The 14th World Conference on Earthquake Engineering
October 12-17, 2008, Beijing, China

INVERSION OF DYNAMIC HYSTERETIC NONLINEAR PROPERITY OF SITE SOIL IN TIME DOMAIN

CHEN Xue-liang[1], JIN Xing[2] and TAO Xia-xin[3]

[1] *Ph. D, Division of Engineering Earthquake and City Disaster Reduction, Institute of Geophysics, China Earthquake Administration, Beijing. China*

[2] *Professor, Earthquake Administration of Fujian Province, Fuzhou. China*

[3] *Professor, Dept. of Civil Engineering, Harbin Institute of Technology, Harbin. China*

Email: cxl@cea-igp.ac.cn, xueliang_chen@yahoo.com.cn

ABSTRACT :

The inversions of hysteretic nonlinear behavior in time domain of site soils are extremely rare at home and abroad for the complexity of soil dynamic nonlinear constituted relations, low efficiency of implicit integral formula, and low performance of optimized method. In view of these questions, this paper using soil dynamic double- parabola constitute model, which can consider the soil test damping ratio and can character main characteristic of soil's hysteretic constituted relationship, associating with the highly effective explicit integral method with the high accuracy computation acceleration response, adopting global-local optimization technique, which uses genetic algorithm(GA)-simplex method, basing on M_J7.1 and the M_J8.0- two typical big earthquake records at TKCH07 borehole array in Japanese Kik-net network, the hysteretic nonlinear characteristic in time domain of the cohesive soil and the sand at TKCH07 borehole array are inversed and analyzed. The results indicated that, the modulus and the damping ratio of cohesive soil and the sand are basically consistent with the lab test results of Seed-Idriss and Kokusho. The mechanism and the history of hysteretic nonlinear constituted relationship under seismic strong motion are reproduced. The results reveal the permanent displacement mechanism. Simultaneously, velocity history and displacement history of ground strong motion are forecasted and estimated.

KEYWORDS: hysteretic nonlinear, time domain, soil dynamic property, inversion analysis

1. INTRODUCTION

As we all know, the equivalent linearization method can only get one modulus (secant shear modulus) and one damping ratio in the inversion analysis for an earthquake. For the whole strong motion history, it is linear only with the concept of equivalent nonlinear. Therefore, the equivalent linearization method in the qualitative revelation soil nonlinear is feasible, but, quantitative analysis of soil nonlinear is only an expedient measure.

Using the evolution power spectrum analysis method to separate the simplified body waves, by continuously regression analysis and the Kalman filter technique, Loh and Yeh(1992) carries on the modal parameter recognition to the site soil system (inversion analysis), the soil is supposed bilinear, the two soil stiffness and the small yield displacement are obtained. However, after all, the bilinear model is quite simple.

In fact, it is quite different for the inversion method to get all nonlinear dynamic properties parameters of soil, even if a set of parameters is obtained, this inversion will be due to too many parameters and reduce its reliability. Moreover, the inversion calculation is not the only solution but the optimal solution, and even, if the constraints are too loose, the optimal solution may not be the real solution. As a result, meet the following requirements are necessary. The requirements of site condition: Choose clear soil profile, simple typical site. The requirements of known parameters: According to the drill-log, the geological investigation material, and the soil static test easy to obtain some parameters are the known parameter, and the small strain shear velocity, thickness, the density of borehole array site are known. Strong earthquake observation requirements: high-quality earthquake records; different depths of the measuring point is abundant, at least, there are observation apparatus in the bedrock and the surface, and two or more times earthquake records are gotten. To meet these conditions, the inversions of the soil dynamic nonlinear parameters are more real and reliable.

Soil dynamic nonlinear constitutive relations is studying and developing, moreover the inversion calculation need many times of plus analysis, therefore, plus analysis which is nonlinear seismic response of site in time domain must be highly effective. The time domain explicit recursion's nonlinear analysis method obviously has time-saving, the highly effective merit gradually, Explicit gradually recurrence nonlinear analysis method in time-domain have significant time-saving and efficiency advantages, which makes inversion hysteretic nonlinear in time-domain possible. Using soil dynamic double-parabola constitutive model which proposed by CHEN Xue-liang, JIN Xing et al.(2008) and the basic laws and experimental results of soil test, by hybrid optimization algorithm which is the combination of genetic algorithm - simplex method (Hanwei, 2000) , a way of the inversion of soil dynamic nonlinear characteristics is attempted and explored.

