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SYNOPSIS : Rayleigh wave investigation is made in the Marina District to study geotechnical factors controlling the damage patterns in the Loma Prieta earthquake. A portable system has been developed for determining a Rayleigh wave dispersion curve based on the measurements of artificially induced ground vibration or microtremor. Five sites are selected along a line crossing the hydraulic fill zone in which structures and/or buried utilities were significantly damaged. An inverse analysis on the measured dispersion curves results in a cross section of shear wave velocity profiles in the District. Site amplification and liquefaction potential of each site are estimated and discussed based on the V_s-profiles. It is shown that soil liquefaction is likely to have occurred throughout the fill zone, and that the predominant period of ground motions in the zone of structural damage is longer than and closer to the natural period of structures with soft first story than that in the non-damaged zone. These results appear to be consistent with the damage patterns in the District, indicating that the proposed investigation is effective for seismic zonation.

INTRODUCTION

The Loma Prieta Earthquake of October 17, 1989 affected not only the epicentral area but also the San Francisco Bay area, and caused extensive damage to various structures found on soft soils. This emphasizes the significant effects of local geological conditions on seismic ground response and resulting damage patterns during earthquakes. The damage observed on soft soils seem to be caused by a combined effect of seismic ground amplification, inadequate design of structure, and ground failures including soil liquefaction.

To confirm and calibrate our understanding and knowledge concerning seismic ground amplification and soil liquefaction during earthquake, and resulting damage patterns, it is necessary to know soil profiles including shear-wave velocity.

Shear wave velocity is in fact an important soil parameter in the evaluation not only of dynamic ground response characteristics but also of liquefaction potential of sands. Recent studies by Stokoe et al. (1988) and Tokimatsu and Uchida (1990) suggested the effectiveness of shear wave velocity for liquefaction evaluations.

However, most of the field tests currently conducted for determining shear wave velocity profiles require boreholes, and thus are costly and time consuming, and may not be performed conveniently in all cases. Although seismic prospection such as refraction and reflection methods does not require boreholes, it cannot reliably be used in the routine practice, because of its inability to detect relatively soft layer sandwiched in between stiffer soils.

Rayleigh wave method, which has been improved in recent years (Stokoe et al., 1984), is promising and attractive, since it can be performed on the ground surface without any boreholes and it has potential capability to detect soft layer in between stiff layers. Such simple and yet efficient site investigation is particularly preferable for seismic microzonation such as the evaluation of safety of each private home in a large area and the identification of weak spot along various life lines. Although several Rayleigh wave methods have been proposed, they have their own limitations and have not been used routinely and reliably.

To improve the reliability and performance of Rayleigh wave investigation, a portable system was developed and the method of measurements was modified. Field investigation was then made in the San Francisco Bay area using the improved system and procedure for characterizing shear wave velocity profiles.

The object of this paper is to present a preliminary report concerning the result of Rayleigh wave investigation and to discuss the effects of local soil conditions on damage patterns in the Marina District.

RAYLEIGH WAVE METHOD

The principle of the Rayleigh wave methods lies in the fact that the Rayleigh wave is dispersive. Its phase velocity varies depending on wavelength or frequency, i. e., waves with short wavelengths sample soil properties at small depth, whereas waves with large wavelengths reflect properties of soil from near surface to much large depth. Thus, Rayleigh wave investigation is to measure the variation of phase velocity with wavelength which is called dispersion curve. An inverse analysis of the measured curve results in V_s-profile, on the condition that the soil layers in the deposit are horizontally stratified (Haskell, 1954).

There are basically two methods to determine a Rayleigh wave dispersion curve, i. e., active and passive methods. The active method measures Rayleigh waves in vertical ground vibrations which are generated artificially in some way. The passive method, on the other hand, observes Rayleigh waves in microtremor, i. e., ambient vibration of the ground, without generating any particular ground motions.

In the active method, the vertical motions with predominant Rayleigh waves are generated either by an impulsive source (Stokoe and Nazarian, 1984) or an exciter oscillating steady-state vertical harmonic motions (e. g., Tokimatsu et al., 1991b). In this method, a pair of sensors is set apart on the ground surface in a line with the source, and phase velocity is computed based on the phase lag between the motions measured by two sensors. The method is suited to explore surface soils at a depth smaller than 10 to 20 meters. Its application, however, appears restricted to the determination of much deeper soil profile because of the difficulty in generating long wavelength.

