第 1 回 構 造 物 の 破 壊 過 程 解 明 に 基 づ く
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ON SEISMIC FAILURE OF RC BRIDGE PILE FOUNDATION IN VIEW OF NONLINEAR PILE-SOIL INTERACTION

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1995年1月17日に発生した兵庫県南部地震では，阪神高速道路の橋脚だけではなく，杭基礎にも多くの破壊が発生した．強震時には，上部構造，杭，地盤のいずれも，非線系挙動を示す可能性がある。本研究の目的は，地盤と構造物の相互作用が杭の挙動に与える影響を調べることである．その数値解析結果から，（1）杭頭付近および層境界に強い非線形挙動が集中していること，（2）圧縮軸力と曲げモーメントの相互作用により杭が破壊する可能性が高いこと，（3）杭頭における最大応答には上部構造（慣性力）と地盤（kinematic）の両方が影響すること、が判明した。

***Key Words** : pile foundation, soil-structure interaction, 2-D nonlinear analysis*

1. INTRODUCTION

The 1995 Hyogo-ken Nanbu brought serious damages to many RC structures, especially the highway bridge piers. Not only pier bodies but also their piles were seriously failed. This behavior indicates clearly that the effect of the interaction between pier, piles and soil can not be ignored in the analysis and the inelastic behavior is an inherent factor to prescribe the complete system response during severe loading conditions. The behavior of bridge-pile system depends on many factors, such as the characteristics of seismic excitation, the fundamental periods of the seismic excitation, soil and structure. The nonlinear behavior is attributed to the big inertial forces and/or substantial ground deformations depending upon system properties.

Recently, the seismic response of pile-supported bridge has been a subject of considerable research effort. However, much is yet to be learned on this issue before a complete understanding of its seismic response. Hence, the objective of this study is to clarify the role of the above mentioned nonlinear effects on the seismic response of bridge-pile system. This paper presents some useful results from the analysis of a typical RC Pilz type bridge of the Hanshin Expressway.

2. METHODOLOGY

A 2-D nonlinear soil-structure interaction analysis is performed by the time domain FEM-BEM hybrid technique developed by Takemiya and Adam[1)]. The FE region is treated as the non-homogeneous nonlinear zone while the BE region is considered as linearly elastic domain. In the model, the deeper soil is modeled by BEM, pier and piles are discretized by beam elements, the neighboring soil by FEM, and the vertical boundary is offset by a substantial distance from the area of interest.

The inelastic behavior of a beam is represented by one component model with the consideration of sway motion at both ends of each element as was formulated by Takemiya and Shimabuku[2)]. The RC hysteresis model is represented by the Q-hyst, which was modified by Shimabuku and Takemiya[3)] to take into account the relationship between bending moment and axial force. The soil nonlinear behavior is characterized by the Mohr stress circle criterion and the Hardin-Drnevich model.

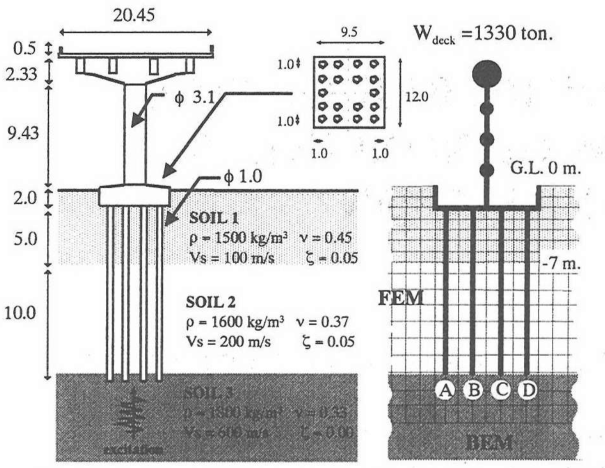

Fig.1 The Hanshin Highway and its model for analysis.

Table 1 Properties of concrete and reinforcement.

CONCRETE	Compressive strength (σ_{ck})	270 kgf/cm^2
	Modulus of elasticity (E_c)	2.8 x 10^5 kgf/cm^2
	Strain under maximum compression stress (ε_{cc})	3.0 x 10^{-3}
	Ultimate strain of restrained concrete (ε_{cu})	3.6 x 10^{-3}
STEEL	Yield strength (σ_{sy})	3500 kgf/cm^2
	Modulus of elasticity (E_s)	2.1 x 10^6 kgf/cm^2

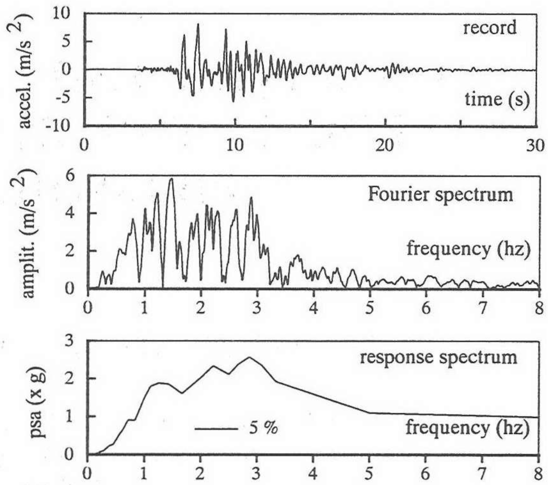

Fig.2 Kobe-JMA-NS acceleration.

