Hualien 대형지진시험 모델의 지진응답해석

Estimation of Seismic Responses of Hualien LSST Model By the Substructure Method of Soil-Structure Inraction Analysis

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국문요약

주요구조물의 내진설계를 위한 지진응답은 대상 구조물 하부의 지반상태에 따른 영향 즉, 지반구조물 상호작용영향에 의하여 현저한 차이를 보일 수 있다. 본 논문에서는 지진다발지역인 대만의 유연층상지반상에 건설된 대형지진시험모델을 대상으로 실제지진에 대한 응답을 예측하고, 그 결과를 계측치와 비교, 분석하였다. 지진응답예측을 위한 해석을 위해서는 크기와 특성이 서로 다른 세 개의 실측된 지진운동을 입력운동으로 사용하였으며, 해석방법으로서는 진동수 및 시간영역에서의 집중파라메타모델을 이용하는 부분구조법을 사용하였다. 해석결과의 분석을 통해서, 제시된 지반구조물 해석방법이 공학적으로 신뢰할 수 있는 지반구조물 상호작용시스템의 지진응답을 준다는 사실을 확인하였다. 단, 이를 위해서는 해석시 입력운동의 정의 및 뒷채움재의 모델링 등에 있어서 세심한 주의가 전제되어야 한다는 사실도 확인하였다.

주요어 : 지반구조물 상호작용, 부분구조법, 집중파라메탈모델, 대형지진시험모델

ABSTRACT

Seismic responses of the Hualien large scale seismic test model on a layered soil site are estimated for three recorded earthquakes with different level of peak acceleration using two different approaches of soil-structure interaction analysis. The analysis results are then compared and evaluated with the recorded. The method adopted for the analysis is based on substructuring method using a lumped parameter model in both the frequency and time domain. The study results indicate that the proposed method can reasonably estimate the earthquake responses of a soil-structure interaction system for engineering purposes if the techniques of defining input motion and modeling of the backfilled soil are prudently selected.

***Key words** : soil-structure interaction, substructure method, lumped-parameter model, Hualien large-scale-seismic test*

1. Introduction

Dynamic soil-structure interaction(SSI) effects have been investigated for more than 30 years using various types of solution techniques. However, to this day, SSI analysis still constitutes one of the most indefinite steps and major source of uncertainty in the seismic design of critical structures. To improve and resolve this situation, an international research project has been studying SSI analysis since 1990.

This research constitutes an extension of the Lotung project[1] to confirm and expand the findings in Lotung for cases with stiffer soil. It has been sponsored and participated by

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researchers from five countries; Korea, Japan, USA, Taiwan, and France. To facilitate SSI, a quarter-scale containment model of a PWR-type nuclear power plant was constructed at Hualien, Taiwan. Hualien is located south of Louting on the east coast of Taiwan in a highly active seismic zone. The model stands on a layered soil with a relatively higher stiffness than Lotung. About one-third of the model height is embedded to the ground. The properties of the soil around and beneath the structure have been systematically investigated by both in-situ and laboratory tests. The details of the test model are shown in Fig.1.

Two forced vibration tests (FVT), before and after backfill, have been conducted using harmonic input motions with varying frequencies. Blind predictions and post-correlation analyses have been performed for the FVT's. The modified model with the FVT-correlated material properties has been presented in references 2 and 3. After the FVT's, more than ten actual earthquakes have been recorded in the test model with peak ground accelerations varying from 20 to 150 gals.

In this paper, the responses of the SSI analysis model subjected to the major recorded earthquake motions are estimated and compared with the actual recorded responses. Through the review and evaluation of the results, some considerations and limitations of the adopted method are suggested for practical applications. A substructure method of SSI analysis using lumped-parameter model is used for the analysis. In order to determine the sensitivity of the responses to earthquake motions, three representatiive earthquake records with different amplitudes are selected as the target input motion of the analysis. Prior to structural response analysis, free field responses at the basemat level are obtained through a deconvolution analysis of the recorded motion and results are used as input motion for the SSI analysis. The dynamic soil properties relative to the strain levels of the recorded motion are also obtained from the free-field analysis. In the subsequent structural response analysis, acceleration responses at major sensor locations are estimated using the substructure analysis method in both the frequency and time domain. Analysis results are then compared with the actual recorded values.

