EFFECTS OF SOIL-PILE MODELING ON SEISMIC RESPONSE OF RC BRIDGE PIER

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This paper investigates the effects of the soil-pile interaction modeling on the seismic response of RC pier structures. The Q hysteresis model with inclusion of shear degradation effect is used to represent the RC nonlinear behavior. The modified Ramberg-Osgood hysteresis is assumed for the stress-strain relationship of soils. The far field soil analysis is independently conducted to evaluate the input motion to the near field soil-pile system. Special attention is paid to the response of the superstructure due to the nonlinear behavior of soil and piles in the interaction.

Key Words : *shear degradation, soil-pile interaction, moment-curvature, ductility*

1. INTRODUCTION

Seismic performance of the RC Pilz type bridges has drawn a considerable engineering attention after the devastating damage of the Hanshin Highway viaduct due to the Hyogo-Ken Nanbu Earthquake (January 17, 1995). The Hanshin Highway viaducts at that site were supported by cast-in-place reinforced concrete piles. In this paper, two models are used for the analysis: one takes account of the pile-soil interaction by a set of equivalent linear springs calculated as impedance functions(1) at the footing, and the other considers the total interaction of the sub- and super-structure. In case of strong earthquake motions, soils indicates heavily nonlinear behavior from two different mechanisms: one is due to the soil vibration itself and the other is due to the pile deformations to produce an additional nonlinear soil response. The complete model analysis assumes the independence of the soil vibration as a far field and the pile-soil interaction in the near field.

2. RC NONLINEAR MODEL

The RC nonlinear behavior is represented by the one component type of nonlinear beams(2), in which each member is specified by an elastic beam element between inelastic hinges at the both ends. **Fig. 1** shows the nonlinear Q hysteresis model(3) with the inclusion of the strength degradation (function of degradation parameter β (4)) and the shear degradation effect by lowering the target maximum or minimum point to the level of cracking force(5) until the crack-closing point is reached after which the target is the previous maximum or minimum point.

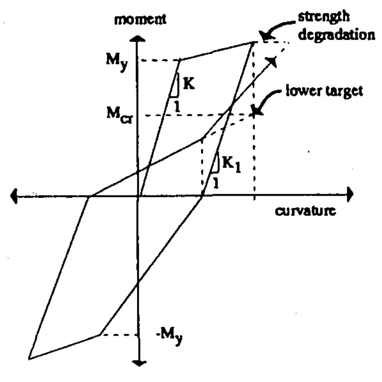

Fig. 1. RC hysteresis model

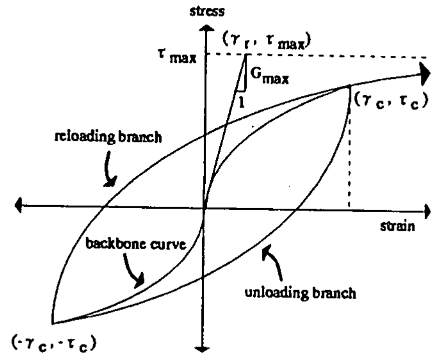

Fig.2. Modified Ramberg-Osgood model

3 . SOIL NONLINEAR MODEL

Fig.2 shows a modified Ramberg-Osgood backbone curve and the hysteresis loop constructed based on the so-called Masing's assumption, which regulates the unloading and reloading on the branch curve varies as the half rate of that of the original backbone curve[6]. The equation for the backbone curve and the reloading or unloading branches are respectively:

$$\gamma = \frac{\tau}{Go}\left[1+\alpha|\tau|^{\beta}\right] \tag{1}$$

$$\gamma \pm \gamma_c = \frac{\tau \pm \tau_c}{G_0}\left[1+\alpha\left|\frac{\tau+\tau_c}{2}\right|^{\beta}\right] \tag{2}$$

$$\alpha = \left[\frac{2}{G_0\gamma_r}\right]^{\beta} \qquad \beta = \frac{2\pi h_{max}}{2-\pi h_{max}} \qquad (3), (4)$$

in which G_0 is the initial tangent shear modulus, h_{max} is the maximum equivalent damping factor, and γ_r is the reference strain when $G/G_0 = 1/2$.