In the passive method, several vertical sensors are distributed over the ground surface, and phase velocity vs. wavelength relationship is determined based on the measurements of microtremor. Although the method has been often used to characterize deep soil structure (e. g., Toksöz, 1964), it has seldom been applied to the determination of shallow soil profiles. This is partly because the inclusion of higher modes of Rayleigh waves in microtremor in the short wavelength range makes both reliable determination of dispersion curve and its inverse analysis difficult.

TEST APPARATUS AND TEST PROCEDURE

Test Apparatus

The method adopted in this study is a hybrid one which combines the advantages of the active and passive methods discussed above. An attempt was made in this method to compute a Rayleigh wave dispersion curve from observed motions in the field using a laptop computer.

A portable system which was devised for this purpose, consists of several sensors, amplifiers, and a laptop computer. The sensors are vertical velocity transduces with a natural frequency of 1 Hz. The computer is a model PC-386LS from EPSON, equipped with an AD converter. The AD converter has a resolution of 12 bits. All equipments of this system can be functioned by a compact battery and may not need common electric current. The total weight of the system is less than 40 kgs.

Determination of Dispersion Curve

Fig. 1 shows a schematic diagram of the system. Two to six sensors are distributed over a ground surface and construct a Rayleigh wave measurement array (e. g., Toksöz, 1964, Capon, 1973, and Stokoe and Nazarian, 1984). The distance between the sensors in the array depends on measured wavelength. The minimum and maximum distances between the sensors used in this investigation were 0.5 m and 10 m.

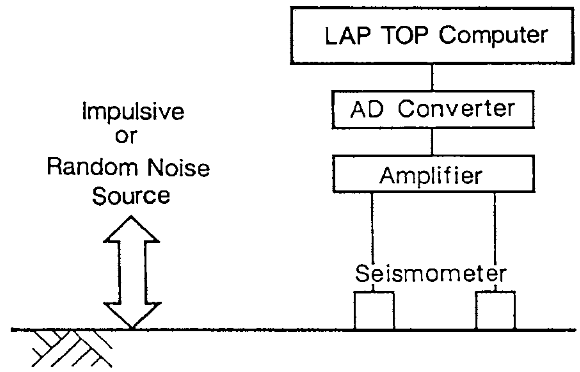

Fig. 1 Schematic diagram of test system

The sensors monitor vertical ground surface motions of either microtremor or artificially induced random vibration which is induced away from the array by tapping the ground surface using appropriate equipments or foot. The artificially induced vibration was used for short wavelengths, and microtremor for long wavelengths. The analog motions measured with the sensors are amplified, converted into digitized form, and stored in the memory of the computer.

Based on spectrum analyses on the digitized motions measured at different locations, phase velocity, c, at each frequency, f, is calculated (e. g., Toksöz, 1964, Capon, 1973, Stokoe and Nazarian, 1984, and Tokimatsu et al., 1991b). The corresponding wavelength, λ, can be given by

$$\lambda = c/f \qquad (1)$$

In this way, the correlation between phase velocity and wavelength is determined for a frequency range approximately from 3 Hz to 50 Hz. It takes about an hour to measure Rayleigh wave and to compute its dispersion curve. The detailed procedure will be published elsewhere.

Determination of Shear Wave Velocity Profile

The determination of shear wave velocity of a deposit requires an inverse analysis on the measured dispersion curve. The soil deposit is assumed to be horizontally stratified and consists of N layers as shown in Fig. 2. Each layer is homogeneous and isotropic, and is characterized by thickness, H, mass density, ρ, P-wave velocity, V_p, and S-wave velocity, V_s. The dispersion curve corresponding to the assumed soil model can be computed based on the Haskell's theory (Haskell, 1953). Thus the inversion is to find the soil model that provide the same dispersion curve as the observed one.