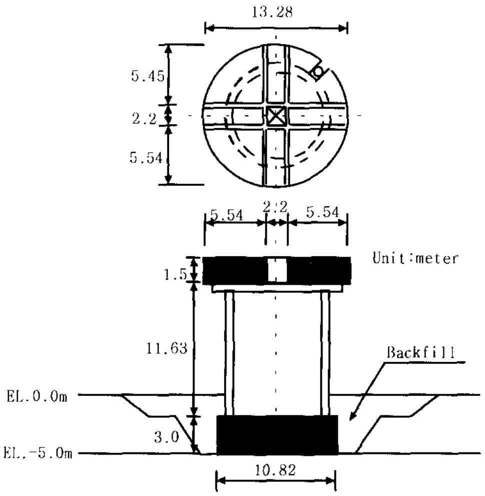


Fig 1 Plan and elevation of the test model

2. Analysis model and input motions

2.1 Analysis Model

At the beginning phase of the project, the best correlated Young's modulus of the superstructure was determined to be 95% of the test value from the correlation study of the FVT's. This reduced material property of the structure was caused by twelve holes made in the upper portion of the shell wall used to support formwork beams of the roof slab during construction. Of course, these holes were filled afterwards, however, they resulted in the overall stiffness of the shell wall being significantly lower than initially intended. Also, from the correlation study using the recorded motion of the strong earthquakes, 0.95Eo was found to be a bit stiffer than the actual value (Refer to Fig.2), where Eo is the test value. The final value of Young's modulus was determined to be 0.80Eo($2.3 \times 10^5 kg/cm^2$) based on results of the parametric studies.[3]

Material properties for the backfill and base soil were adopted from the previous correlation study results of the FVT and shown in Fig.3. These properties are zero-strain values and are used as only starting values for the free-field analysis. Final strain dependent values were determined from the results of the free-field analysis for each input motion and are used in the subsequent response analyses of the SSI model.

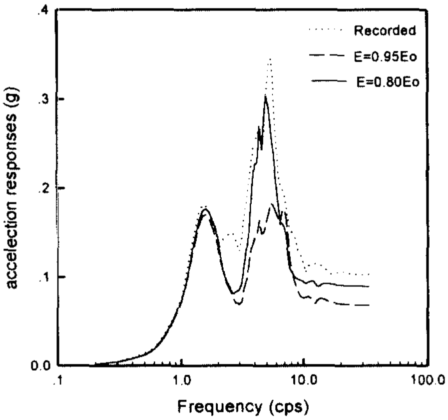


Fig 2 Comparison of analysis results using different values of Young's modulus (horizontal roof responses to May 1, 1995 earthquake)

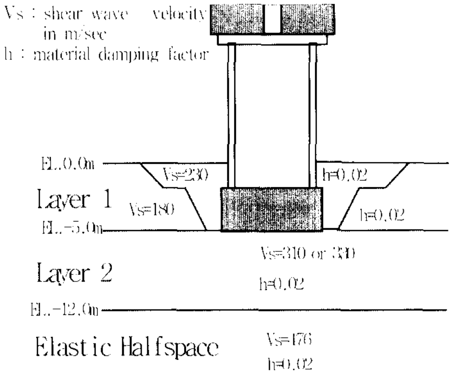
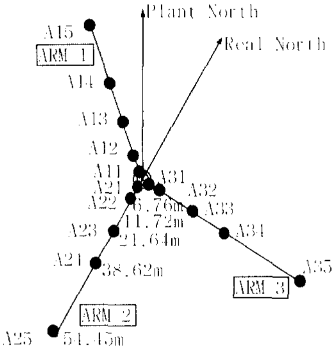


Fig 3 Soil properties at zero strain computed from the FVT correlation studies.

2.2 Input Motions

Among more than 10 earthquakes recorded at the site, three major earthquake motions with different levels of magnitude were selected as the input motions for this study. They are as shown in Fig.4. Among the outermost stations (A15,A25,A35), A35 recording values are significantly smaller than the other two station values, which implies that they include some interaction effects. And A15 values have comparatively more discrepant than the A25 values between the two horizontal orthogonal components. Therefore, the motion at station A25 was selected as the representative free field control motion. Fig.5 shows the time histories and response spectra of the selected input motions.