2. SOIL TEST ACHIEVEMENTS AND KEY PARAMETERS DISCUSSION

The key parameters of soil test which are strain-dependent shear modulus ratio $G_d/G_{\max}\sim\gamma$ and damping ratio $\lambda\sim\gamma$ are discussed. Usually, the hyperbolic curve skeleton curve form of Hardin-Drnevich model is widely recognized and approved by domestic and foreign soil dynamics experts and geotechnical engineers in.

$$\frac{G_d}{G_{\max}}=\frac{1}{1+|\gamma|/\gamma_r} \qquad (2.1) \qquad\qquad \gamma_r=\frac{\tau_{ult}}{G_{\max}} \qquad (2.2)$$

And, $G_{\max}$ and γ_r respectively are the biggest shear modulus and reference shear strain. τ_{ult} is soil ultimate stress. The key is the determination of reference shear strain, if γ_r is known, the soil's $G_d/G_{\max}\sim\gamma$ can be obtained. By equation (2.2) know, γ_r with related to the soil largest shear modulus $G_{\max}$ and the final ultimate strength τ_{ult}. The research of Hardin etc. indicated that(Hardin and Drnevich, 1972a, 1972b), τ_{ult} with related to the initial stress and the way of shear stress, for the initial stress and shear stress action on the surface, τ_{ult} is also controlled by soil stress envelope, as shown in Figure 1, ultimate shear stress τ_{ult} may be calculated as following t(Hardin and Drnevich, 1972a, 1972b),

$$\tau_{ult}=\left[\left(\frac{1+K_0}{2}\bar{\sigma}_v\sin\bar{\phi}+\bar{c}\cos\bar{\phi}\right)^2-\left(\frac{1-K_0}{2}\bar{\sigma}_v\right)^2\right]^{1/2} \qquad (2.3)$$

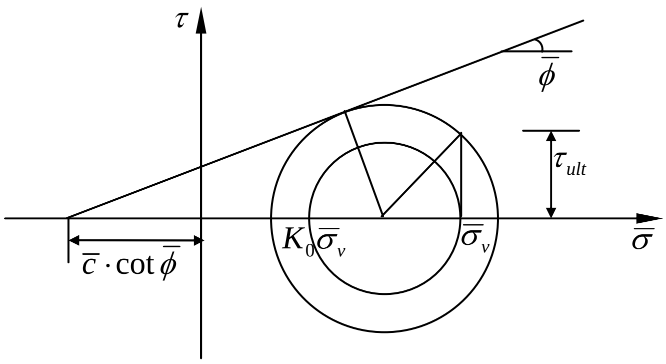

Fig.1 Ultimate shear stress τ_{ult}

In above formula, K_0 is the static coefficient of earth pressure, $\bar{\sigma}_v$ is vertical effective stress, $\bar{c}$ and $\bar{\phi}$ are respectively effective soil cohesion and effective internal friction angle, when the pore water is not considered, this is the total stress case. According to past results and engineering experience, by the soil type, this kind of soil corresponds $\bar{c}$, $\bar{\phi}$, ρ and the poisson ratio v's approximate scope, may be known. By the poisson ratio, $K_0=\dfrac{v}{1-v}$ is estimated. This would achieve a rough τ_{ult} range.

Generally, The shear modulus measured in soil dynamic test When $\gamma\le 0.25\times10^{-4}$ is regarded as G_{max}, and there is $G_{max}=\rho c_s^2$, c_s is shear velocity. In this paper, c_s is known. Regarding the damping ratio research, the following form is widely applied in engineering practice (ZHANG Ke-xu, XIE Jun-fei. 1989),

$$\lambda=\lambda_{max}\left(1-\frac{G_d}{G_{max}}\right)^M \qquad (2.4)$$

According to the achievement of Hardin(Hardin and Drnevich, 1972a, 1972b)and soil dynamics(ZHANG Ke-xu, XIE Jun-fei. 1989), λ_{max} can be given some experience of the estimated. M is the test parameters, usually between 0.1 to 1.8. In this way, the curves of $G_d/G_{max}\sim\gamma$ and $\lambda\sim\gamma$ can be controlled by γ_r, G_{max} and M. The above soil experiment achievement and the understanding, will provide the approximate scope of the key parameters, will reduce the inversion space and raises the counting yield and the precision.

3. SOIL DYNAMIC DOUBLE-PARABOLA CONSTITUTIVE MODEL(CHEN Xue-liang, JIN Xing et al, 2008)

Based on the basic characteristic curves $G/G_{max}-\gamma$ and $\lambda-\gamma$ revealed by the results of the soil dynamic tests, the rules of adjustable double-parabola constitutive curves of reverse unload after load process or reverse load after unload process in irregular load history are established by CHEN Xue-liang, JIN Xing et al.(2008). The model can be a good simulation of test damping ratio in the irregular load history.