A nonlinear optimizing method originally proposed by Dorman and Ewing (1962) was modified and used in the inverse analysis (Tokimatsu et

Layer No.	Thickness	Density	P-Wave Velocity	S-Wave Velocity
1	H_1	ρ_1	V_{P1}	V_{S1}
2	H_2	ρ_2	V_{P2}	V_{S2}
⋮	⋮	⋮	⋮	⋮
N-1	H_{N-1}	ρ_{N-1}	V_{PN-1}	V_{SN-1}
N	∞	ρ_N	V_{PN}	V_{SN}

Fig. 2 One-dimensional soil layer model

al., 1991b). Although mass density, P-wave velocity, S-wave velocity, and thickness of each layer are the variables controlling the correlation between phase velocity and wavelength, the effects of the first two properties are significantly less than the remaining two. Thus the mass density and P-wave velocity are predetermined and only the shear wave velocity and thickness of each layer are sought in the inversion. The effects of higher modes of Rayleigh waves which are dominant in the high frequency range are taken into account in the analysis according to the study by Harkrider (1964). The details of the inverse analysis have been described by Tokimatsu et al. (1991b).

DAMAGE PATTERNS IN MARINA DISTRICT

Fig. 3 shows the map of the Marina District after Seed et al. (1990) in which apparent zone of liquefaction is also shown. Solid line in the map shows the shore line in 1869. The central part of the Marina District was hydraulically filled with sand after 1896. Their study indicates that the apparent zone of liquefaction encompasses the entire hydraulic fill zone, as well as portions of the earlier fill around the perimeter of the District and overlying the coastal marshes at the western end of the District. Considerable damage to buried

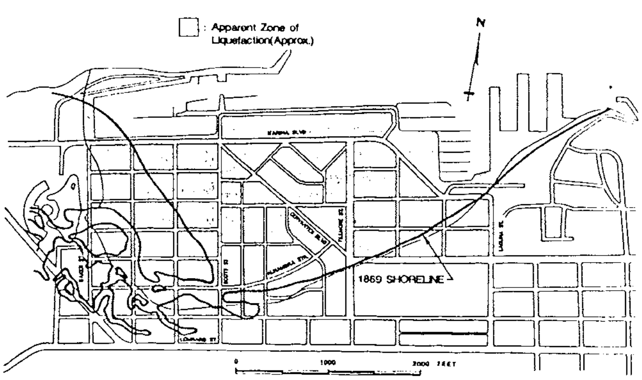

Fig. 3 Map of Marina District and old coastline (after Seed et al., 1990)

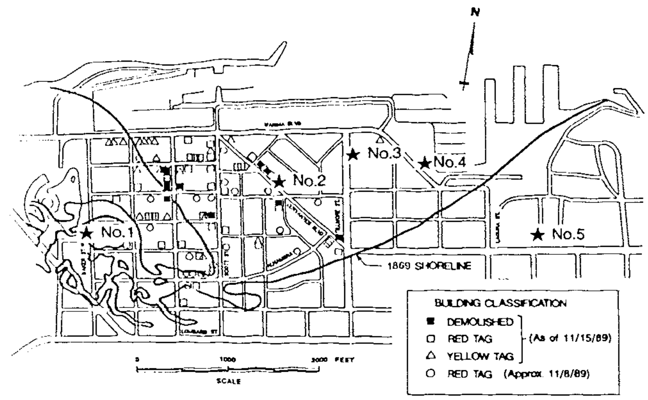

Fig. 4 Map showing damage to structure in the Marina District (after Seed et al., 1990) and test sites

utilities occurred throughout much of the apparent zone of liquefaction. Thus it appears that the major cause of the damage to buried utilities was due to soil liquefaction.

Most of the collapsed and damaged houses, on the contrary, concentrate on the west part of the liquefied zone, and very few structural damage is observed on the east part of the liquefied zone, as shown in Fig. 4. They concluded therefore that a majority of the damage to structures in the District on October 17, 1989 was not due primarily to soil liquefaction but caused by strong shaking.

FIELD INVESTIGATION AND ITS RESULTS

The field investigation was carried out at various sites in the San Francisco Bay Area including five sites in the Marina District from late March to very early April. The five sites (called Sites No. 1 to 5) were distributed over a band running from the west to the east through the hydraulic fill zone as shown in Fig. 4 to characterize a cross section of shear wave velocity profiles across the District. Sites No. 2 to 4 are in the fill zone, Site No. 1 on the boundary between liquefied and non-liquefied zones, and Site No. 5 outside the liquefied zone. It took one day in total for this particular investigation.