(a) accelemeter stations at ground surface

STA.No.	LSST 2		LSST 7		LSST 8	
	EW	NS	EW	NS	EW	NS
A13	40.70	29.98	67.0	144.0	33.9	60.6
A14	35.99	23.33	66.8	99.5	36.8	57.4
A15	43.77	32.12	72.0	135.0	62.6	86.9
A23	31.43	27.47	63.7	108.9	28.1	70.4
A24	34.90	32.98	63.4	119.6	29.9	72.0
A25	36.36	39.36	71.0	116.0	36.3	88.5
A33	32.99	22.60	57.2	81.3	21.3	37.3
A34	38.49	26.88	71.9	77.0	24.3	31.8
A35	48.77	26.16	69.2	84.0	27.9	30.8

(b) peak accelerations of the earthquakes at each station

Fig 4 Target earthquakes selected for the analysis

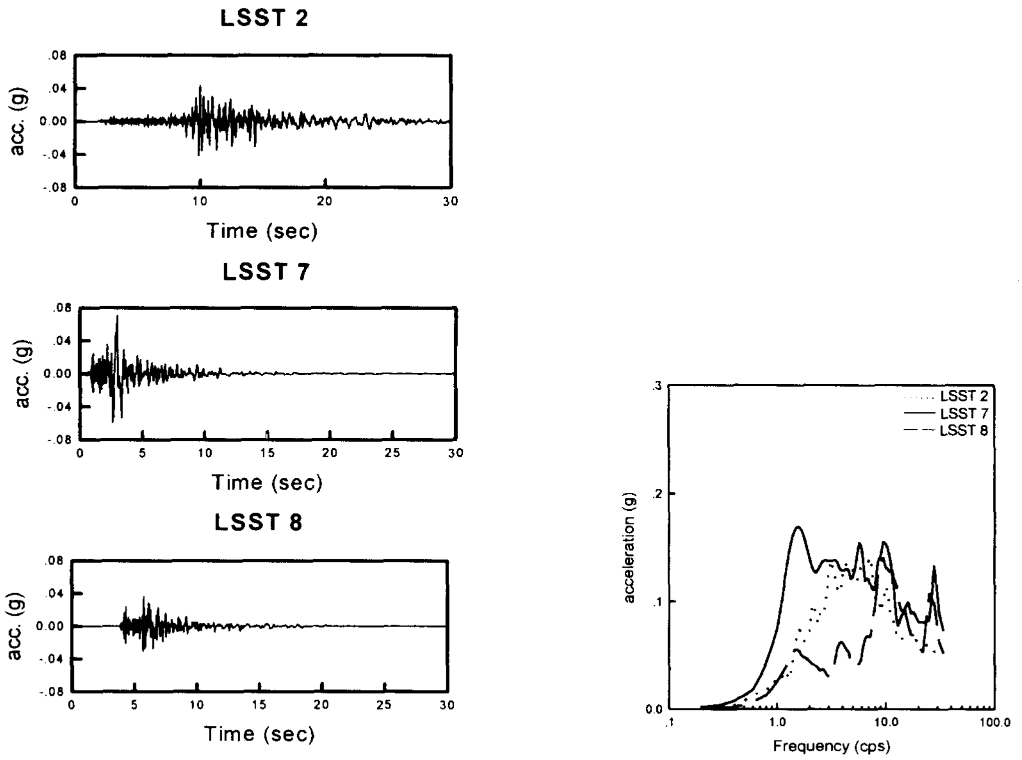


Fig 5 Time histories and response spectra of the control motion(A25) of the three earthquakes selected (EW-component)

3. Free field analysis

Free field analysis is performed for the purpose of obtaining the soil properties compatible to the earthquake induced strain level as well as determining the deconvolved input motions at the basemat level. The computer program MSHAKE[4] based on one-dimensional elastic wave propagation theory was used for the analysis, where nonlinearity of the soil can be considered indirectly by equivalent linear analysis technique. Empirically derived curves of shear modulus and damping ratio vs. shear strain[5] were used for the analysis. The deconvolved motion at each level of the downhole array points showed good agreement with the recorded motions for all the earthquakes selected. Fig.6 shows an example of the results, which is a comparison of the computed and recorded free field response for the EW component motion of LSST7 earthquake. As can be seen, the estimated motion at the basemat level agrees with the recorded satisfactorily. These free field analyses, up to now, have been performed assuming that the properties of the backfilled material are the same as the far-field material. However, the effect of the discontinuity of the surface layer(Layer 1) by the backfill material should also be considered in obtaining more reasonable basemat motions for the subsequent response analyses. Therefore, two different cases were assumed for the backfill effect and the results were compared. In the first case, the backfill effect was indirectly considered using the approximate procedure suggested by EPRI.[6] In the second case, the effect of the backfill material were neglected assuming that the properties of the backfill are the same as the farfield. The final material properties determined from the free field analysis for the subsequent response analyses are shown in Table 1.

