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INVESTIGATIVE ASSESSMENT OF THE DRILLED SHAFTS INSTALLED SHORTLY BEFORE THE SOUTHERN HYOGO-KEN (KOBE) EARTHQUAKE OF JANUARY 17, 1995

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ABSTRACT

Investigations were carried out as part of a post-earthquake investigation of the construction work in progress to assess the integrity of the drilled shafts installed three days before the Osaka-Kobe region of Japan was struck by the devastating earthquake on January 17, 1995. A combination of the newly developed mid-range method of stress-wave measurement and analysis, in combination with the nonlinear ground response and pile-soil interaction analysis using a recorded earthquake motion scaled to account for the epicentral distance is utilized to show that the drilled shafts were not unduly affected in this case. The effectiveness of the methodology in the post-earthquake assessment of drilled shafts for possible adverse effects not readily discernible otherwise is demonstrated.

KEYWORDS

Stress-wave; mid-range method; drilled shaft; integrity; post-earthquake investigation, ground motion; response analysis; ground displacement; dynamic soil-pile interaction.

INTRODUCTION

The magnitude 7.2 earthquake of January 17, 1995 that rocked the Osaka-Kobe region of Japan, early in the morning at 5:46, caused extensive devastation resulting in thousands dead and the property damages in excess of 100 billion dollars. There were several ongoing construction projects when the earthquake struck. At a construction project site in the Ibaraki city, located about 50km from the epicentral region, the foundation work was in progress at the time. Drilled shafts (cast-in-place concrete piles) of diameters 1200mm and 1600mm were being installed by the so called all casing method as per the project schedule only days before the earthquake struck on January 17. The intensity of the earthquake shaking in the area was reported by the Japanese Meteorological Agency (JMA) to be 4 to 5 on the Japanese intensity scale of 7.

From close inspection around the project site area after the quake, it was noted that there was no evidence of ground liquefaction resulting from the earthquake. This was to be expected because the surficial ground condition at the site area consisted primarily of clayey soils. Also, there was no evidence of discernible ground subsidence, lateral movement or other modes of ground failure. However, with the concrete being only a few days old and the earthquake shaking being quite intense, it was necessary to ensure that the drilled shafts were not unduly distressed by the ground shaking before the construction work could resume after the earthquake.

The construction site at Ibaraki, located at a distance of about 50km from the earthquake epicenter, was part of the second phase of a water treatment plant construction project of Osaka regional government. Some of the drilled shafts were installed on January 14, only three days before the January 17 earthquake, and the possible adverse effects of the strong ground shaking was of serious concern. Two of the drilled shafts of diameter 1200mm and length 15.5m installed on January 14 were tested for integrity by the mid-range method (Horiguchi *et al.*, 1996), a newly developed technique that utilizes medium high strain impact pulse of short duration for integrity testing and the stress-wave analysis. Subsequently, nonlinear ground response analysis was carried out to estimate the extent of movement and forces exerted on the drilled shaft during the earthquake shaking. The earthquake motion recorded at Kobe about 30km from the project site was suitably scaled to account for the difference in hypocentral distance for use as the input motion for the response analysis. The paper discuses the two faceted investigative assessment method utilized to confirm the integrity of the drilled shafts.

STRESS-WAVE MEASUREMENT AND ANALYSIS

The Mid-range Method

Low strain integrity testing based on the stress wave theory is fairly well known as a simple means of quality assurance in pile foundations (Goble *et al.*, 1980). However, if the pile is installed in stiff or high radiation damping soil condition, the low strain input signal results in too weak a response of pile, causing difficulty in the assessment of the integrity. Mid-range method attempts to employ a optimum balance between the strain level and the wavelength of impact. Table 1 shows the relative comparison between the low strain, mid-range and high strain methods. Mid-range method utilizes a comparatively high strain impact while attempting to keep the wavelength of impact force relatively short so as to retain the integrity assessment potential. For this, a doughnut shaped hammer of suitable weight passing through a central axis is dropped freely from a height of about 1.5m to hit the pile head centrally. The method attempts to improve the integrity assessment capability while at the same time providing some ability to assess the bearing resistance mobilized by the impact.

