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Prediction and evaluation of measured seismic responses of the Hualien LSST model structure

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ABSTRACT : This paper deals with the prediction and the evaluation of the measured seismic responses of the Hualien large-scale seismic test soil-structure system. The predicted analysis was carried out for the model structure by the computer code SASSI utilizing soil properties derived from geotechnical investigations and correlation analysis of recorded earthquake responses of soil. Utilizing the soil properties, seismic responses were predicted and compared with measured ones. The nonlinear effects of soil on structural responses were also evaluated.

1. INTRODUCTION

A soil-structure interaction (SSI) experiment is being conducted by an international consortium in a seismically active region in Hualien, Taiwan [9]. To obtain earthquake data for quantifying SSI effects and providing a basis to benchmark analysis methods, a 1/4-th scale cylindrical concrete containment model similar in shape to that of a nuclear power plant containment was constructed in the field where both the containment model and its surrounding soil, surface and sub-surface, are extensively instrumented to record earthquake data. In between September 1993 and May 1996, fifteen earthquakes with Richter magnitudes ranging from 4.2 to 6.4 were recorded. The Hualien site contains competent, gravelly soils which are similar to some of the prototypical nuclear power plant foundation soils.

In this study, seismic responses of the model structure in Hualien were predicted utilizing the recorded earthquake response data at the site. The predicted analysis was carried out using the computer code SASSI based on a three-dimensional flexible volume substructure method [4]. Soil properties derived from the field geophysical and geotechnical investigations, and earthquake records at the site based on the approach reported by Elgamal et al. [1] were used. Utilizing the soil properties, seismic responses were predicted and compared with measured ones. The nonlinear effects of soil on structural responses were also evaluated.

2. HUALIEN LARGE-SCALE SEISMIC TEST

The Hualien site is located along the east coast of Taiwan, south of Lotung. The general geology in Hualien consists of massive unconsolidated, poorly bedded conglomerate composed of pebbles varying in diameters from 10 cm to 20 cm. Geophysical and boring tests conducted in 1989 by the Taiwan Power Company (Taipower) show that the shear wave velocity for the top 100m layer of soil is around 400 m/sec and for the layer below (up to about 7 km depth) is 1500 m/sec to 1850 m/sec. The 50 m boring revealed that the top 5 m is of silty sand and the layer below consists of gravels varying in diameters from 3 cm to 7 cm

[9]. More detailed geophysical and geotechnical investigations were subsequently carried out by the Central Research Institute of Electric Power Industries (CRIEPI) [5].

The Hualien test model dimensions are 10.52m in diameter and 16.13m in overall height. The model is roughly 1/4 th scale of a typical nuclear reactor cylindrical concrete containment. Similar to actual construction, about 1/3 of the model is embedded. During construction, the site was excavated to 5m depth to the gravelly layer on which the model was situated. After conducting forced vibration test on the free standing test model, the excavated part was backfilled to result in the final embedded testing configuration. The forced vibration test was conducted prior to earthquake monitoring to study the dynamic characteristics of the soil-structure system under low level excitations.

The test model and its surrounding areas were extensively instrumented. The ground surface has accelerometers on three arms radiating from the test model to a radius of 5.5 times the model diameter (Fig. 1). Three downholes were deployed with accelerometers at depths of 5.3 m, 15.8 m, 26.3 m, and 52.6 m under stations A15, A25 and A35. Fig. 2 shows accelerometers installed at the roof, the base and mid-sections of the wall of the test model.

3. PREDICTION AND EVALUATION OF SEISMIC RESPONSES

3.1 Control Motions

Since September 1993, fifteen earthquakes with Richter magnitudes ranging from 4.2 to 6.4 have been recorded at the test site. Among the earthquakes, three earthquake events were selected for the prediction of seismic responses : events LSST 2 (January 20, 1994 earthquake), 7 (May 1, 1995 earthquake) and 8 (May 2, 1995 earthquake). LSST 7 has the largest peak ground acceleration and LSST 2 is one of weak events. LSST 8 is a moderate event with clear shapes of cross-correlation functions for deriving the earthquake soil model. The north-south (NS) and the east-west (EW) components of free field ground surface motions of station A25 were used as control motions.