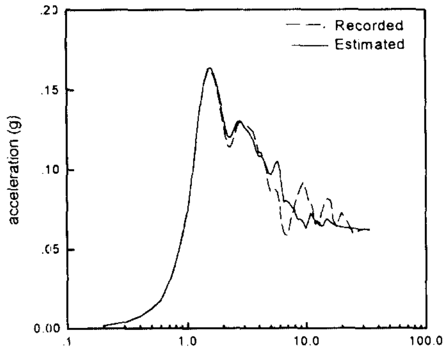


Fig 6 Comparison of the free-field motions at the basemat level (Station A25, EW-component of LSST7)

Table 1 Final values of soil properties determined by free field analysis.

		LSST 7	LSST 8
EW direction	Layer 1	Vs =154m/sec h = 0.031	Vs =164m/sec h = 0.023
	Layer 2	Vs =286m/sec h = 0.048	Vs =326m/sec h = 0.022
	Elastic Halfspace	Vs =411m/sec h = 0.047	Vs =462m/sec h = 0.025
NS direction	Layer 1	Vs =150m/sec h = 0.038	Vs =157m/sec h = 0.028
	Layer 2	Vs =287m/sec h = 0.047	Vs =312m/sec h = 0.031
	Elastic Halfspace	Vs =430m/sec h = 0.038	Vs =466m/sec h = 0.023

4. Structural response analysis

Structural responses of an earthquake in this study have been obtained by the substructure method in both frequency and time domain. In the frequency domain analyses, frequency dependent impedance functions of the base soil are calculated assuming a rigid footing using Green's function based on a semi-analytical approach proposed by Wong.(7) The responses are computed by the typical dynamic complex stiffness matrix method in the frequency domain.(2) Time domain analyses are performed using the modal superposition method, where different modal damping values of the SSI system are computed by Roesset's approach(8) based on the ratio of the dissipating energy to the strain energy of each mode. Frequency independent impedance values are adopted in the time domain analysis, which are iteratively determined by tuning to the fundamental frequency of the SSI system.(2) For these analyses, the soil-structure interaction system of the test model was idealized as a lumped parameter model shown in Fig.7.

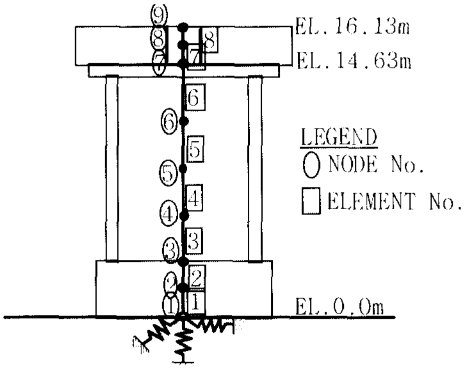


Fig 7 Analysis model of the soil-structure interaction system

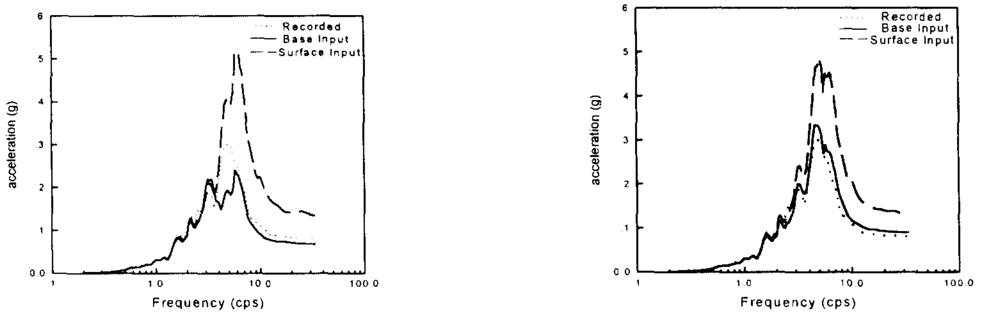
The superstructure consists of nine masses and eight 3-dimensional beam elements. The analysis method of this study includes two major uncertainties in defining input motions at the basemat level. The first uncertainty is that rotational motions at the basemat due to scattering effects are not included in the basemat motions. The second is that the discontinuity of the material property of the surface layer due to the backfill can only be considered by an approximate technique which is not practically validated. For these reasons, the response analyses in this investigation focused on evaluating the impact of these two uncertainties through some comparison studies. The first comparison is between cases using basemat level motions vs. using surface motions as input acceleration time histories. The second comparison is between the case of approximate backfill effect analysis vs. the case of assuming no discontinuity between the backfill and the far-field materials. A comparison between the results of the frequency vs. time domain analysis have also been performed to determine the practical applicability of the time domain analysis which adopts a more simplified procedure. Major analyses results are illustrated in Fig 8 through Fig 10 and are compared with the recorded values, where various comparisons have been made for the different types of analyses discussed above.