Table 1: The concept of the Mid-range Method

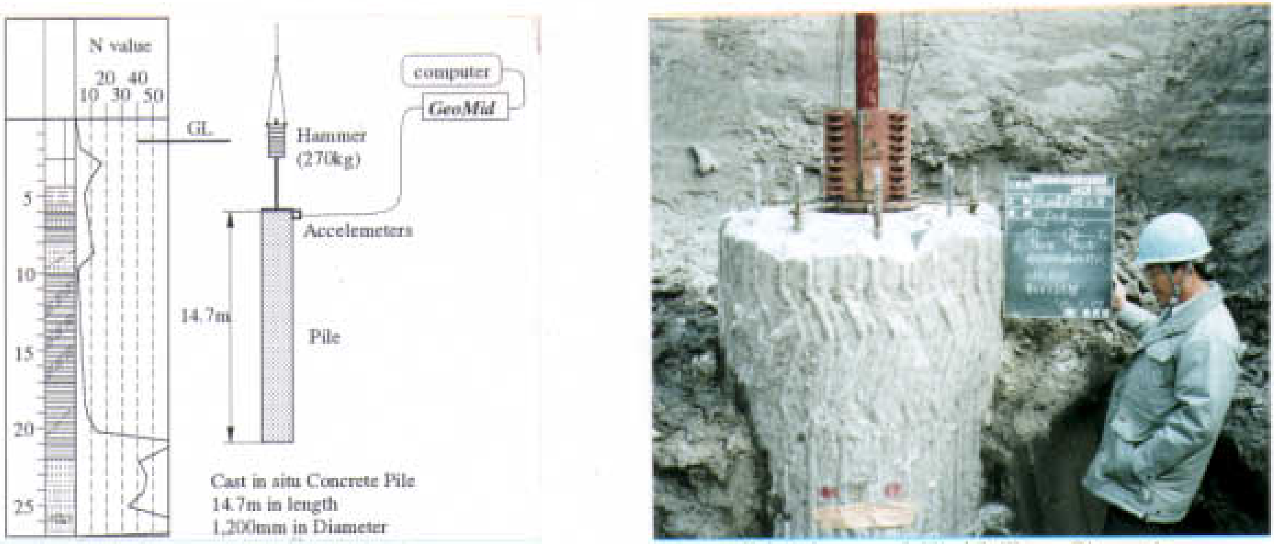
Method	Strain Level	Wavelength of Impact	
Low strain	Low (few μ strain)	Short ($< \frac{1}{10} L_p$)	Pile shape estimation
Mid-range	**High** ($1000 \sim 2000\mu$)	**Short** ($\frac{1}{10}$ *to* $\frac{1}{5} L_p$)	
High Strain	High ($1000 \sim 3000\mu$)	Long ($> \frac{1}{2} L_p$)	
	Resistance prediction	L_p = Pile length	

The Measurement System

The equipment utilized for field measurement consists of two pairs of acceleration and strain sensors, a portable computer and the so called GEOMID system developed for this purpose. Impact pulse is applied to the pile head by dropping a hammer of few tens of kilograms to a few hundred kilogram depending on the diameter and the length of the pile being tested. The stress-wave signal generated by the impact is recorded automatically in the portable computer through the sensors and the GEOMID system. The data thus recorded contains the information regarding any changes in the material properties and the cross-section of the pile, as well as the changes in soil resistance along the pile length. In addition, the signal reflected by the pile toe is recorded as a distinct peak appearing after the time taken by the stress-wave to travel a distance of $2L_p$, where L_p is the length of the pile being tested.