The open circle in Fig. 5(a) shows the correlation between phase velocity and wavelength observed at Site No. 4. The inverse analysis was then conducted for the observed correlation assuming a three or four-layer model with appropriate initial soil properties.

Also shown in Fig. 5(a) is the computed dispersion curve from the inverted soil model shown in Fig. 5(b). The computed dispersion curve appears to show a good agreement with the observed correlation, indicating that the inversion was successfully conducted. Particularly noted is a good agreement in trend at short wavelengths which cannot be obtained without considering the effects of higher modes of Rayleigh waves in the

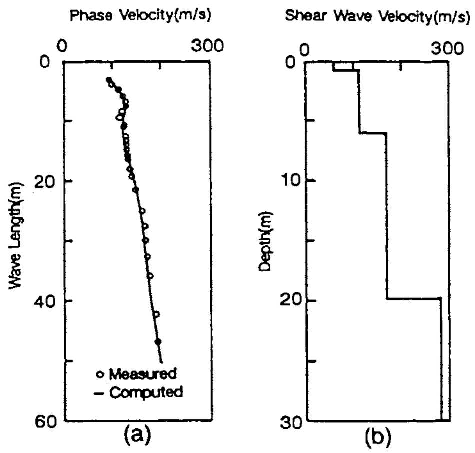

Fig. 5 Measured and computed dispersion curve and resulting shear wave velocity at Site #4

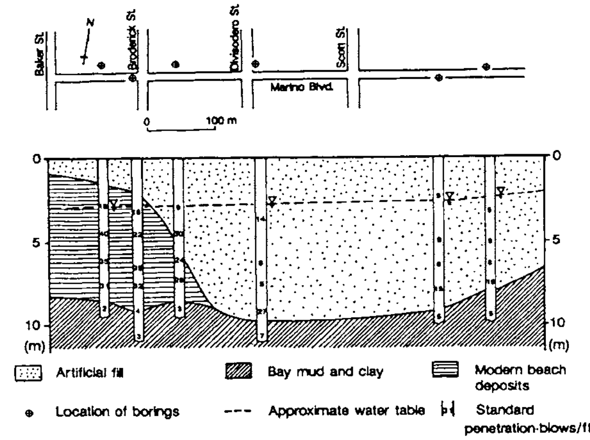

Fig. 7 Cross section of boring log along Marina Blvd. (after Lane, 1987)

inversion.

The inverse analyses were also conducted for other four sites, and the resulting cross section of shear wave velocity profile across the District is shown in Fig. 6. The shear wave velocity of the top layer is approximately 110 to 135 m/s in the liquefied zone and more than 140 m/s outside the liquefied zone. The shear wave velocity of the second layer is between 165 and 185 m/s in the hydraulic fill zone and about 235 m/s outside the fill zone, i. e., Sites No. 1 and 5. The second layer is underlain by a stiffer layer with a shear wave velocity greater than about 300 m/s. Broken lines in the figure indicate the boundaries between these layers.

Fig. 7 shows a cross section along Marina Blvd. after Lane (1987). It appears that the boundary between the top and the second layers in Fig. 6 corresponds to the top of the Bay Mud underlying the fill in Fig. 7. Thus, much of the top layer with V_s less than 135 m/s in Fig. 6 is considered as a sandy fill, and much of the second layer with V_s = 165 to 235 m/s is young Bay Mud. The latter is consistent with the statement by Seed et al. (1990) that shear wave velocities within the Bay Mud are 150 m/s to 210 m/s. The stiff layer underlying the young Bay Mud may probably correspond to either old Bay Mud or dense, sandy strata, considering the fact that these strata typically have shear wave velocities of about 330 m/s (Seed et al., 1990).

The thickness of the top layer takes its maximum of about 10 meters in the center of the hydraulic fill zone, i. e., Sites No. 2 and 3, and decreases with distance toward the perimeter of the liquefied zone. Such a layer with a low shear wave velocity appears to diminish at Site No. 5 which is outside the fill zone.

