharf obtained during the Loma Prieta earthquake was analysed to identify the natural frequencies of the structure during the seismic event. Potential natural frequencies of the structure were determined to be at 3.2 Hz, 4.5 Hz and 6.6 Hz. The results of this analysis for the first two modes compared well with those obtained from a finite element model produced for the structure. Animation of the structure using its calculated displacement time histories derived from the recorded input ground accelerations showed that differential motion of the separate sections of the wharf were as high as 4.4 cm during the seismic event.

ACKNOWLEDGEMENTS

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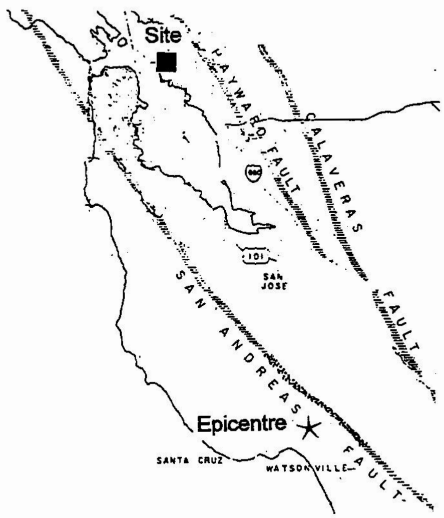


Fig. 1 - Oakland Outer Harbour Location Relative to Earthquake Epicentre (Adapted from Shakal et al 1989)

Fig. 2 - Aerial Photograph of Oakland Outer Harbour (Adapted From McNutt and Syndor, 1990)