Field Measurement

Two of the drilled shafts of diameter 1200mm and length 14.7m installed on January 14 were tested for integrity by the mid-range method on February 24, 1995. The top of the drilled shafts were about 3.5m below the existing ground level of the site, and excavation to a depth of about 5.0m was necessary to



(a) Ground Profile and Field Test Arrangement (b) Photo of Field Test Situation

Fig. 1. General Arrangement and Field Condition for the Test by Mid-range Method

expose a portion of the pile top for fixing the pick-up sensors. Only two channels of the GEOMID system for acceleration signal pick-up were utilized. The acceleration sensors were attached to the drilled shaft at a level about one shaft diameter below the top. The arrangement for the field test together with the schematic ground condition is shown in Fig. 1(a) and a photo of the field test situation is shown in Fig. 1(b).

The impact force on the top of the drilled shaft was applied by dropping a weight of 270kg from a height of 1.5m, as shown in Fig. 1. The incident impact force together with the response of the drilled shaft-soil system was measured by the two accelerometer sensors fixed to the exposed shaft top by anchor bolts. As can be noted, the weight is dropped centrally on the top with the vertical shaft acting as the guide, causing a stress wave pulse incident at the head.

Inverse Analysis of the Stress-wave Record

The acceleration signal measured near the top of the drilled shaft was integrated once to give the velocity signal, which on multiplication by the impedance gives the force signal. The average of the force signals corresponding to the two sensors is shown Fig. 2 for each of the two shafts tested. As the incident impact pulse is of short duration, it is possible to separate the incident and the response parts of the signal without incurring appreciable error. The initial pulse of about 2.5ms duration in Fig. 2 was considered to be the input impact force for the analysis by the dWAVE method (Sakai, 1988), a

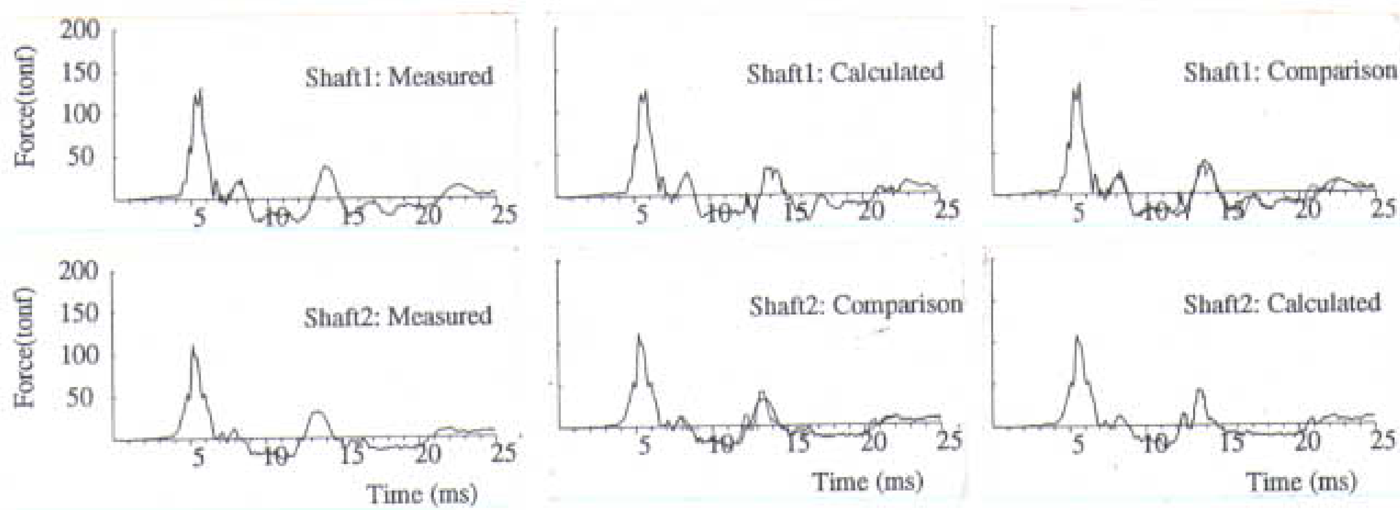


Fig. 2. Measured and Calculated Waveforms and their Direct Comparison for the Two Drilled Shafts Tested by Mid-range Method on February 24, 1995.