3.2 Analytical Modelling

Seismic responses of the model structure were predicted using the computer code SASSI based on a three-dimensional flexible volume substructure method [4]. The soil-structure system was modelled with a quarter model using two planes of symmetry. The structural model consists of beam elements, eight-node solid elements, and rigid links as shown in Fig. 3. The near field soil zone was modelled with eight-node solid elements.

Two kinds of soil properties for the analysis were provided by CRIEPI [5, 6] on the basis of geotechnical investigation, which were called the "unified" and the "modified" soil models, respectively. In this study, a third soil model which was directly derived from earthquake records at the site based on approach reported by Elgamal et al.[1] was also used for the earthquake event LSST 8, since only LSST 8 shows clear shapes of cross-correlation functions for deriving the earthquake soil model. This model is called "earthquake soil model" hereafter. Shear wave velocities in the earthquake soil model were estimated from time lags of cross-correlation functions between the motions of adjacent downhole stations [2]. Table 1 and Fig. 4 summarize the three soil models used in the analysis.

Standard structural properties were provided from the laboratory test by the National Taiwan Univ. as : Young's modulus of concrete $E = 2.88\times10^6$ tonf/m^2 and hysteretic damping $\zeta = 2\%$. However, the correlation study of forced vibration test showed that Young's modulus of the wall should be reduced mainly due to the construction procedure of the structure [3, 8]. In this study, the results reported by Luco and de Barros [3] were adopted where $E = 2.0\times10^6$ tonf/m^2 and $\zeta = 2.0\%$.

3.3 Site Response Analysis

Prior to the structural analysis, site response analysis was performed for free field soil using the computer code SHAKE [7] to take into account the primary soil nonlinearity. The results have shown that the deconvolution analysis using free field ground surface motions as control motions resulted in good agreement with downhole measurements. Table 2 presents variations of shear wave velocities at soil layers due to the primary soil nonlinearity. The table shows that maximum shear wave velocity variations are 17.9% (23% in terms of shear modulus) for LSST 7, 9.0% (17% in terms of shear modulus) for LSST 8 and 5.9% (11% in terms of shear modulus) for LSST 2. The damping value variation has been much more than that of the soil stiffness.

3.4 Seismic Response Analysis

The free field soil properties calculated from site response analysis were used in the seismic response analysis for the unified and the modified soil models. Shear wave velocities at the near field soil were scaled down by shear wave velocity reduction at each free field layer from site response analysis. Damping values from the site response analysis were used for the near field. Earthquake soil model was used in the seismic analysis without site response analysis since the properties in the model were in situ properties in the earthquake event.

Table 3 shows the analysis cases performed in this study, and amplitudes and peak frequencies of Fourier amplitude ratios (transfer functions) of structural responses to free field ground surface motions. From the table, it can be seen that the peak frequencies of Fourier amplitude ratios are reduced by 6-8% and 14-23% due to soil model variation from the unified one to the modified one and from the modified one to the earthquake one, respectively. The amplitudes increase in all cases. The analysis case (4) in the table is the case in which seismic response analysis was performed without site response analysis in order to investigate the effects of the primary soil nonlinearity. The nonlinear effects on the peak frequencies are 6-8% for LSST 7, 2-4% for LSST 8 and 2-3% for LSST 2. This indicates that the effects in the event of larger earthquake are larger than those in smaller one. Comparing with site response analysis results, the variation of the fundamental system frequency due to the soil nonlinearity is less than half of maximum soil stiffness variation in terms of shear wave velocities.

Figs. 5-7 compare the measured and calculated response spectra (5% damped) of roof responses. These figures show that the modified soil model gives better estimation of structural responses than the unified one. Seismic response analysis without site response analysis, i.e. the analysis with neglecting the primary soil nonlinearity, poorly estimates the responses. From Fig. 7(d), it can be seen that the calculated response spectra from the analysis with using the earthquake soil model show good agreement with measured ones. This indicates that it is needed to re-investigate soil properties carefully.