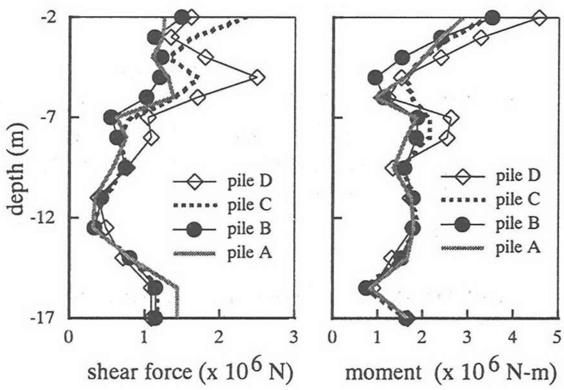

Fig.3 Maximum responses of piles.

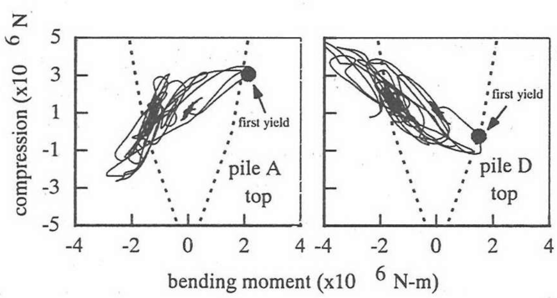

Fig.4 Bending moment-axial force relationship at piles.

3. MODEL AND RESPONSE ANALYSIS

Fig.1 illustrates the analyzed structure-soil system and their modeling, based on a typical Pilz type bridge of the Hanshin Expressway. The constitutive properties of piles are given in **Table 1**. The yielding points of the bending moment-axial force interaction diagram are derived using the conventional reinforced concrete theory. The motion used in the analysis is the Kobe-JMA-NS record. **Fig.2** shows this record, its Fourier spectral density and the response spectrum of 5% damping ratio.

The maximum shear forces and bending moments along the length of pile are depicted in **Fig.3** (the location of each pile was indicated in **Fig.1**). At pile top, the inner piles attains the larger shear force than the outer piles; at other depths, however, the behavior is inverse. The reason for this is that the inertial shear force developed on the superstructure is proportionally transmitted onto each pile top and that the presence of soil interface with significantly different layer properties produces a sharp variation of soil strains, which results in the maximum shear forces of the outer piles at this zone. With respect to the bending moment, we can observe that the maximum bending moment at pile top corresponds to the pile D and the minimum to the pile A, which coincides with the maximum compressional and tensional force, respectively, as can be recognized in **Fig.4**. In this figure, the dot lines indicates the yielding state.

The bending moment-rotation is significant at pile top and at the interface of soil layers. At these locations, the RC nonlinear behavior appears clearly as can be observed in **Fig.5**.

Fig.6 shows the dissipated energy, the rotational ductility and the Park-Ang damage function (DPA). The ductility is calculated with respect to the first yielding of longitudinal reinforcement, the dissipated energy from the bending moment-rotation hysteresis and DPA with the parameter β equal to 0.15. We can see that the pile D has larger values than the pile A, which confirms the predominance of bending moment-compressional force relationship in the possible failure of piles.

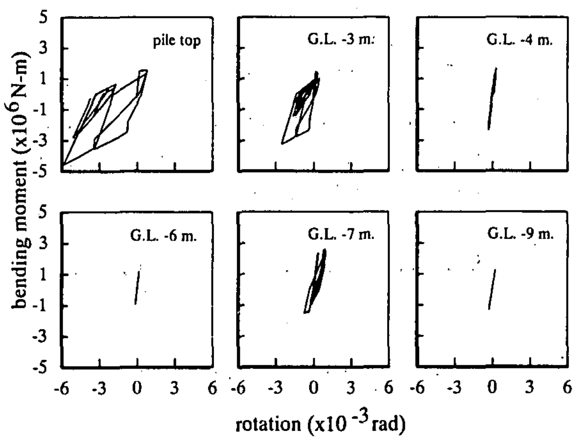

Fig.5 Bending moment-rotation hysteresis of pile D.