The second layer also takes its maximum thickness at the center of the liquefied zone, and decreases its thickness with distance toward the edge of the liquefied zone.

Comparison of the shear wave velocity profiles with the damage patterns in the District indicates the following characteristics:

1) The hydraulic sandy fills in the liquefied zone have shear wave velocities less than 135 m/s with thicknesses varying up to 10 meters. Such sandy soils are likely to liquefy during moderate to strong earthquakes (e. g., Tokimatsu and Uchida, 1990, and Tokimatsu et al., 1991a).

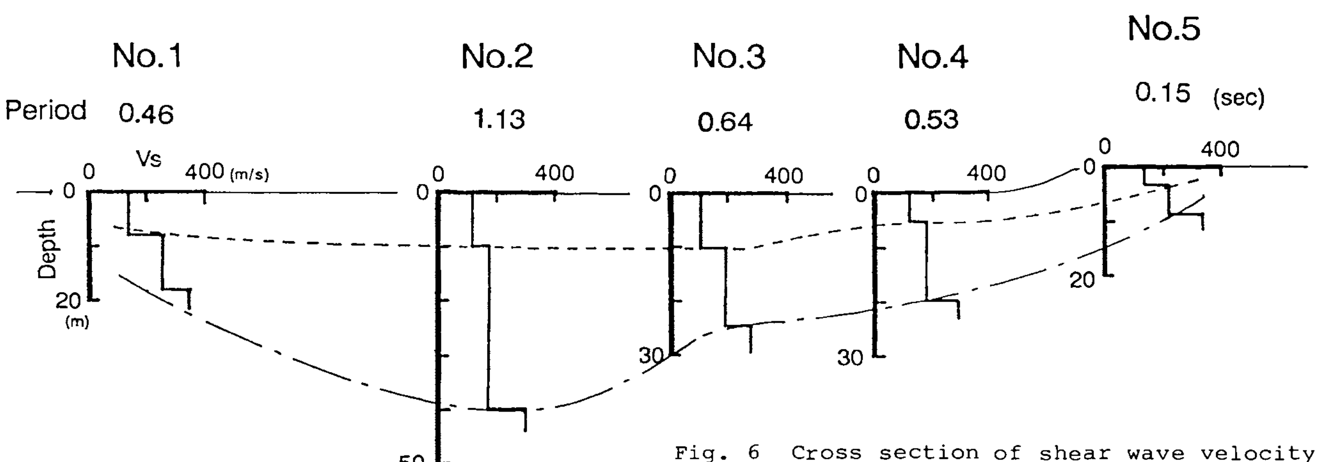

Fig. 6 Cross section of shear wave velocity across the Marina District

2) The hydraulic fills in the zone of structural damage are thick and loose, i. e., the thickness is about 10 meters and V_s = 120 m/s which combination appears worse than that of any other site with little or no structural damage.

3) The second layer is thicker and softer in the fill zone than outside the fill zone, and even thicker within the zone of structural problems, i. e., the thickness is about 30 meters at Site No. 2 and equal to or less than 15 meters in other sites within the fill zone.

EFFECTS OF SITE AMPLIFICATION ON STRUCTURAL DAMAGE

To characterize local geological effects on seismic response and resulting damage patterns, a preliminary computation was made using the equivalent linear dynamic response analysis similar to the well-known program SHAKE (Schnabel et al., 1972).

It is assumed in the analysis that the third layer with a shear wave velocity greater than about 300 m/s is the bedrock at all sites and that the bedrock input motion is the E-W component record of Telegraph Hill strong motion station. The nonlinear soil properties of shear modulus and damping with shear strain presented by Seed et al. (1984) and Seed et al. (1990) were used for the fill and the young Bay Mud.

Although significant non-linear behavior within the Marina District during the earthquake cannot adequately be simulated by the equivalent linear analysis, it is conceivable that the analysis could provide qualitative features of the site effects on ground response.