4. CONCLUSIONS

In this study, seismic responses of the model structure in Hualien were predicted utilizing the recorded earthquake response data at the site. Based on the results of the study, the following observations are made :
(1) Variation of the fundamental system frequency due to the primary soil nonlinearity is less than half of maximum soil stiffness variation in terms of shear wave velocities.
(2) The peak frequencies of Fourier amplitude ratios are reduced by 6-8% and 14-23% due to soil model variation from the unified one to the modified one and from the modified one to the earthquake one, respectively. The amplitudes increase in all cases.

(3) The modified soil model gives better estimation of structural responses than the unified one.
(4) The seismic response analysis with neglecting the primary soil nonlinearity, poorly estimates the responses.
(5) The calculated response spectra from the analysis with using the earthquake soil model show good agreement with measured ones. This indicates that it is needed to re-investigate soil properties carefully.

ACKNOWLEDGMENTS

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Table 1 Soil models used in seismic response analysis

Soil Region(1)	Unit Weight (g/cm^3)	Poisson Ratio	Unified Model(2) Vs(4) (m/sec)	Unified Model(2) h(5) (%)	Modified Model(2) Vs	Modified Model(2) h	Earthquake Model(LSST 8)(3) NS Comp. Vs	NS Comp. h	EW Comp. Vs	EW Comp. h
sand 1	1.69	0.38	133	2.0	133	2.0	177	2.5	177	2.1
sand 2	1.93	0.48	231	2.0	231	2.0	177	3.5	177	2.5
gravel 1	2.42	0.48	383	2.0	383	2.0	300	3.2	248	2.9
gravel 2	2.42	0.47	333	2.0	333	2.0	300	3.2	248	2.9
gravel 3(U)(6)	2.42	0.47	476	2.0	476	2.0	300	3.3	248	3.7
gravel 3(L)(7)	2.42	0.47	476	2.0	476	2.0	420	2.4	307	2.9
backfill 1	2.33	0.38	400	2.0	300	2.0	151	2.5	151	2.1
backfill 2	2.39	0.48	400	2.0	300	2.0	151	3.5	151	2.5
backfill 3	2.39	0.48	400	2.0	250	2.0	151	3.5	151	2.5

Note : (1) Refer to Fig. 4 for soil region designations.; (2) The soil properties in these models are initial values prior to site response.; (3) Shear wave velocities in the model were derived from correlation analysis of earthquake records. Damping values were obtained from site response analysis utilizing damping-shear strain relationships with shear moduli being constant.; (4) Vs = shear wave velocity; (5) h = hysteretic damping; (6) upper layer of gravel 3; and (7) lower layer of gravel 3.

Table 2 Shear wave velocity variations from site response analysis

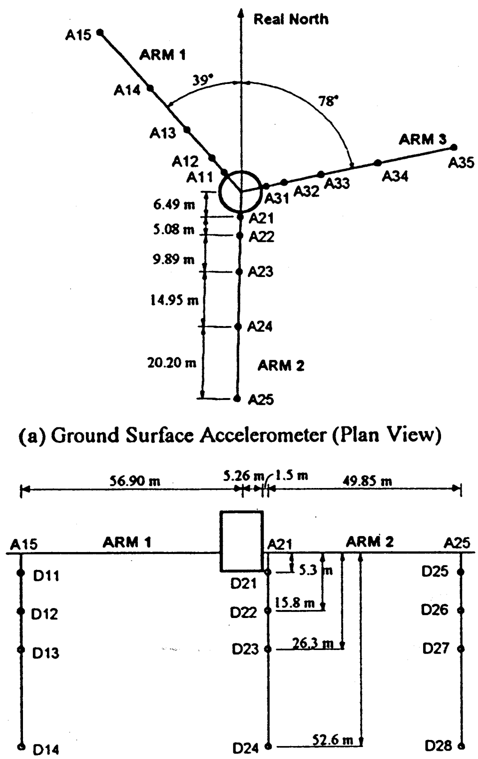
Depth (m)	LSST 2 NS comp.	LSST 2 EW comp.	LSST 7 NS comp.	LSST 7 EW comp.	LSST 8 NS comp.	LSST 8 EW comp.
0-2	128 (-3.8)(1)	127 (4.5)	117 (-12.0)	123 (-7.5)	121 (-9.0)	129 (-3.0)
2-5	222 (-3.9)	221 (4.3)	207 (-10.4)	211 (-8.7)	217 (-6.1)	226 (-2.2)
5-12	318 (-4.5)	310 (6.9)	286 (-14.1)	286 (-14.1)	311 (-6.6)	325 (-2.4)
12-15.8	472 (-0.8)	463 (2.7)	445 (-6.5)	440 (-7.6)	471 (-1.1)	475 (-0.2)
15.8-26.3	463 (-2.7)	456 (4.2)	432 (-9.2)	423 (-11.1)	468 (-1.7)	467 (-1.9)
26.3-52.6	452 (-5.0)	448 (5.9)	418 (-12.2)	391 (-17.9)	464 (-2.5)	453 (-4.8)