Table 2: Pile-soil Parameters Bases on the Inverse Analysis

Shaft 1				Shaft 2			
Shaft AE $\times 10^3$ (ton)	Segment (m)	Resistance (ton)	Damping Factor	Shaft AE $\times 10^3$ (ton)	Segment (m)	Resistance (ton)	Damping Factor
2940.0	1.0	0.00	0.000	2940.0	1.0	0.00	0.000
2940.0	5.0	0.00	0.000	2940.0	4.6	0.00	0.000
2060.0	1.0	0.00	0.004	2470.0	1.0	0.00	0.006
2670.0	2.0	5.00	0.004	2900.0	1.9	5.00	0.006
2670.0	1.0	5.00	0.004	2900.0	1.0	5.00	0.020
2940.0	3.0	0.00	0.004	2790.0	1.9	5.00	0.020
2940.0	3.2	0.00	0.004	2940.0	3.3	5.00	0.020
Pile toe		45.0		Pile toe		36.0	

stress wave simulation program based on the finite difference formulation of the equation of motion. The program can be used for iterative analysis to arrive at a satisfactory match between measured and calculated waveforms.

The calculated waveforms and the direct comparison of measured and calculated waveforms by plotting them together are also shown in Fig. 2. As can be seen, fairly good match was achieved. The parameters of the system corresponding to the calculated waves in Fig. 2 are given in Table 2, where A is the area of cross section and E is the Young's Modulus of the drilled shaft.

Discussion on the Investigation by the Mid-range Method

The slight variation in the value of AE along the length of the drilled shafts in Table 2 is most likely an indication of a larger A owing to the variation in the lateral push of the concrete under its own weight as the casing is withdrawn and the variation in the soil condition along the depth during installation by the all casing method. However, from practical point of view, the value 0f AE is more or less uniform throughout the length, indicating that the drilled shafts were not adversely affected by the earthquake. The equivalent static resistance at different shaft segments together with that at the pile toe,mobilized by the 270kg hammer and drop height 0f 1.5m utilized in the test is also given in Table 2. The total equivalent static resistance mobilized by the small hammer weight is only about 56ton ($\simeq$ 560kN), but it is of the order of 200 times the hammer weight. Of course, heavier hammer weight will be necessary to mobilize larger bearing resistance.

RESPONSE ANALYSIS OF THE SITE

The effect of an earthquake on foundation structures can be considered to result from two broad types of phenomenon: *ground shaking* and *ground failure.* As mentioned above, the ground failure aspect of the earthquake effect on the drilled shafts was not of concern at the Ibaraki site. To evaluate the extent of ground shaking, it was thought to be necessary to consider the effects of the incident motion as well as those arising from local site condition. For this, a nonlinear time domain response analysis of the site was carried out by considering a one dimensional shear beam model of a horizontally layered soil profile (Karkee *et al.*, 1992) under the action of the incident motion estimated based on the available earthquake record at some distance from the site.

Dynamic Properties of the Site

The available soil profile from the site was limited to the variation with depth of soil types, standard penetration N-values, and thickness and unit weight of individual layers. The initial shear modulus profile was estimated from the correlation with N-values, $G_0 = 1200N^{0.8}(t/m^2)$ suggested based on Japanese practice by Ohsaki *et al.* (1973). The stiffness of soil layers during the nonlinear time domain response was determined by a hysteretic model. The strain dependance of the shear modulus and the

Table 3: Model for Strain Dependance of Shear Modulus and Hysteretic Damping Factor