The equation of soil dynamic double-parabola constitutive model is,

$$\tau(\gamma)=\begin{cases}\dfrac{G_{max}\gamma}{1+|\gamma|/\gamma_r} & |\gamma|\ge\gamma_M\\ (1-A_d)\tau_-^A+A_d\tau_-^B \quad d\gamma<0 & \\ (1-A_d)\tau_+^A+A_d\tau_+^B \quad d\gamma\ge 0 & |\gamma|<\gamma_M\end{cases} \qquad (2.5)$$

Where, supposed $A_d = 0.5$. $\tau_-^B = 2\tau_0 - \dfrac{-b_2 + \sqrt{b_2^2 - 4a_2(c_2 - 2\gamma_0 + \gamma)}}{2a_2}$;

$\tau_-^A = 2\tau_0 - a_1(2\gamma_0 - \gamma)^2 - b_1(2\gamma_0 - \gamma) - c_1$; $\tau_+^B = \dfrac{-b_2 + \sqrt{b_2^2 - 4a_2(c_2 - \gamma)}}{2a_2}$; $\tau_+^A = a_1\gamma^2 + b_1\gamma + c_1$;

$a_1 = -1.5\pi\lambda_T(\gamma_0)\dfrac{\tau_M - \tau_C}{(\gamma_M - \gamma_C)^2}$ $\quad b_1 = \dfrac{(\tau_M - \tau_C)}{(\gamma_M - \gamma_C)}\left(1 + 1.5\pi\lambda_T(\gamma_0)\dfrac{\gamma_M + \gamma_C}{\gamma_M - \gamma_C}\right)$;

$c_1 = \dfrac{\gamma_M\tau_C - \gamma_C\tau_M}{\gamma_M - \gamma_C} - 1.5\pi\lambda_T(\gamma_0)\dfrac{\gamma_M\gamma_C(\tau_M - \tau_C)}{(\gamma_M - \gamma_C)^2}$; $\quad a_2 = 1.5\pi\lambda_T(\gamma_0)\dfrac{\gamma_M - \gamma_C}{(\tau_M - \tau_C)^2}$;

$b_2 = \dfrac{\gamma_M - \gamma_C}{\tau_M - \tau_C}\left(1 - 1.5\pi\lambda_T(\gamma_0)\dfrac{\tau_M + \tau_C}{\tau_M - \tau_C}\right)$; ; $c_2 = \gamma_M - a_2\tau_M^2 - b_2\tau_M$.

γ_C and τ_C was the corresponding strain and the stress of "the inflexion point" in recently load (or unloading) process. (τ_M, γ_M) is the point of the biggest stress and the biggest strain in history, the biggest stress and the biggest strain are always positive. $\lambda_T(\gamma_0)$ is experimental damping ratio.

4. THE NONLINEAR PARAMETERS INVERSION OF SAND AND CLAY SOIL IN BOREHOLE ARRAY TKCH07 SITE

Japan Kik-net seismic network (Kik-net html) provides the detailed material of the borehole array TKCH07 (42.81N, 143.52E) in TOYOKORO site. Figure 2 gives a cross section of TKCH07 borehole. In this borehole array site, three-component broadband seismographs at both the surface and the bedrock are established. Two high-quality seismic 20041129 M_J7.1 and 20030926 M_J8.0 are recorded. The depth of hypocenter of M_J7.1 earthquake is 48km, the epicenter is (42.94N 143.52E), the distance from the station to epicenter is 144km, the surface seismic motion PGA is 125.6gal. Similarly, The depth of hypocenter of M_J8.0 earthquake is 42km, the epicenter is (42.78N 144.08E), the distance from the station to epicenter is 123km, the surface seismic motion PGA is 345.9gal. By calculating, back - azimuths of M_J7.1 earthquake and M_J8.0 earthquake are respectively 85°and 156°.

According to back-azimuth of each earthquake and TKCH07 station, SH-wave fields are isolated from records of the bedrock and surface. At the same time, the usual hypothesis in the domestic and foreign site nonlinear earthquake response, which are bottom rigid boundary condition, the earthquake wave normal incidence, are used. In site profile, 14m thick cohesive soils and the 24m thick sandy soil are only regarded as 2 types nonlinear test curves. The total number of inversion parameters $\gamma_r/10^{-4}$, $\lambda_{\max}$ and M is 6. To reduce uncertainty, geotechnical test parameters of the gravel and bedrock is supposed known, and test parameters of the gravel and the bedrock are listed in table 4.1.