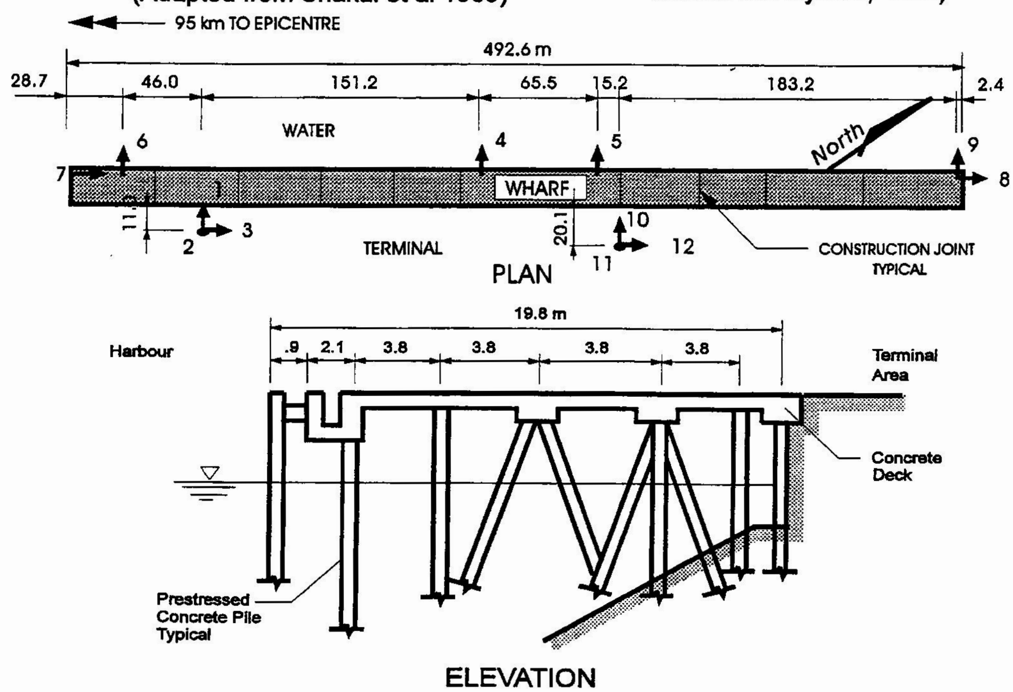


Fig. 3 - Elevation and Plan View of Wharf

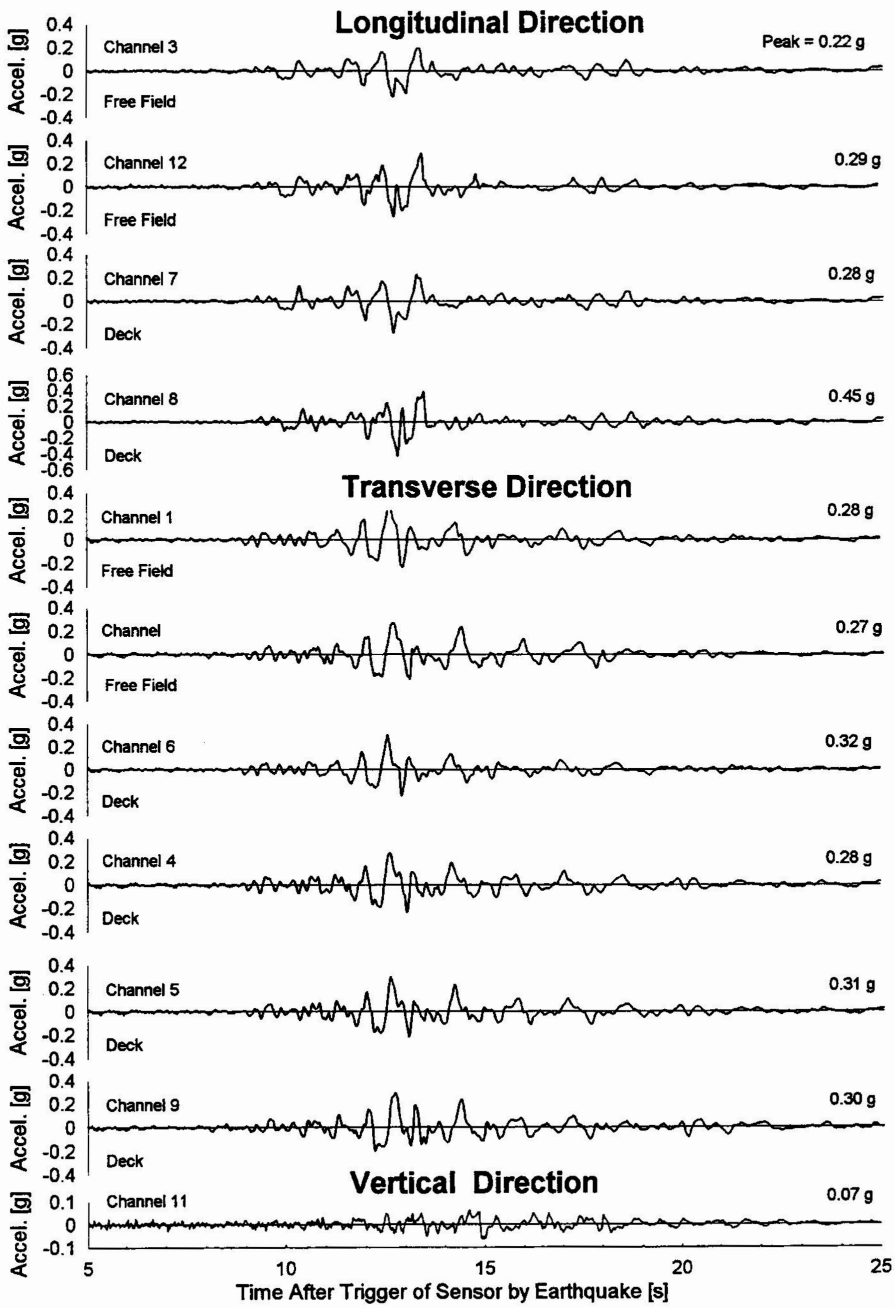


Fig. 4 - Processed Acceleration Time Histories Obtained at the OOHW During the Loma Prieta Earthquake