		Soil Types	α	β	γ
$\epsilon = \frac{\tau}{G_0}\left\{1 + \alpha\left\|\frac{\tau}{S_u}\right\|^{\beta}\right\}$	$\epsilon =$ Shear strain				
	$\tau =$ Shear stress				
	$G_0 =$ Initial modulus				
	$S_u =$ Shear strength				
$\xi = \frac{2}{\pi}\frac{\beta}{2+\beta}\left\{1 - \frac{1}{1+\alpha\left\|\frac{\tau}{S_u}\right\|^{\beta}}\right\}$	$\xi =$ Damping factor	Clayey soils	5.0	1.4	600.0
	$\gamma = \frac{G_0}{S_u} =$ Soil constant	Sandy soils	10.0	1.6	1100.0
	$\alpha, \beta =$ Soil constants	Sandy gravel	12.0	1.7	1300.0

equivalent damping factor was based on the Masing's type model in combination with the nonlinear model developed by Ohsaki *et al.* (1978) shown in Table 3 together with the soil parameters utilized. In addition to the hysteretic damping given by the model, viscous damping of 3% was assumed to account for the initial condition.

The initial shear wave velocity (V_s) profile of the site was obtained from the estimated G_0 profile mentioned above. The soil types were classified as clay (CC), sandy clay (SC), clayey sand (CS), sand (SS), sandy gravel (SG) or gravel (GG) based on the major content. The simplified soil layers were then further subdivided into thinner layers for better accuracy in the discrete numerical solution to the equation of motion. The simplified soil profile with subdivided soil layers, together with the V_s profile is shown in Fig. 4. The gravel layer at the bottom of the soil profile with V_s = 450m/s was considered to be the base layer.

Input Earthquake Motion

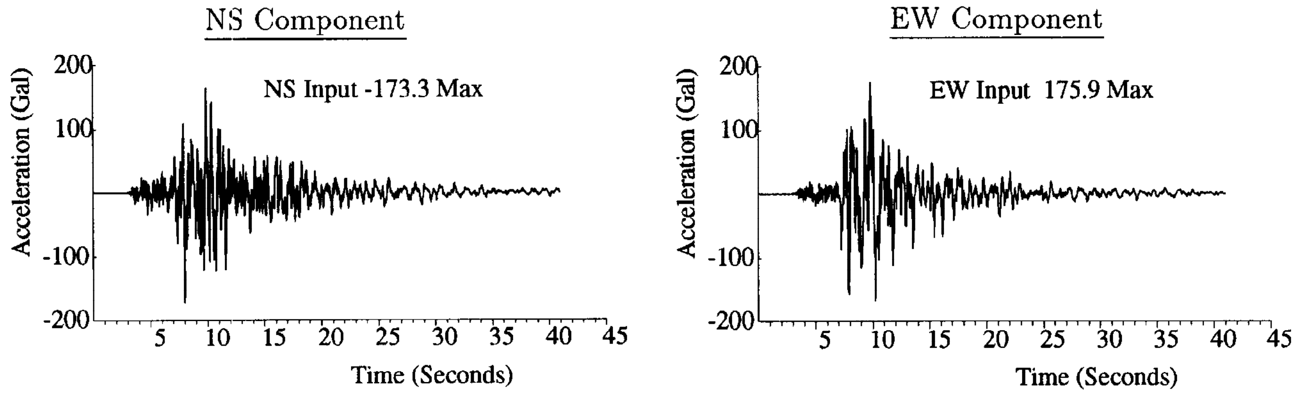
The earthquake motion recorded at a underground rock layer 15.0m deep in the Kita-ku of Kobe city about 34km from the epicenter was utilized to estimate the input motion. The recording station is about 32km from the site. Scaling of the recorded motion to account for the distance from the earthquake hypocenter located about 14km deep at Awaji island was attempted based on the attenuation relation for peak horizontal acceleration (Fukushima *et al.*, 1990). Assuming the attenuation relation to be valid for the subsurface peak acceleration, the input motion at the Ibaraki site was scaled to 82% of the recorded motion. The scaled NS and EW components of the input motions are shown in Fig. 3.