Table 4.1 Strain-dependent shear modulus and damping ratio of gravel and rock in TKCH07 site

Gravel	$\gamma/(10^{-4})$	0.05	0.1	0.5	1	5	10	50	100
	$G/G_{\max}$	0.99	0.97	0.90	0.85	0.70	0.55	0.32	0.20
	$\lambda/\%$	0.40	0.60	1.90	3.0	7.5	9.0	11.0	12.0
Bedrock	$\gamma/(10^{-2})$	0.0001	0.0003	0.001	0.003	0.01	0.03	0.1	1.0
	$G/G_{\max}$	1.00	1.00	0.9875	0.9525	0.900	0.810	0.725	0.550
	$\gamma/(10^{-2})$	0.0001		0.001		0.01	0.1		1
	$\lambda/\%$	0.4		0.8		1.5	3.0		4.6

Note: Sandy gravel curve is recommended test result in DB001-94 code. The bedrock parameter originates from the rock test parameter which is given by Schnabel(1973).

Table4.2 5 times key parameters inversion results of sand and clay

Times	Sand			Clay			Correlation coeff. $C_{xy}(\tau)$		
	$\gamma_r/10^{-4}$	$\lambda_{\max}$	M	$\gamma_r/10^{-3}$	$\lambda_{\max}$	M	Ethq.1	Ethq.2	Mean
1	6.10	0.32	1.13	4.43	0.13	0.42	0.7840	0.6684	0.7262
2	6.01	0.31	1.15	4.23	0.14	0.41	0.7825	0.6682	0.7254
3	6.17	0.29	1.09	4.37	0.12	0.42	0.7840	0.6725	0.7283
4	6.09	0.30	1.10	4.40	0.11	0.44	0.7845	0.6968	0.7407
5	6.14	0.32	1.12	4.29	0.13	0.41	0.7841	0.6805	0.7323
Final	6.102	0.308	1.118	4.344	0.126	0.420	*0.7838*	*0.6899*	*0.7369*

The inversion parameter's experience value range size has an important impact on the accuracy and reasonableness, the efficiency of calculation etc. It has been given a greater range of experience in the Front. To further reduce the space of inversion, the inversion computation carries on administrative levels. The initial experimental stage, parameter area is classified. a representative of several inversion of the combination format, the precision and accuracy parameters of inversion can also be appropriate to reduce, Several types or combinations with a greater correlation coefficient were always able to achieve. Then, the combination of test results, conducted a detailed analysis of inversion, this time selects the parameter precision also correspondingly enhances. As follows, given the parameter area is detailed inversion scope in the second stage. The sand: $\gamma_r/10^{-4}\in[1,10]$, precision is $0.01/10^{-4}$; $\lambda_{\max}\in[0,0.8]$, precision is 0.001; $M\in[0.1,1.5]$, precision is 0.005; The clay: $\gamma_r/10^{-3}\in[1,10]$, precision is $0.01/10^{-3}$; $\lambda_{\max}\in[0,0.8]$, precision is 0.001; $M\in[0.1,1.5]$, precision is 0.005.