Note : (1) property variation between before and after deconvolution analysis (%)

Table 3 Amplitude and peak frequencies of Fourier amplitude ratios for various analysis cases (roof/free field ground surface)

Analysis Case(1)	LSST 2 NS Comp. A(2)	LSST 2 NS Comp. F(3)	LSST 2 EW Comp. A	LSST 2 EW Comp. F	LSST 7 NS Comp. A	LSST 7 NS Comp. F	LSST 7 EW Comp. A	LSST 7 EW Comp. F	LSST 8 NS Comp. A	LSST 8 NS Comp. F	LSST 8 EW Comp. A	LSST 8 EW Comp. F
(1) U	4.415	6.787	4.300	6.738	3.666	6.494	3.819	6.543	4.143	6.690	4.567	6.836
(2) M	5.184	6.348	5.027	6.299	4.233	6.006	4.403	6.104	4.856	6.250	5.396	6.397
(3) E	-	-	-	-	-	-	-	-	5.962	5.371	5.461	4.932
(4) M+L	5.690	6.494	5.690	6.494	5.690	6.494	5.690	6.494	5.690	6.494	5.690	6.494

Note (1) U : unified soil model (CRIEPI); M : modified soil model (CRIEPI); E : earthquake-derived soil model (this study); L : without site response analysis by SHAKE, (2) amplitude; and (3) peak frequency (Hz)



(a) Ground Surface Accelerometer (Plan View)

(b) Downhole Accelerometer (Elevation View)

Fig. 1 Ground Accelerometers

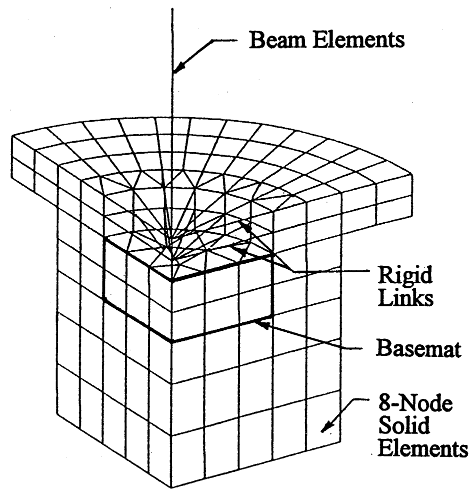


Fig. 3 Mathematical model (SASSI)