Nonlinear Response Analysis and Results

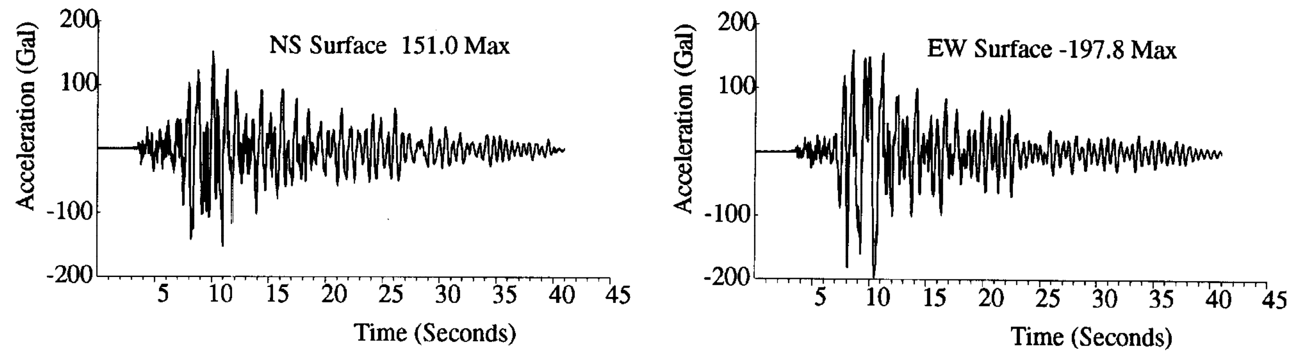
The subdivided soil layers in Fig. 4 were represented by a series of lumped masses connected by shear springs and dashpots, whose characteristics were determined by the nonlinear model described above. The nonlinear time-domain response analysis was carried out by a step-by-step numerical integration procedure to solve the incremental equations of motions considering vertically propagating shear waves. The procedure used was the Wilson's θ-method developed by Ohsaki (1982).

The response time history at ground surface are shown in Fig. 3(b). The acceleration response spectra at 5% damping for base input and surface response motions are compared in Fig. 3(c). The initial fundamental ground period of the site estimated based on the V_s profile and the discretization shown in Fig. 4 was about 0.55 seconds. Owing to the fairly soft nature of the ground, the ground period is likely to further elongate as a result of decreased shear stiffness with the increase in the level of excitation (Karkee *et al.*, 1993). This is seen to be adequately indicated by larger spectral ordinates in Fig. 3(c) at periods longer than about 0.5 seconds.

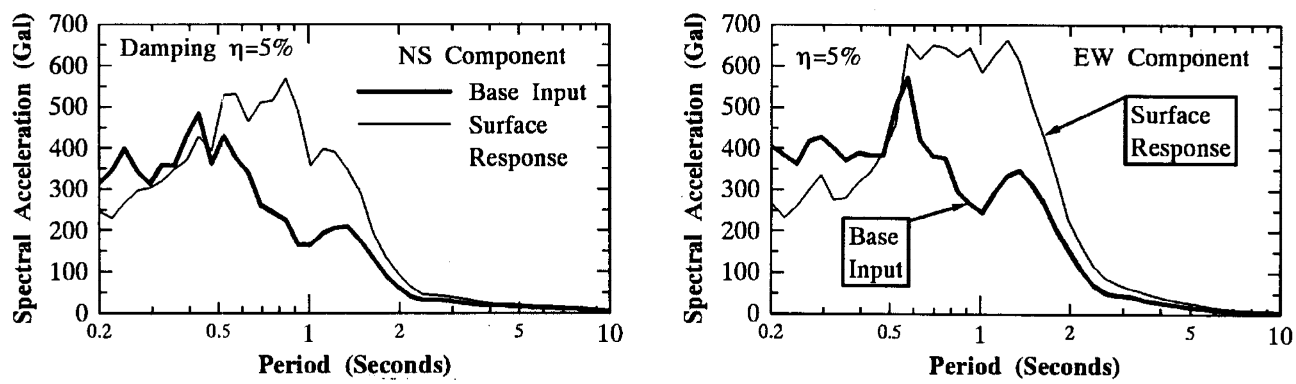
The peak values of the acceleration, the displacement relative to the base, and the shear strain across depth obtained from the free-field response analysis are shown in Fig. 4 along with the soil profile and the relative arrangement of the drilled shaft. It can be noted that the EW component results in larger