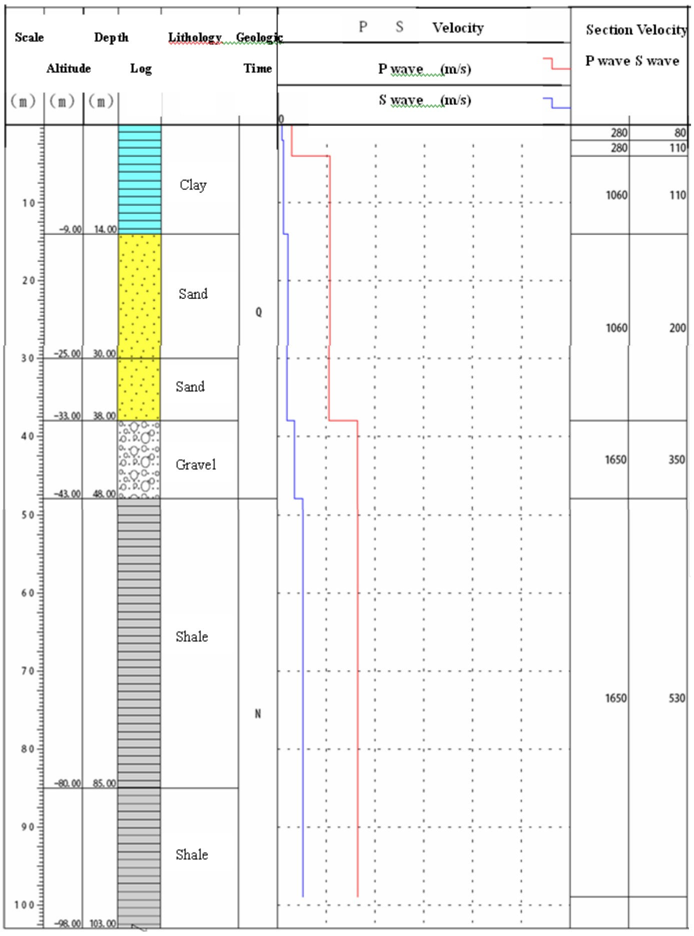

Fig.2 Soil and rock condition of TKCH07 site

For the two events, in the soil test, $G/G_{\max}$ and damping ratio λ curves as a function of shear strain are considered changeless. Thus, in respect of a set of nonlinear parameters, the program needs to be operated twice for the two earthquakes to attain the correlation coefficients and make the target function value smallest (convergence accuracy is 0.30)in the end.

Seed and Idriss summarized and analyzed foregone research results for sand and clay, and attempted a lot of soil tests, suggesting modulus reduction curves for sand(Seed & Idriss，1970) and clay(Sun & Seed,1988), as a function of shear strain and damping ratio curves for sand(Seed & Idriss，1970) and clay(Seed & Idriss，1970), as a function of shear strain. The modulus reduction curves are shown in Fig. 5, and the damping ratio curves are illustrated in Fig. 6 and Fig. 9 respectively. In Japan, typical clay research result is Kokusho's soil test. Kokusho earlier attempted a lot of tests. Because the instruments(high-quality dynamic triaxial apparatus) was advanced, and many factors were considered, the result was reliable. The result was compiled in textbooks in many countries, and widely recognized in the field of international soil dynamics. Where soil test cannot be attempted, the result usually is used instead. Because of the reason, we compared our inverse result with the corresponding soil test result.

It is noticeable that the modulus reduction and damping ratio inverse curves for sand and clay, as a function of shear strain, ultimately correspond to the result of sand test of Seed-Idriss(1970) and clay test of Kokusho respectively. However, the damping ratio for clay is higher than test value when shear strain varies from 10^{-3} to 10^{-2}, and the other shear strain basically accords with test value. Nevertheless, as compared with the curve of Seed-Idriss, the damping ratio of clay still corresponds to the lower limit of the result of Seed-Idriss. In general, modulus reduction and damping ratio curves for clay and sand from our research is close to the result of soil test.

In order to express comparability between inverse results and observation results, anti-plane vibration time history of rock and surface about M_J7.1 earthquake, comparison of time history recorded on surface with corresponding theoretic time history, and theoretic displacement and velocity time history about the surface are showed in Fig. 7 to 8. Moreover, with M_J7.1 earthquake, hysteretic nonlinear constitutive relation inverted in time domain at the depth of 37m of the sand is illustrated in Fig. 9.

5. DISCUSSION AND CONCLUSION

By detailed theoretical analysis, we know that: In the course of two earthquakes, the largest shear strain appeared at the bottom of the sand layer in M_J8.0 earthquake, the value is 0.0045. the greatest value of shear strain of clay also come forth at the bottom layer of clay, its value was 0.0055. Moreover, in M_J7.1 earthquake, the largest sand layer strain is 0.0012, the biggest clay-shear strain is 0.0006.

Some basic recognitions have been concluded by analyzing Fig. 3 to 9 and Table 4.1 to 4.2.