4 . FAR FIELD ANALYSIS

At large distance from pile foundations, soils are less affected by the motion of these piles, and the one dimensional wave propagation is adequately assumed for the behavior of layered soil deposits. The nonlinear mechanism is introduced according to the statement of section 3. The equivalent linearization procedure by an iterative scheme is used for the analysis, adjusting the material properties to be compatible with the strain level. The curves of the variation of the shear modulus and damping ratios with strain level are given by

$$\frac{G}{G_0} = \frac{1}{1+\left\{2\left(\frac{\gamma}{\gamma_r}\right)\left(\frac{G}{G_0}\right)\right\}^{\beta}} \tag{5}$$

$$h_{eq} = h_{max}\left[1-\frac{G}{G_0}\right] \tag{6}$$

5 . NEAR FIELD ANALYSIS

The soil reaction to the motion of piles is described by the Wrinkler type soil modulus[7] in lateral and longitudinal directions respectively by

$$E_h = 2\pi GkB\left[\frac{\frac{1}{S}H_2^{(2)}(kB)\ H_1^{(2)}(hB) + H_2^{(2)}(hB)\ H_1^{(2)}(kB)}{H_0^{(2)}(kB)\ H_2^{(2)}(hB) + H_0^{(2)}(hB)\ H_2^{(2)}(kB)}\right] \tag{7}$$

$$E_v = 2\pi GkB\left[\frac{H_1^{(2)}(kB)}{H_0^{(2)}(kB)}\right] \tag{8}$$

in which $H_i^{()}$ denotes the modified Bessel functions of the i^{th} order, B is the radius of a pile, s is the ratio of shear wave velocity to dilatational velocity, and h and k are the ratio of angular frequency to dilatational and shear wave velocity respectively.

The governing equation of motion is expressed by

$$M_p\ddot{u}_p + C_{p+s}\dot{u}_p + K_{p+s}u_p = -M_p[1]\ddot{u}_g \tag{12}$$

in which the subfix p correspond to pile and s to soil. The Eq. (12) is solved by the modified Newton Raphson method for the nonlinear response. The nonlinear soil reaction is assigned to the real part of Eq. (7), where the shear modulus is evaluated by the Ramberg-Osgood type hysteresis with a adjusted reference strain γ_r' by multiplying the original reference strain used for the far field analysis by the real part of E_h/G[8]. The soil strain is obtained by the relative displacement of pile and far field along pile depth L_i as

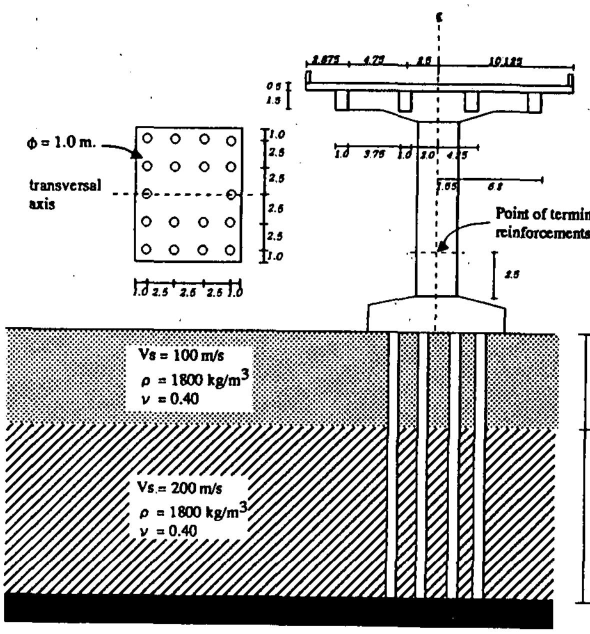

Fig. 3.Hanshin Highway pier bridge (B505)