The total dissipated energy by 18 piles at their heads is equal to the dissipated energy at pier base; but this balance is not for ductility values. We can see that the DPA has the same shape to ductility , which reflects the impulsive character of excitation. Therefore, the ductility factor is the principal parameter to take into account to prevent the failure of piles during impulsive excitations.

The following figures demonstrate the relationship between period of excitation, soil and structure. The fundamental periods of the fixed-based superstructure and the free field are 0.52 s.(1.93 Hz.) and 0.32 s.(3.08 Hz.), respectively; both of which are calculated by the 1-D idealization. According to the response spectrum (see **Fig. 2**), the maximum responses are expected for components at frequencies around 2.2 and 3.0 Hz. Therefore, a considerable contribution from soil and structural dominant modes to the response are expected. The Fourier transform of pile top and the free field surface accelerations are presented in **Fig.7**. A comparison with complete linear analysis is presented to check the variation of frequency contents in the response due to the nonlinear behavior. According to the linear analysis, the peaks at pile top appear around 2 and 3 Hz., which is very near to the superstructure and soil fundamental frequencies. However, in the nonlinear analysis, the peaks appears at low frequencies (around 1.2 Hz) due to the shift of energy by the inelastic behavior.

In order to detect the transient feature of nonlinear response, the variation of frequency contents with the time are shown in **Fig.8**. This was performed by wavelet transform application; herein the Daubechies 10^{th} order wavelet function was used. This analysis extracts the frequency content with time at different levels, where the lower and higher levels corresponding respectively to the high and low frequencies involved. The acceleration amplitudes were normalized with respect to the maximum value for a convenience of visualization. The Kobe-JMA-NS presents peaks at level 8 (1.56 to 3.125 Hz) around 7.6 and 9.6 sec., which corresponds to 2

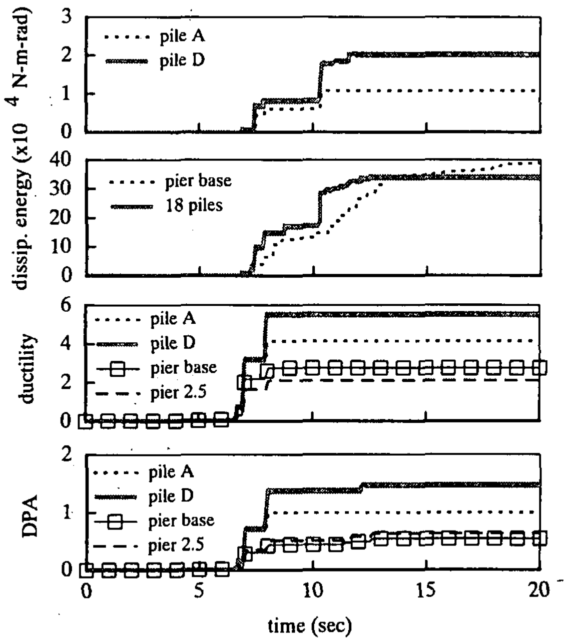

Fig.6 Dissipated energy, rotational ductility and damage factor.

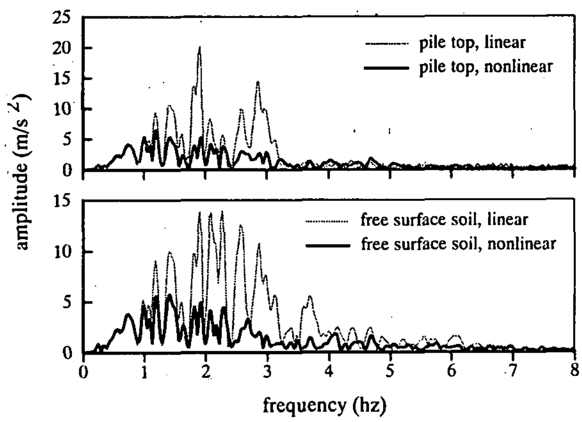

Fig.7 Fourier amplitudes, linear and nonlinear response.

peaks of the record. If the linear and nonlinear responses are compared, we note that the peaks in the linear analysis appear at the level 8; whereas at the level 9 (0.78 to 1.56 Hz.) in the nonlinear analysis. It clearly indicates the variation of frequency contents due to the inelastic behavior which starts around 7 s. for the structure. The pattern of the free surface soil and pile top figures is very similar, which induces a strong soil effect in the pile top response. To clarify this point the soil(kinematic) and superstructure(inertial) interaction effects are revised. Their superposition is not exactly applied for the nonlinear behavior, but as an engineering approximation may be accepted. **Fig.9** shows the acceleration and displacement at the free surface soil and the pier top. We can appreciate that both responses give arise to an in-phase behavior, which apparently result in a constructive interplay between the soil and superstructure. **Fig.10** depicts the relationship between above responses; here, we can see that also at pile top the kinematic effect is predominant over the inertial effect. The reason of this