Fig 8 shows analysis spectra of responses at the roof level(horizontal direction) for each earthquake compared with the actual values. Specifically, estimated responses using surface input motion are compared with the responses from the basemat level input motion. This figure shows that the general trend of the frequency characteristics of the estimated responses reasonably agree with the recorded responses for all analysis cases. However, the amplitude of the spectral values, for the "surface input analysis" are higher than the recorded values, while the "basemat input analysis" shows an opposite trend. The lower results in the "basemat input analysis" may be due to the fact that the rotational component caused by scattering in the deconvolved basemat level has been neglected. As observed, the degree of this phenomenon becomes more significant as the magnitude of the earthquake is reduced. This indicates that the nonlinearity of the soil behavior due to strong earthquake motions reduces the difference between the surface motion peak amplitude and the convolved motion peak amplitude. From a design point of view, the reduction in the estimated responses can be compensated to some extent by using the surface motions as input motion instead of using an uncertainly deconvolved motion. As can be seen in the figures, surface motion analyses give a conservative response and its conservatism will not so significantly differ from that of a design level earthquake.

The plot in Fig.9 are the response comparisons between the "backfill analysis" and "no backfill analysis". As discribed above, the discontinuity of the surface layer by the existence of the backfill material has been considered by the EPRI procedure in the former analysis, while material properties of the backfill are assumed to be exactly same as the far-field in the latter analysis. It can be observed that the trends of the "backfill analysis" response spectra show very rough correlation with the actual field responses. The "no backfill analysis" always shows comparatively better coincidence in the peak frequency range with reasonable conservatism in the spectral amplitudes. Specifically for the cases of basemat input level analysis, the backfill analysis gives responses which have considerably different trends from the recorded values both in peak frequency and amplitude while the "no backfill analysis" shows responses which agree reasonably well with the recorded values. In general, it can be said that the "backfill analysis" does not improve the responses from the "no backfill analysis. Therefore, engineers should be very careful in adopting the approximate backfill analysis technique because it may change the responses in an unconservative manner.

In Fig.10, similar type response comparisons are made for the frequency domain vs. time domain analysis. As can be seen, time domain analysis using frequency independent impedance values give exactly the same trends of responses in the frequency characteristics. However, the amplitudes of the peak responses are higher than the frequency domain analysis results by 10 ~ 20%. These phenomena are observed in all cases of analyses without exception and might have been caused by the smaller estimation of radiational damping in the time domain analysis where coupled terms between the horizontal and rocking degree-of-freedom have been ignored.

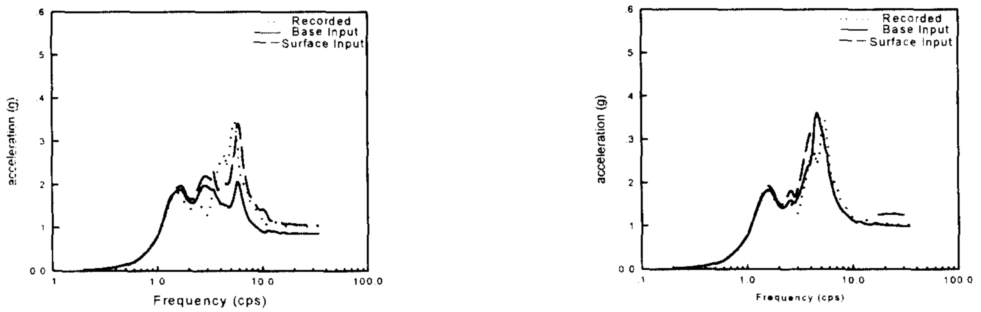
The general trend and observations of the estimated responses examined above from Fig.8 through Fig.10 have only been for the EW response components. However, responses for the NS-component also show very similar trends for all observations.