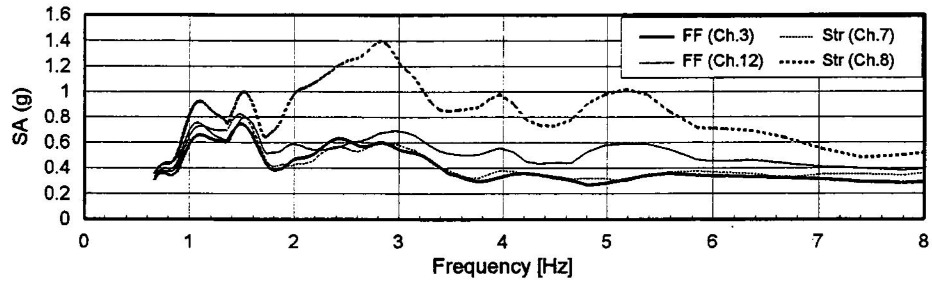


Fig. 5 - Response Spectra (Longitudinal Direction)

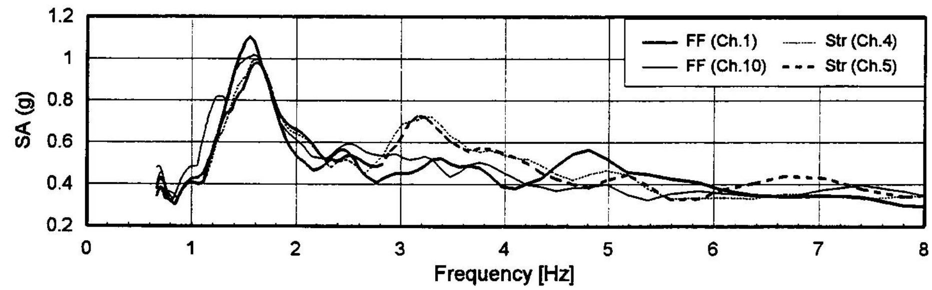


Fig. 6 - Response Spectra (Transverse Direction)

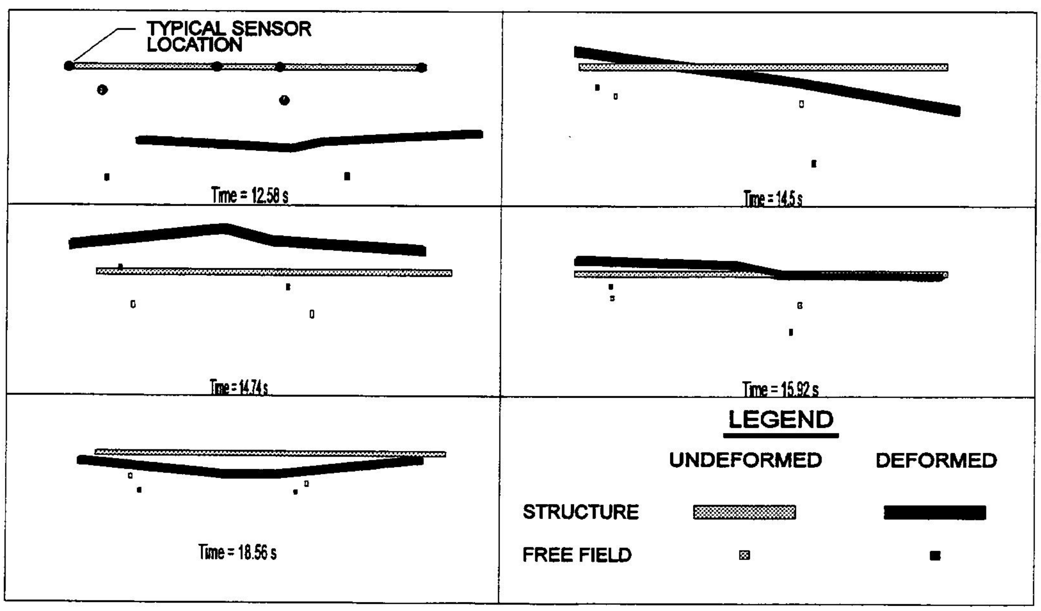


Fig. 7 - Absolute Displacement During the Loma Prieta Earthquake (Exaggerated 20 Times)