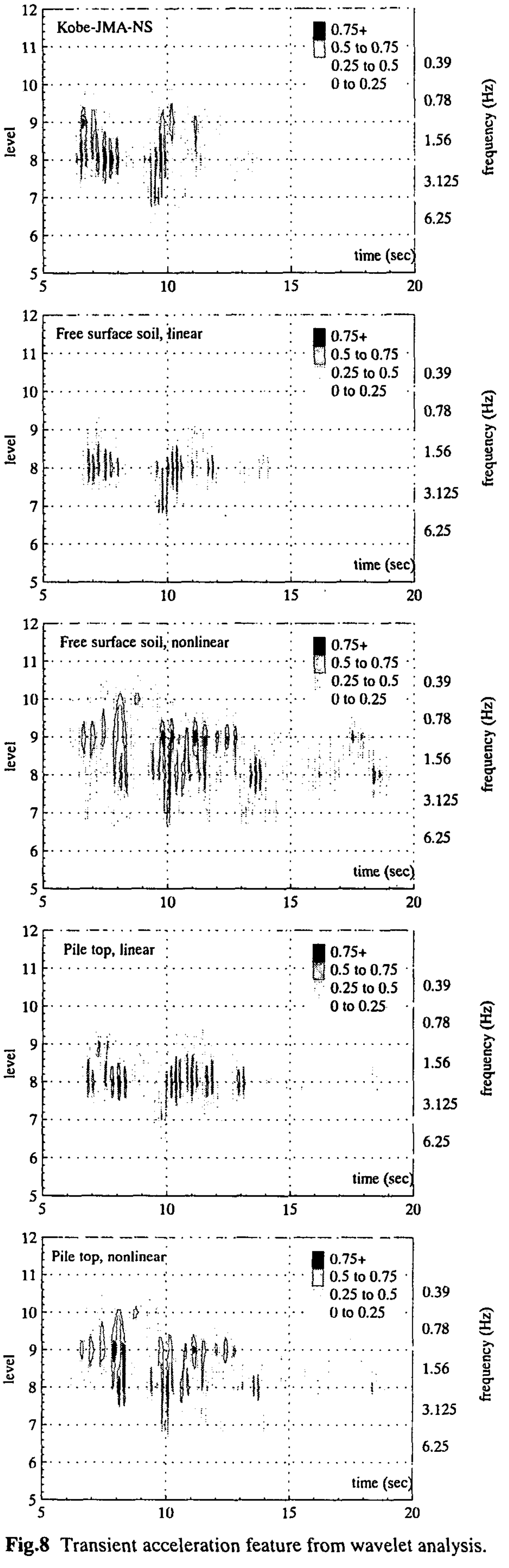

Fig.8 Transient acceleration feature from wavelet analysis.

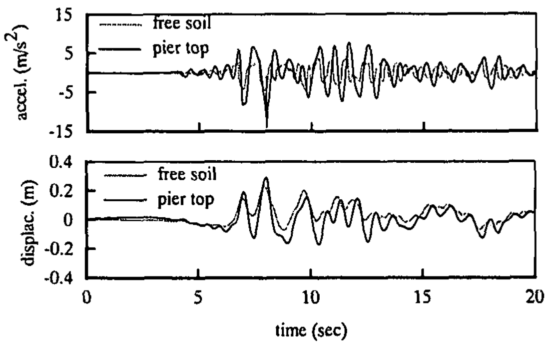

Fig.9 Absolute acceleration and displacement responses.

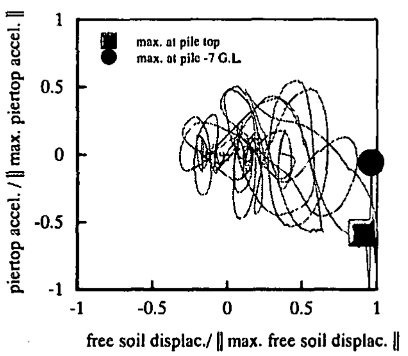

Fig.10 Comparison between inertial and kinematic effect.

behavior may due to softer superficial soil, which produces a lower acceleration at pier top.

4. CONCLUSIONS

(1) In case of pulse type destructive seismic motion, the ductility is the main parameter to take into account to prevent the failure of piles before to pier.

(2) The contained frequencies in the response are dominant by the inelastic behavior over the natural periods of soil and structure.

(3) When an inelastic behavior in conjunction with softer soil are presented; apparently, the kinematic effect have a predominance over the inertial even at pile head.

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