(a) Input Motion at Base Layer Obtained by Scaling the Recorded motion



(b) Motion at Ground Surface Computed by Nonlinear Response Analysis



(c) Acceleration Response Spectra for Input and Computed Motions

Fig. 3 Time Histories and Response Spectra of Input and Computed Motions

peak values, and that the soft clayey layers undergo larger relative displacement resulting in larger peak values of shear strain.

SOIL-PILE INTERACTION ANALYSIS

Although there was no superstructure supported by the drilled shafts when the area was hit by the earthquake, it is difficult to be sure whether the shafts were moving together with the soil during ground shaking. So an attempt was made to evaluate the extent of the dynamic soil-pile interaction involved. The method developed by Sugimura (1973), where the soil pile interaction spring is derived from Mindlin's (1936) second solution considering a rectangular section of pile-soil system, was used. The effective soil mass and damping coefficient were determined by assuming that the kinetic energy and the energy dissipated by the model to be equal to those of the actual soil around the pile in the envelop. The Young's modulus was estimated from G_0 considering a Poisson's ratio of 0.5. It should be noted that the soil-pile interaction part of the analysis is based on the assumption that the system behavior is linear. Both ends of the drilled shaft were assumed to have a pinned end conditions.

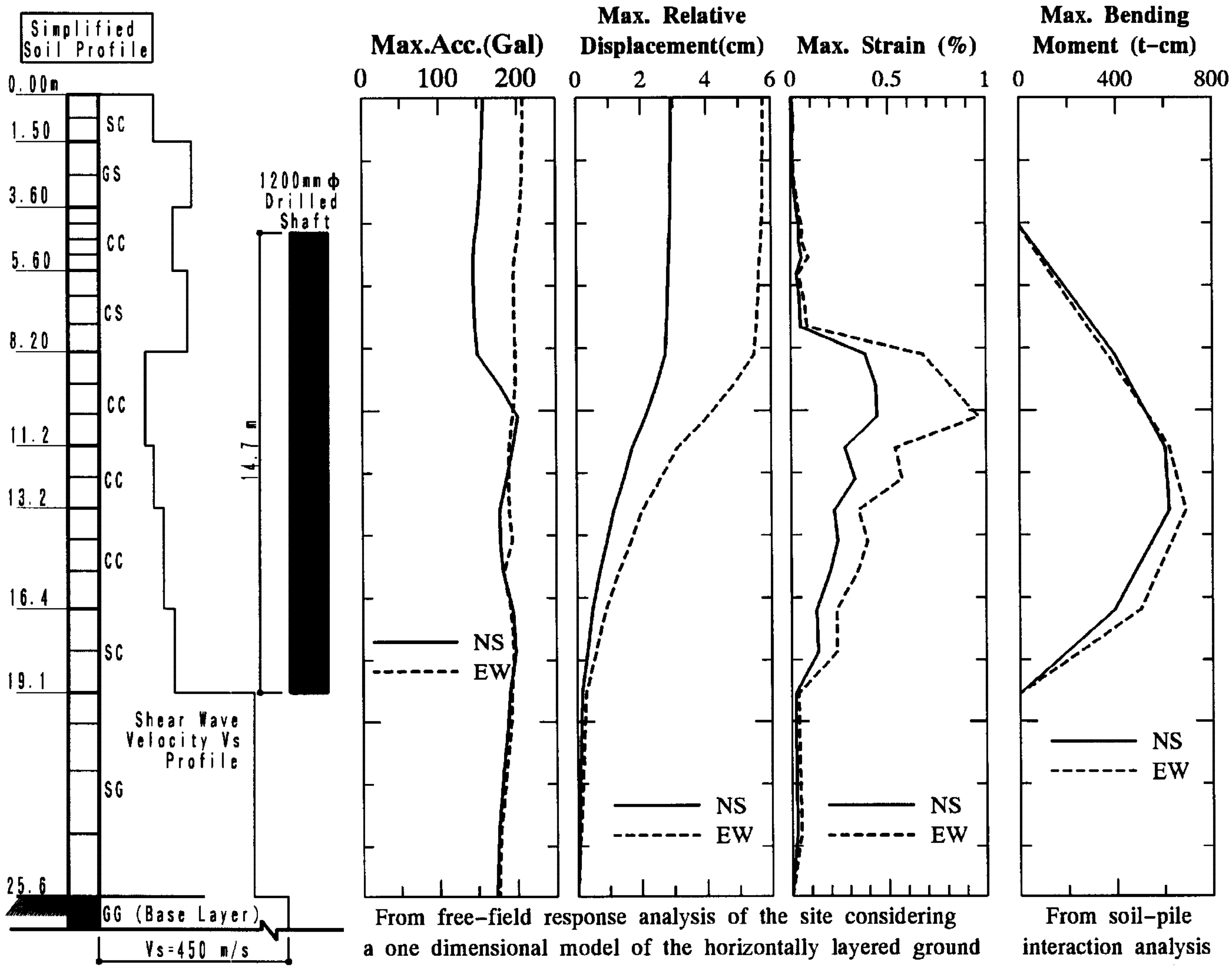


Fig. 4 Peak Values of Acceleration, Displacement, and Shear Strain from Nonlinear Free-field Response Analysis and the Maximum Bending Moment in the Drilled Shaft from Interaction Analysis.

Results of the Interaction Analysis