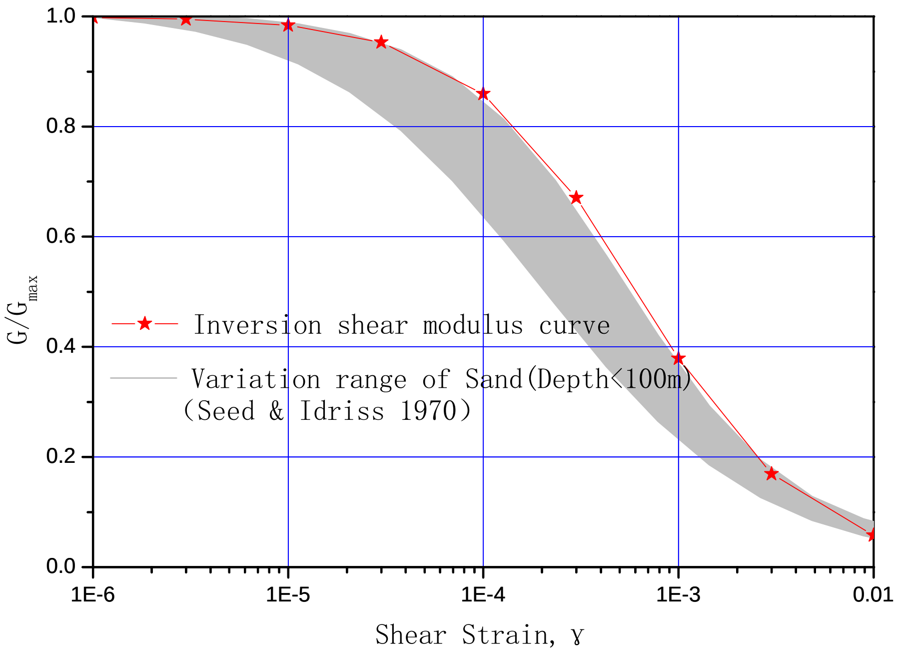

Fig.3 Inversion shear modulus results of sand in TKCH07 site

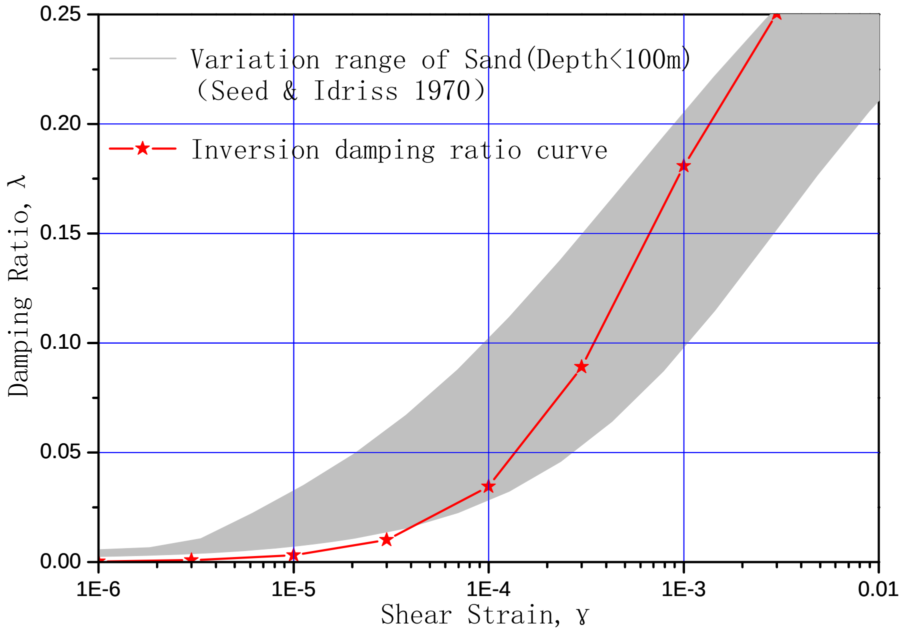

Fig.4 Inversion damping ratio of sand in TKCH07 site

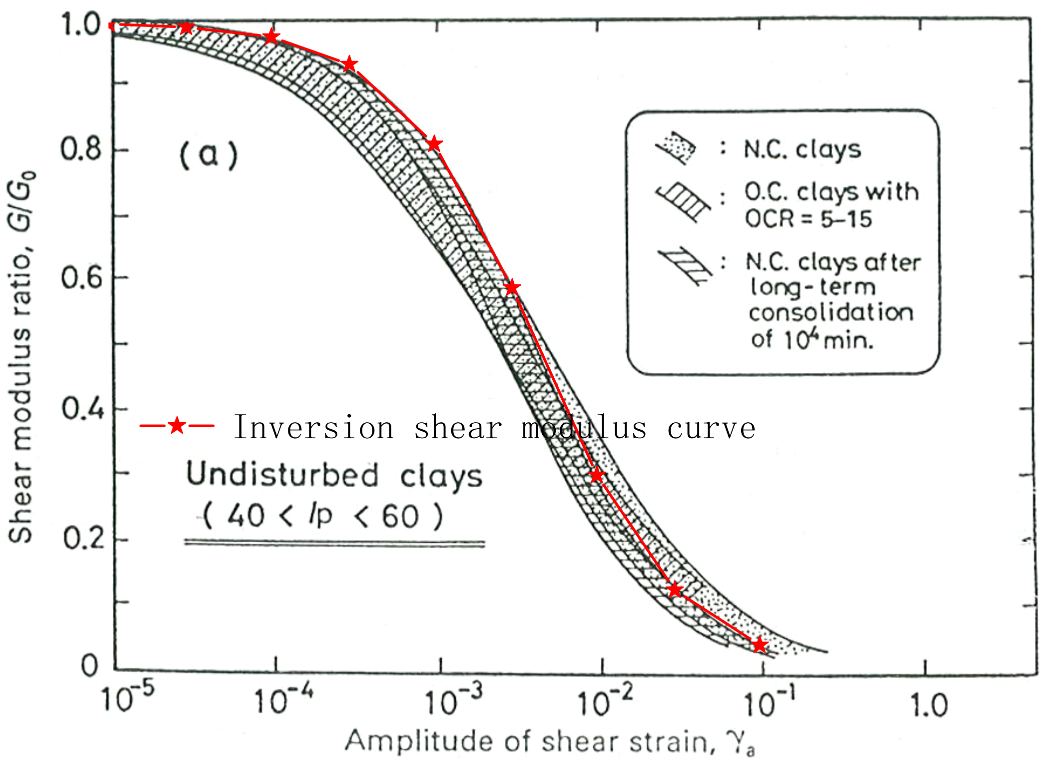

Fig.5 Inversion shear modulus curve of clay in TKCH07 site