approximate backfill effect analysis

without backfill effect analysis

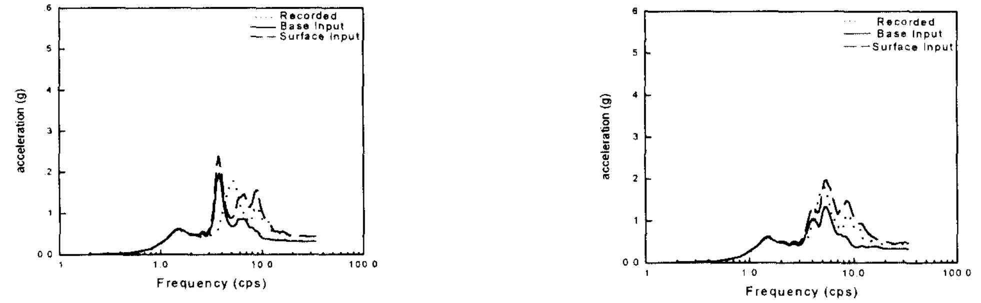
(a) LSST2 (Jan. 20, 1994) earthquake



approximate backfill effect analysis

without backfill effect analysis

(b) LSST7 (May. 1 , 1995) earthquake

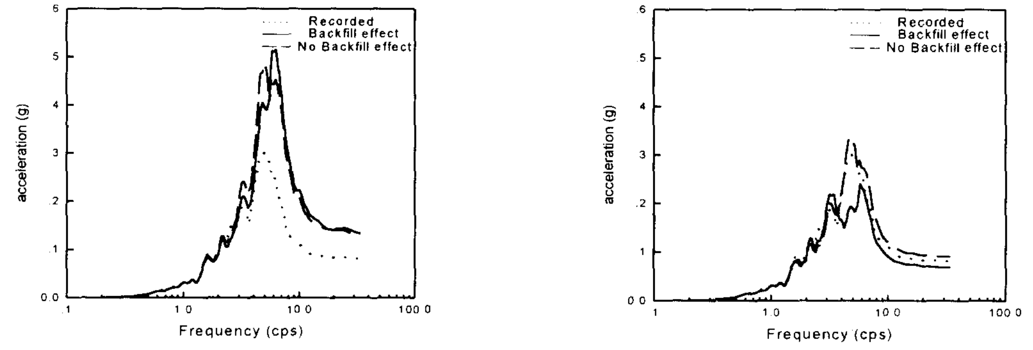


approximate backfill effect analysis

without backfill effect analysis

(c) LSST8 (May. 2, 1995) earthquake

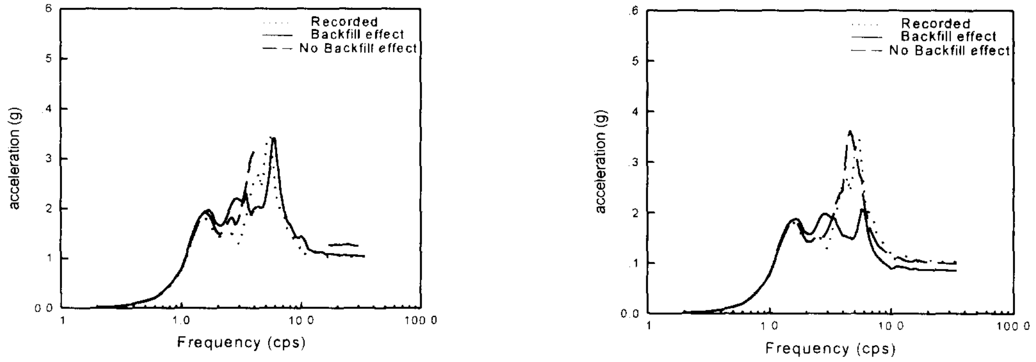
Fig 8 Comparison of response spectra of horizontal roof responses (EW direction) between surface input vs. basemat level input