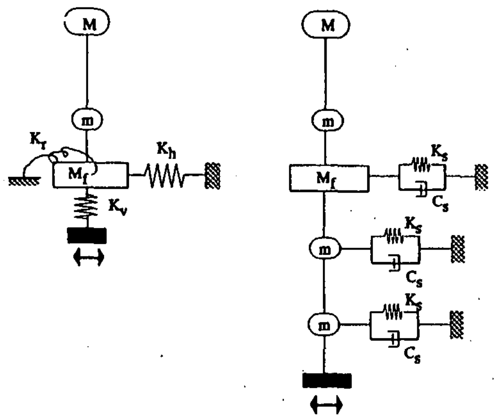

Fig. 4. Simple and total models

$$\gamma_i = \frac{\left[u_{p_{i+1}} - u_{s_{i+1}}\right] - \left[u_{p_i} - u_{s_i}\right]}{L_i} \tag{13}$$

6. ANALYSIS OF TYPICAL HANSHIN HIGHWAY PIER

Fig. 3 shows a typical Pilz type pile-supported bridge of Hanshin Highway and the site condition. **Fig. 4** shows the models for analysis; the model with a set of equivalent linear springs which is called an simple model (SM), and the model with the inclusion of grouped piles which is called an total model (TM). For both models the pier is assumed to behave nonlinear, and the TM model considers nonlinear pile and soil behavior.

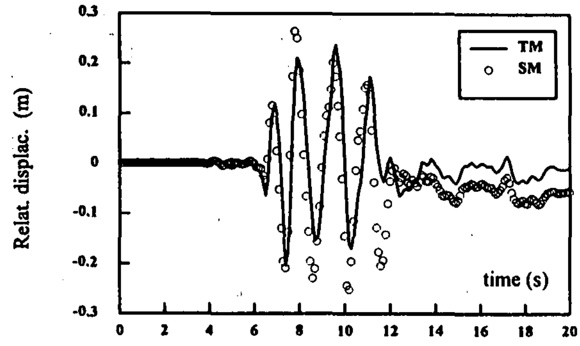

Fig. 5. Pier top displacement

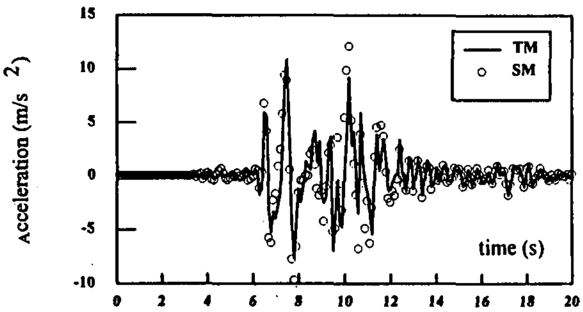

Fig. 6. Pier top acceleration

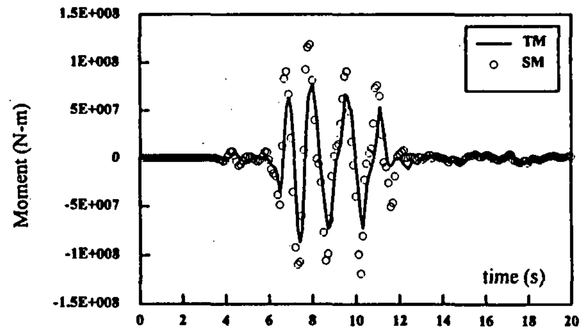

Fig. 7. Moment at base of pier

The structure was excited by the Hyogo Ken Nanbu earthquake motion, JMA-NS component.

Fig. 5 and **6** are the pier top displacement relative to the footing and the acceleration time histories. **Fig. 7** is the bending moment at the base of pier. The effect of the nonlinear soil and pile interaction is obvious due to the shift of the predominant period. The TM attains the maximum displacement when the second strong response occurs at 9.6 sec., while the SM at 7.9 sec. This trend is most conspicuous after the shear cracks appears.

Fig. 8 and **9** show respectively the time histories for the relative displacement and the acceleration at the footing; in the model TM after 12 seconds in time history we can observe oscillations, which reflect the long predominant period of the structure at the final response stage.