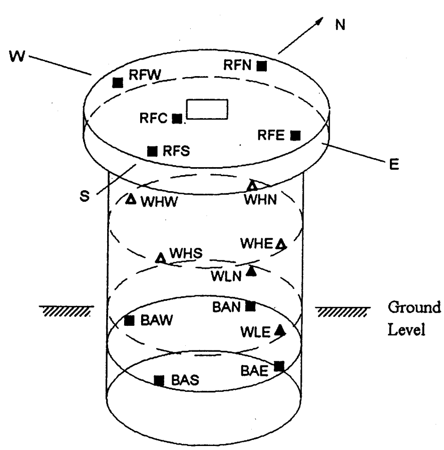


Fig. 2 Structural instrumentation

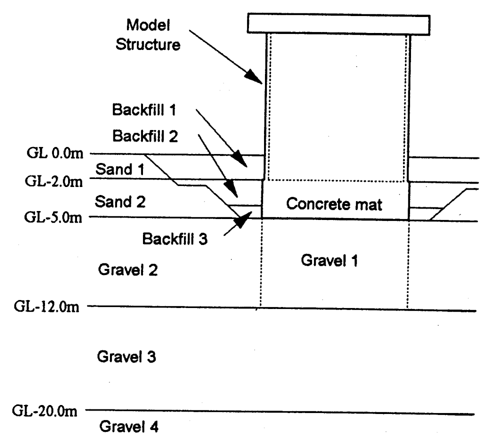


Fig. 4 Soil profile

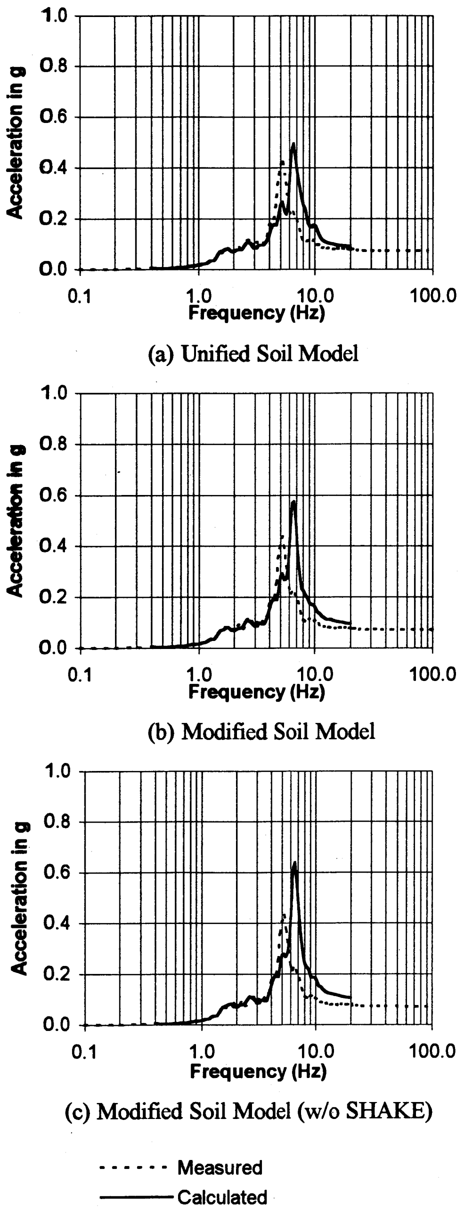


Fig. 5 Measured and calculated roof response spectra (5% damped) for NS component in LSST 2