surface input

basemat level input

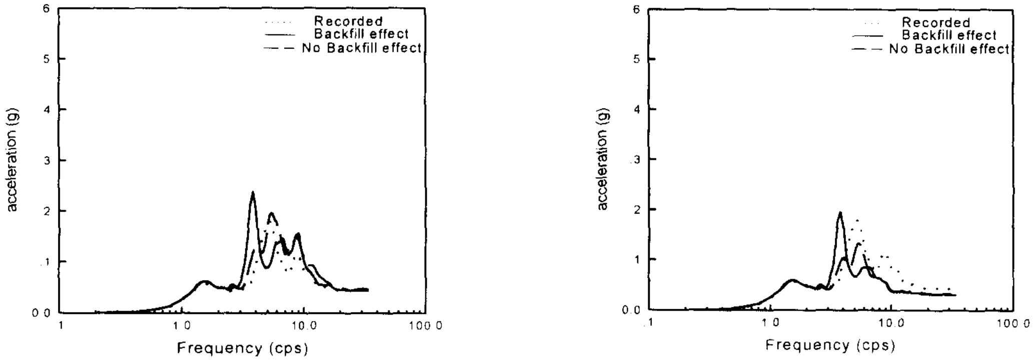
(a) LSST 2



surface input

basemat level input

(b) LSST 7

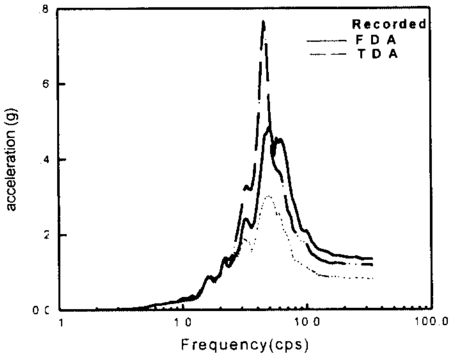


surface input

basemat level input

(c) LSST 8

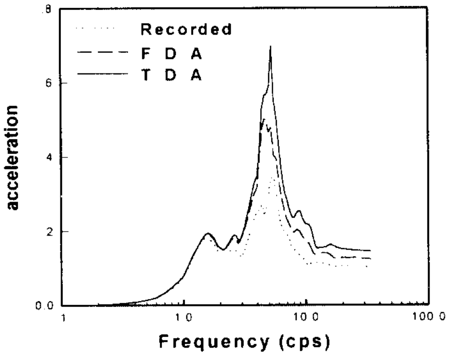
Fig 9 Comparison of response spectra of horizontal roof responses (EW direction) between the "backfill effect analysis" vs. "no backfill analysis"



surface input

basemat input

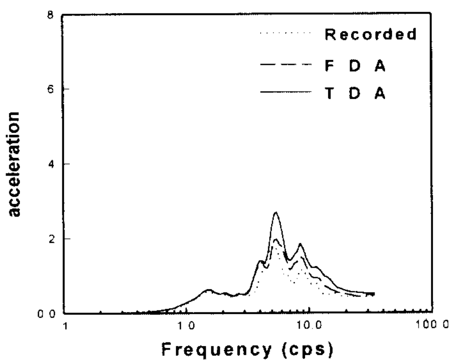
(a) LSST 2



surface input

basemat input

(b) LSST 7



surface input

basemat input

(c) LSST 8

Fig 10 Comparison of horizontal response spectra (at roof, EW direction) between "frequency domain analysis" vs. "time domain analysis"

5. Conclusion

Seismic responses of the Hualien large scale seismic test model on a layered soil site have been estimated using two different types of soil-structure interaction analysis approaches and the results have been compared and evaluated with the recorded values. From the study results, some conclusions have been drawn as follows.

1. The proposed method of soil-structure interaction analysis based on the substructure method using a lumped-parameter model in both the frequency and time domain can reasonably estimate earthquake responses of a soil-structure interaction system for practical design purposes. However, prudent technical consideration is required in selecting input motions and modeling of backfilled soil to obtain reliable results.
2. It has been reconfirmed that the responses are highly underestimated if a deconvolved motion at the basemat level without rotational component is used as input motion. This tendency becomes more pronounced if the magnitude of the earthquake motion becomes smaller. Therefore, surface motion may induce a more reasonable response estimation than the reduced motion at the basemat level.
3. The approximate procedure of modeling backfilled soil proposed by EPRI may considerably underestimate the responses especially in the frequency range of the peak amplitude. Therefore, this procedure must be carefully applied for practical purposes.
4. Time-domain analysis, even though it follows a much simpler procedure using frequency independent impedance, can estimate the responses with the same level of accuracy as and is more conservatism than the frequency domain analysis.

ACKNOWLEDGEMENT

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