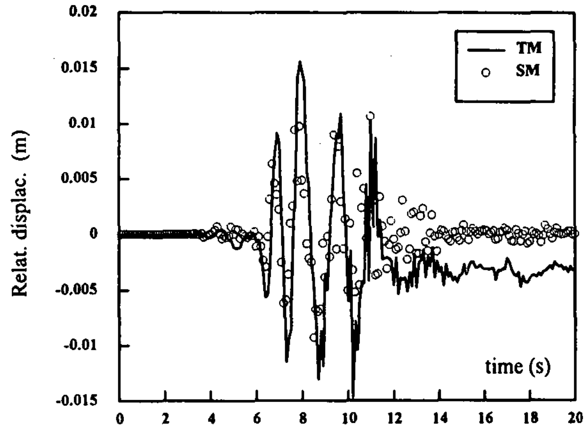

Fig. 8. Footing displacement

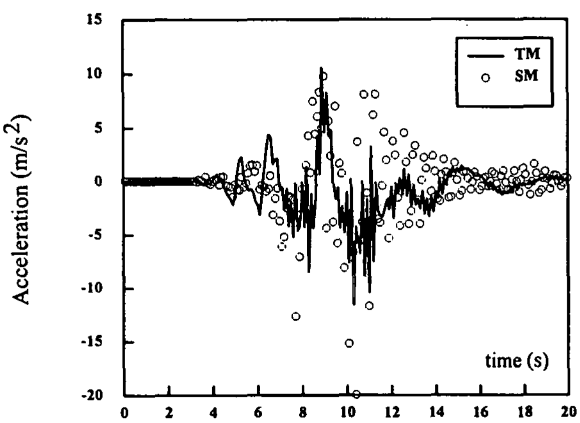

Fig. 9. Footing acceleration

Fig. 10 is the moment curvature relationship in the first 10 seconds at the base of the pier for the TM. The ductility is noted approximate 4, which is bigger than the value of 3.2, calculated by the seismic design code at construction time.

Fig. 11 is the moment curvature relationship until 10 seconds at the head of the inside pile, for which the interpretation of the internal force and the load distribution factor(9) are made. This pile present a ductility of approximate 3.6.

In this Pilz type bridge,it is noted that the soil pile interaction is strongly affected by the shear cracks and the P- Δ effect in the final collapse.

7 . CONCLUSION

Conventionally, a set of equivalent linear springs are employed to represent pile head impedance at footing, these only reflect the inertial interaction between the foundation and the structure. In case of strong earthquakes, the pile deformation with respect to the soil deformation becomes important. The difference of

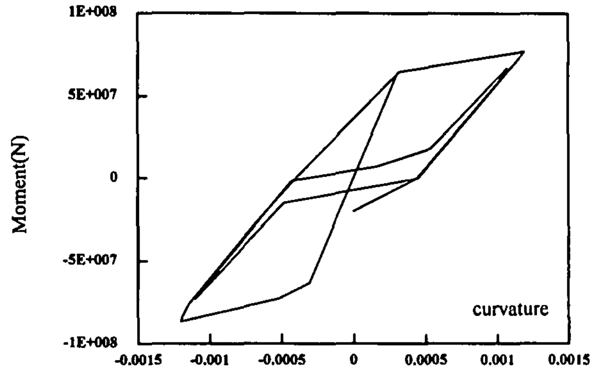

Fig. 10. Hysteresis at base of pier

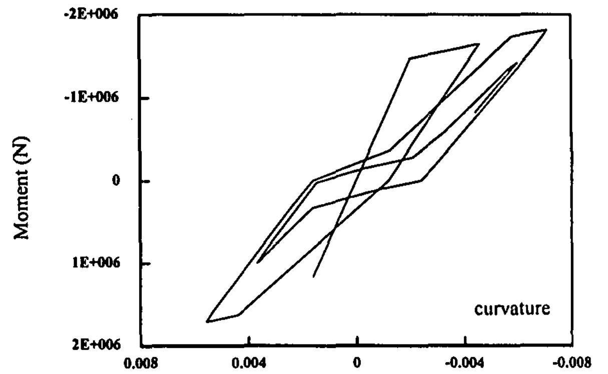

Fig. 11. Hysteresis at head of pile

response between these modeling are clarified through the illustrative computation.

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