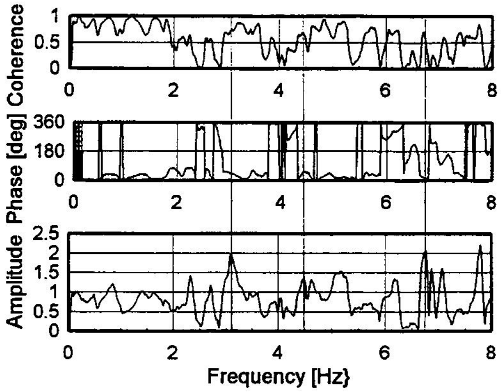


Fig.8 - Transfer, Phase and Coherence Functions Between Sensor #10 (Input) and Sensor #4 (Output)

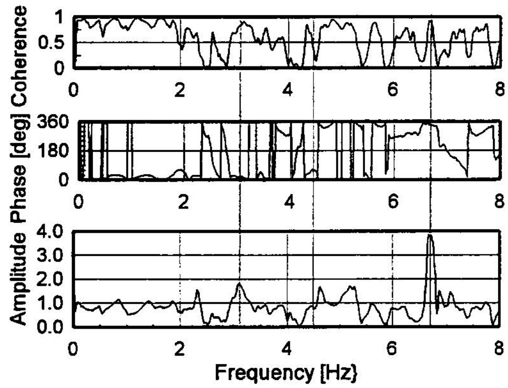
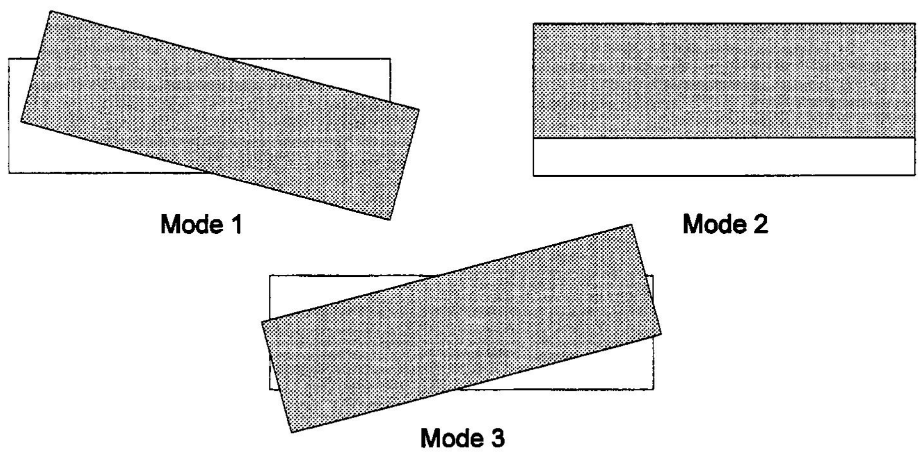


Fig.9 - Transfer, Phase and Coherence Functions Between Sensor #10 (Input) and Sensor #5 (Output)



Mode	From Measured Data [Hz]	From Computer Model [Hz]	Principal Direction	Participating Mass X-dir	Participating Mass Y-dir
1	3.2	3.2	Translation/Torsion	41.23	0.03
2	4.5	4.3	Translation	0.01	99.95
3	6.6	9.4	Torsion	58.39	0

Fig. 10 - First Three Analytical Mode Shapes and Comparison of Natural Frequencies Obtained From Strong Motion Data and Computer Model