Fig. 8 summarizes the amplification characteristic curve for all sites. The maximum amplification ratios between the assumed bedrock and the ground surface during the earthquake are on the order of 2 and does not seem to vary significantly from site to site. However, the predominant

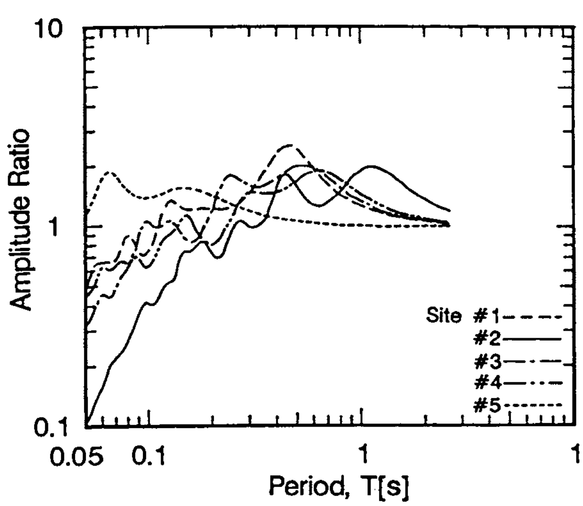

Fig. 8 Amplification characteristic curves at five sites in Marina District

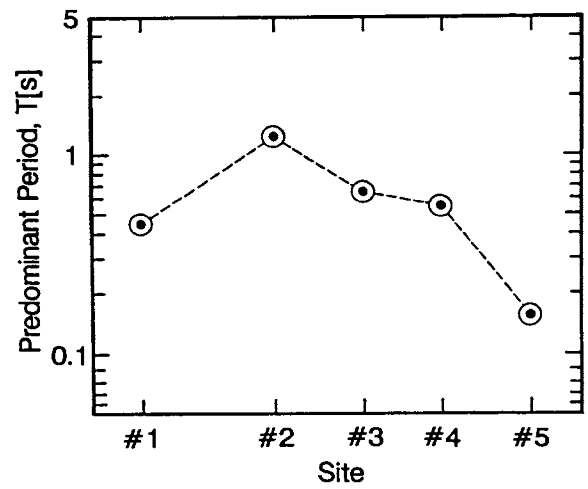

Fig. 9 Variation in predominant period with site in the Marina District

period which provides the maximum amplification ratio significantly depends on site condition. Fig. 9 shows the variation of the predominant period of ground motion with site. There is a definite trend in which the deeper the bedrock and/or larger the thickness of the young Bay Mud deposit, the longer becomes the predominant period.

Comparison of the amplification characteristics with the damage patterns in the District indicates the following:

1) The predominant period in the zone of structural damage is about 1 sec which is considerably longer than that in any other site.

2) The damage patters of structures appear to be significantly affected by the predominant period of the ground motions in such a way that the damage increases as the predominant period of the ground motions increases at least up to about 1 sec.

These appear to be consistent with the finding by Seed et al. (1990). They indicated that the amplification of acceleration caused by cohesive soils underlying the fill appears to have been the primary cause of structural damage in the fill zone, and that much of the structural damage was associated with the collapse of weak ground floors consisting primarily of garages with few walls and thus little structural capability for carrying lateral shear forces at the ground floor levels of two to four-story apartment structures. They further suggested that these structures may have had longer natural periods which were more nearly resonant with the long period ground motions produced by the underlying soil conditions.

It is uncertain but reasonable to consider that the natural periods of two to four-story buildings with soft first story were close to the predominant period of the ground motion in the zone of structural damage. Thus the concentration of damage to structures on the west part of the fill zone may be considered to be due primarily to the longer period ground motions which was amplified by the thick, soft Bay Mud underlying this zone.

CONCLUSIONS

A system was devised for determining shear wave velocity profiles of sub-surface soils through the measurements of Rayleigh waves in random vibrations or microtremor. The field investigation was made using this system to facilitate the understanding of the damage patterns in the Marina District during the Loma Prieta earthquake. It is shown that soil liquefaction is likely to have occurred throughout the fill zone, and that the predominant period of ground motions in the zone of structural damage is longer than and closer to the natural period of structures with soft first story than that in the non-damaged zone. These results appear to be consistent with the damage patterns in the District. Although further refinement is evidently needed, the proposed Rayleigh wave investigation would be a simple and economic means to evaluate the effects of local geological conditions on dynamic response and soil liquefaction during earthquakes.

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