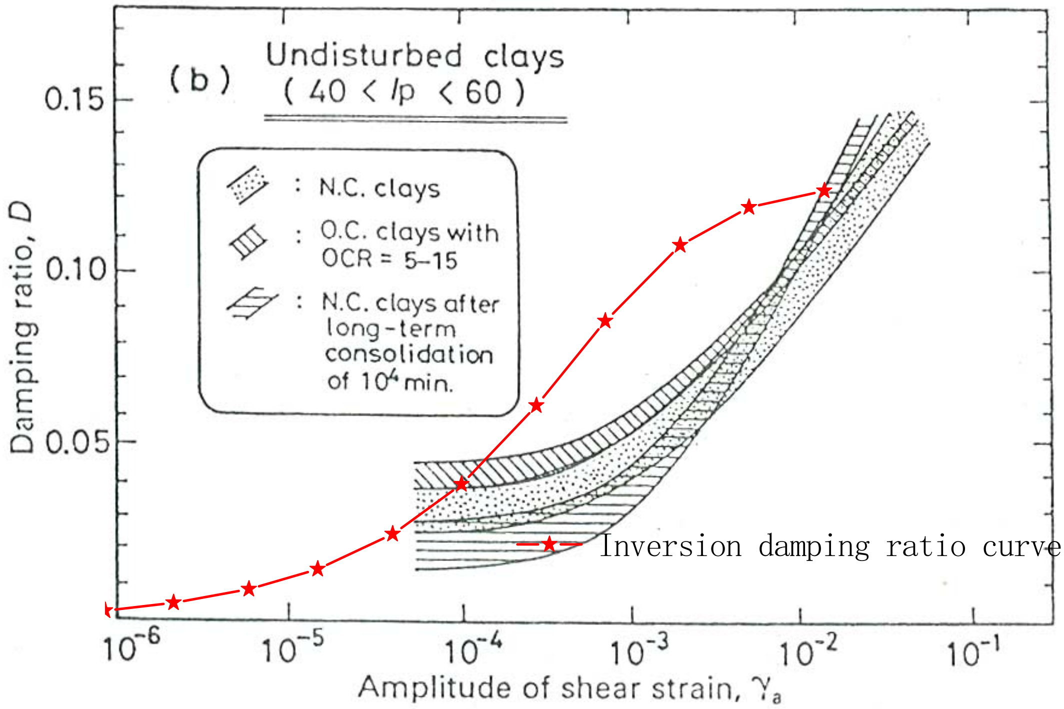

Fig.6 Inversion damping ratio curve of clay in in TKCH07 site

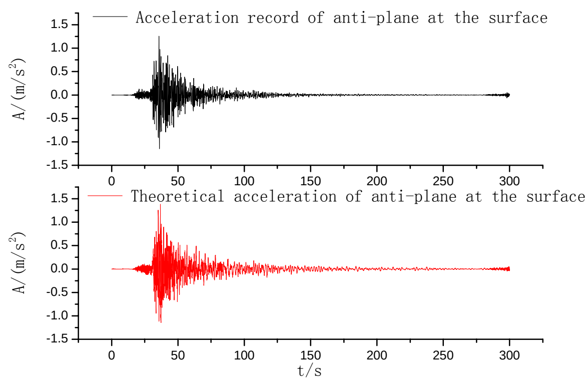

Fig. 7 Theoretical value and the actual observations of the surface at TKCH07 site in M_J7.1 earthquake