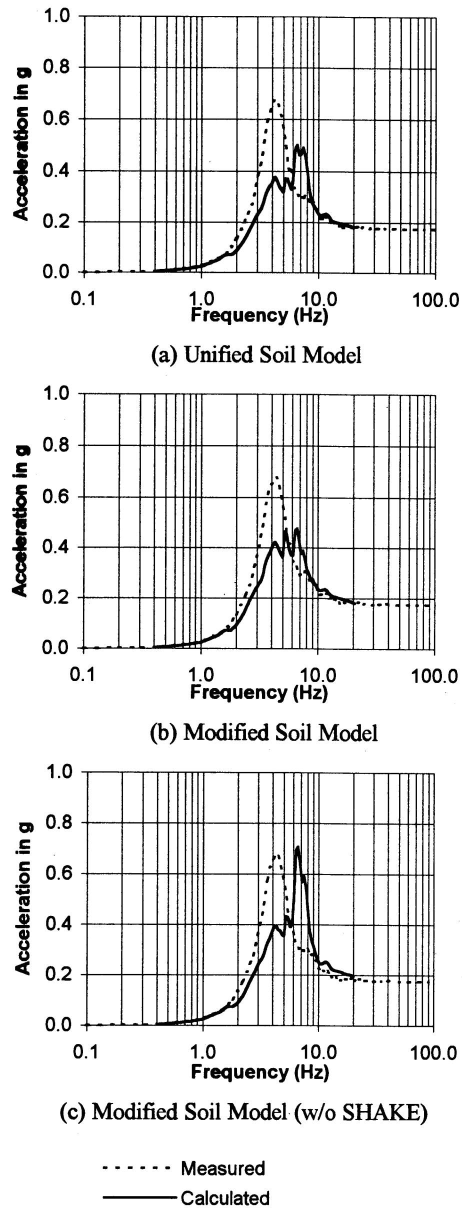
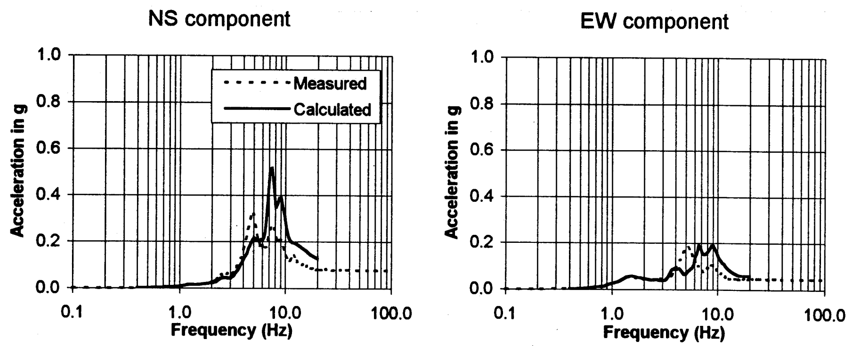
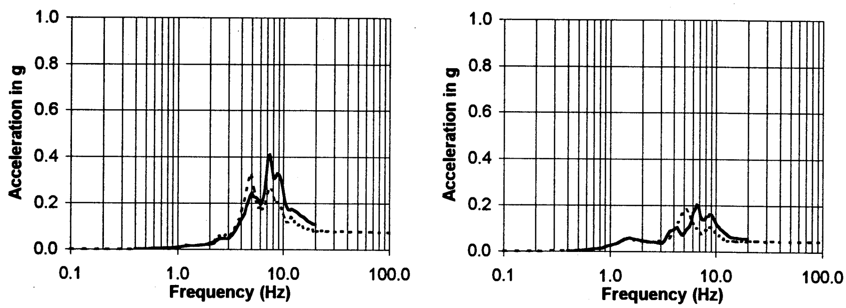


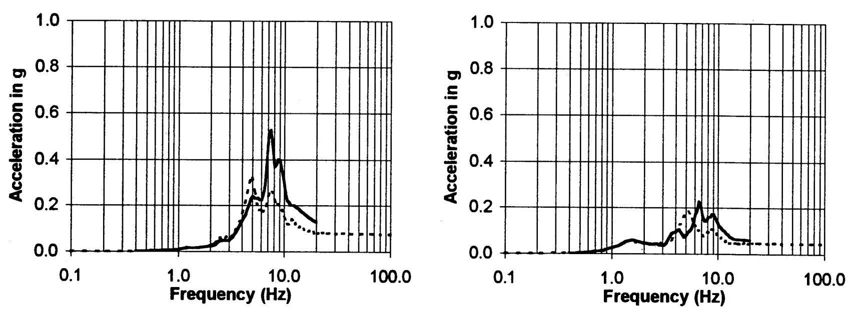
Fig. 6 Measured and calculated roof response spectra (5% damped) for NS component in LSST 7



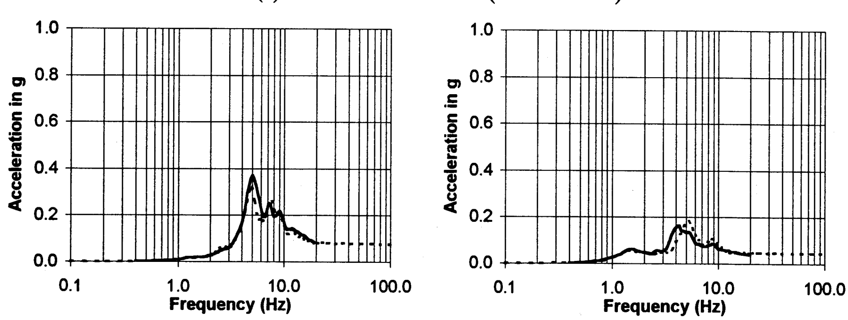
(a) Unified Soil Model



(b) Modified Soil Model



(c) Modified Soil Model (w/o SHAKE)



(d) Estimated Soil Model

Fig. 7 Measured and calculated roof response spectra (5% damped) for LSST 8