The envelop of the maximum bending moment estimated based on the interaction analysis is shown in Fig. 4. It can be noted that the bending moment under the action of the EW component is slightly larger. The maximum bending moment is about 680ton-cm. Based on this, the maximum bending stress works out to be about $4.0kg/cm^2$. The design compressive strength of concrete used in the drilled shaft was $300kg/cm^2$ and the 28 day sample compressive strength was of the order of $350kg/cm^2$. Considering that the concrete was designed to have high early strength, and assuming the concrete maturity function (Mehta *et al.*, 1993) for cold weather construction to be applicable, it was estimated that the concrete might have gained about 40% of the 28 day strength ($140kg/cm^2$) in the three day maturity period when the earthquake struck. From this consideration, the maximum instantaneous bending tension acting on the drilled shaft during the earthquake would be only about 3% of the compressive strength, which is quite small showing that the mid-range method adequately indicated the integrity of the drilled shafts during the field test.

CONCLUDING REMARKS

The mid-range method was successfully utilized to assess the integrity of drilled shafts installed just before the site was shaken by earthquake with a JMA intensity of 4 to 5. The integrity of the two piles tested was verified. The equivalent bearing resistance mobilized in the test was quite small, but it was about 200 times the hammer weight. The nonlinear response analysis of the site together with the interaction analysis of the drilled shaft and the soil system showed that the pile was not adversely exerted by the earthquake shaking indicating the adequacy of the prediction made based on the test by mid-range method. The mid-range method in combination with the interaction analysis was found to be very effective in the post-earthquake assessment of the drilled shafts for possible adverse effects that are not readily discernible otherwise.

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