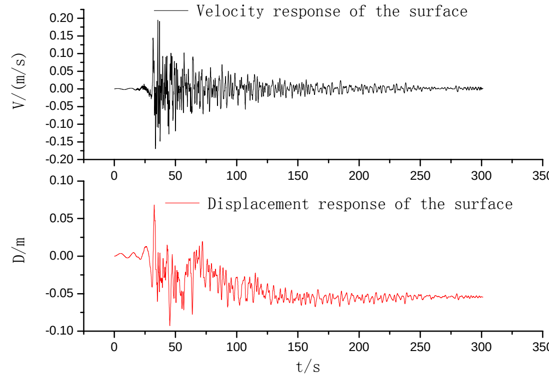

Fig. 8 Theoretical velocity and displacement history of the surface at TKCH07 site in M_J7.1 earthquake

1. Nonlinear soil behavior on shearing strain can be inverted when large-scale strain occurred during a strong ground motion. Inverted modulus reduction curves for sand and clay，as a function of shear strain, are similar to the soil test curves. And the damping ratio of sand corresponds to the test result, but there are some differences between the damping ratio of clay and Kokusho damping ratio which varies from 10-3 to 10-2. The damping ratio of clay approximately corresponds to the lower limit of the result of Seed-Idriss(1970). In one word, inverse result basically accords with the soil test result.

2. The seismogram comparability, hysteretic nonlinear constitutive curve in time domain, and final inverse result show that soil dynamic double-parabola nonlinear constitutive model is be fit for the inversion of in-situ soil nonlinear dynamic parameter, that hysteretic nonlinear inversion in time domain by using many strong ground motion records is feasible, and that the inverse result can strongly help soil test.

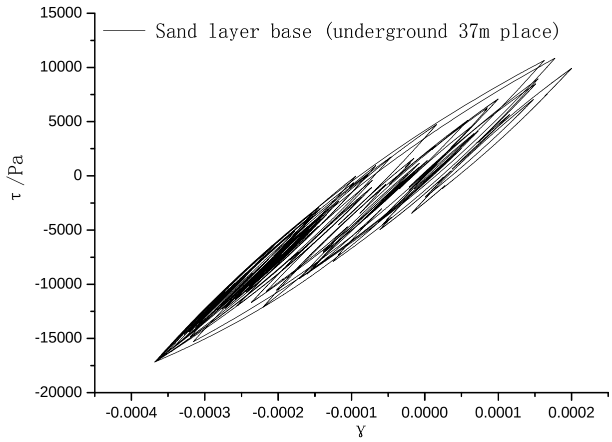

Fig. 9 Inversion nonlinear hysteretic constitutive relations in time domain at the bottom of sand layer (underground 37m place) in M_J7.1 earthquake

3. Soil dynamic inversion of hysteretic nonlinear site in time domain can rebuild soil dynamic constitutive history of hysteretic nonlinear site in time domain when anti-plane strong ground motion occurs, predict and evaluate site velocity time history, and disclose site displacement time history and mechanism of static displacement.

REFERENCES

Loh, C.H. and C. S. Yeh. 1992. Identification of site response using 3-D array records[C]. *10th WCEE*, Balkema, Rotterdam. ISBN 90 54 10 060 5.

CHEN Xue-liang, JIN Xing, TAO Xia-xin. et al. 2008. One soil dynamic double-parabola constitutive model considering the effect of soil test damping, *Rock and Soil Mechanics*, **29:8,** 2102-2110. (*In chinese)*

HAN Wei. 2000. A global-local optimization algorithm and its application in the pile bearing capacity Inversion[D]. Harbin: Institute of Engineering Mechnaics, China Earthquake Administration.(*In chinese)*

Hardin, B.O., Drnevich, V.P., 1972a. Shear modulus and damping in soil: measurement and parameter effects[J]. *Journal of the Soil mechanics and Foundation Engineering Division, ASCE,* **98:6,** 603-624.

Hardin, B.O., Drnevich, V.P., 1972b. Shear modulus and damping in soil: design equations and curves[J]. *Journal of the Soil mechanics and Foundation Engineering Division, ASCE,* **98:7,** 667-692.

ZHANG Ke-xu, XIE Jun-fei. 1989. Soil dynamics, Beijing: Earthquake press. (*In chinese)*

Kik-net. http://www.kik.bosai.go.jp/kik/index_en.shtml